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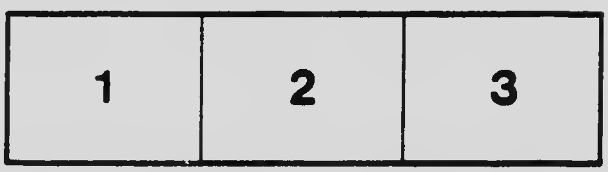
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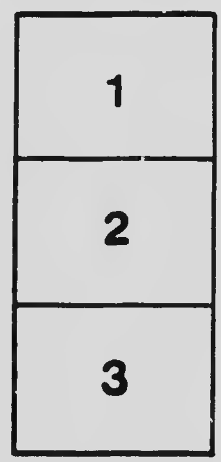
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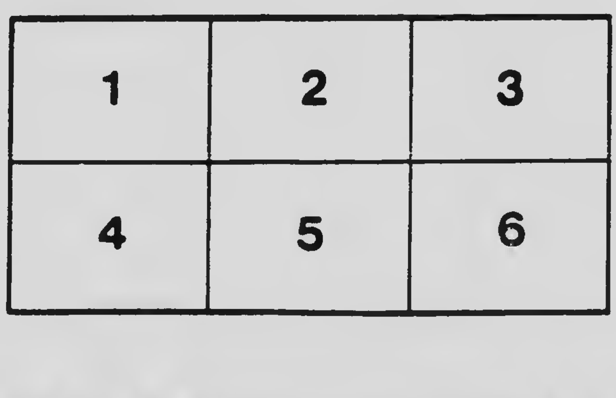
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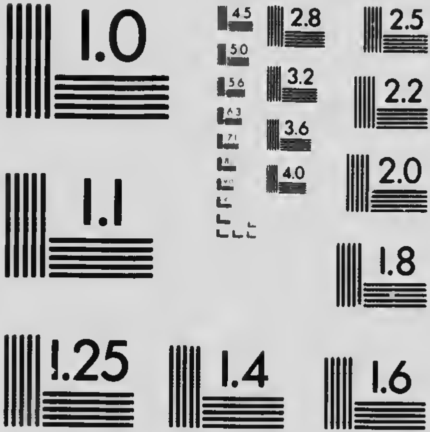

Frontispiece

POTOMAC MEMORIAL BRIDGE, WASHINGTON, D. C.

CONCRETE BRIDGES

AND

CULVERTS

FOR BOTH
RAILROADS AND HIGHWAYS

BY

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CHICAGO AND NEW YORK

THE MYRON C. CLARK PUBLISHING CO.

LONDON

E. & F. N. SPON, LTD., 57 Haymarket

1909

PREFACE.

Bridges of solid concrete are superior to those of any other material. They are as permanent as stone, and have a less cost. Masonry bridges and aqueducts built by the Romans are still standing, and some of them in use. A few old cast iron bridges remain, dating back a century or more, but a majority of the modern ones built of wrought iron and steel have a very limited existence. Forty years ago, steel bridges were believed to be permanent structures, but it is now well known that they do not generally last longer than from twenty to thirty years.

Solid concrete bridges are superior to those in which reinforcing metal is required for resisting tensile stresses in the arch ring. Continuous water-soaking reduces the adhesion of concrete to steel by about 100 per cent, and the effect of shocks and vibration also tends to destroy the bond. It frequently occurs that cracks develop, sufficiently large to admit water, and when water and moisture reach the reinforcing metal, it is then only a few years before the metal is destroyed by rust.

An old wire suspension bridge that recently failed, was examined and reported on by the writer, and it was found that failure occurred because of the rusting and breaking of the wire cables embedded in the anchorage. When the bridge was built, it was doubtless considered that the cables when painted and embedded in concrete, were secure against corrosion. Sufficient caution was not taken to exclude moisture from the anchorages, and the bridge failed as stated above, by the rusting and breaking of the embedded metal. It is evident, therefore, that the most enduring bridges are those of solid masonry, where no metal is required.

Many of the largest masonry bridges built in recent years, have arch rings built of solid concrete, without reinforcing metal for resisting direct stresses. Details of some of these are given in Table No. I. Even in arches with reinforcement, the best designers are now proportioning the arch rings, so the line of pressure for uniform loads will at all times fall within the middle third of the arch ring, and require no reinforcing for these loads.

In the Engineer's Pocket-Book, Mr. Trautwine makes the following statements:—"Nearly all the scientific principles which constitute the foundation of Civil Engineering, are susceptible of complete and satisfactory explanation to any person who really possesses only such knowledge of arithmetic and natural philosophy, as is taught to boys in public schools. The little that is beyond this, may safely be intrusted to the savant. Let *them* work out the results, and give them to the engineer in intelligible language. We can afford to take their word, because such things are their specialty. The object has been to elucidate in plain English, a few important elementary principles, which the savants have enveloped in such a haze of mystery, as to render pursuit hopeless to any but a confirmed mathematician."

Several complete and very comprehensive treatises have already been written, covering the mathematical theory of arches, and as far as this feature of the subject is concerned, there is little left to be desired.

In the preparation of this manual, the effort has therefore been made, to as far as possible eliminate mathematical formulae, and to present the subject in the simplest possible manner. Only such material is given as is directly required in the design and construction of ordinary concrete or masonry arches, so it will be unnecessary for the busy engineer to spend valuable time and thought in the perusal and study of obstruse mathematical treatises. Practicing engineers have but little time for mathematical investigation, and generally must accept formulae as given to them by others.

A real need for this book is believed to exist, owing to the increased use of concrete bridges.

The designs and data tables for culverts and trestles are original with the author, and are here presented for the first time. They are the result of his own practice in the design and construction of railroad structures.

In the preparatio of this manual, I have received valuable assistance from my wife, Maude K. Tyrrell, a graduate of the Chicago Art Institute, and experienced in architectural design. I am indebted also to the following gentlemen for assistance as noted:—To Julius Kahn for two views of concrete trestles, to Whitney Warren, Architect, for views of the proposed Hudson Memorial Bridge, to Messrs. Lea and Felgate for views and drawings of the Rocky River Bridge, to George S. Webster for a photograph of the Walnut Lane Bridge at Philadelphia, and to H. Hawgood for the illustrations and drawings of the Santa Ana Bridge. I am also indebted to the Engineering News for drawings of the proposed Hudson Memorial Bridge, the Spokane and the Grand Avenue Bridges, and to the Engineering Record for two drawings of the Rocky River Bridge and drawing of Edmondson Avenue Bridge.

H. G. Tyrrell.

Evanston, Illinois.
November, 1909.

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PART I.

PLAIN CONCRETE ARCH BRIDGES.

Composition.

Masonry arches were formerly built almost entirely of brick and stone. In recent years, however, owing to the increased production of cement and modern methods of making concrete, including the crushing of stone and the mixing and handling of materials, a large number of our modern bridges are built of concrete. Brick arches lack the bond of stone. They are usually laid in concentric rings, the edge of the brick appearing in the soffit of the arch. Occasionally the bricks have been laid dry, and grout run in to fill solid all cavities. As brick is a softer material than stone or concrete, its use does not appear to have any special advantage. All masonry arches, whether built of brick or stone as block structures, or made of concrete in a solid monolith, carry their loads entirely through compression in the arch ring, and while the mortar joints would doubtless resist considerable tension if so required, no reliance should be placed on the tensile strength of such joints.

Advantages of Masonry Construction.

In many respects a masonry arch is superior to either a steel bridge or a combination of steel and concrete. Some of these advantages may be enumerated as follows:—Cement hardens with age, and consequently the older the bridge, the stronger it becomes. Therefore, if it successfully sustains its first test load it will always be secure. This condition is reversed in steel structures, which deteriorate with age through the action of rust and the loosening of rivets and pins. As travel increases, concrete bridges become stronger to support it; neither is there any yearly expense for painting or other maintenance. They can generally be built from local material, and largely by local and unskilled labor. The building and completion of such bridges is not dependent on mills, shops, or the operation of trusts, as is frequently the case with steel structures. In this respect, concrete bridges have an advantage over those of combined steel and concrete, for in the latter case, it is frequently necessary to await the convenience of the shops for the reinforcing steel. A consideration that should appeal to the purchasers of bridges is, that local labor and materials for concrete structures can usually be secured and used, and the money expended by a municipality goes back to its own people, instead of going to distant points in payment for manufactured steel.

Arches in general, which form is usually adopted for masonry bridges, present a more substantial and pleasing appearance than can be secured by any form of truss, even though an arched truss be considered, for in a truss, the outline of the arch is not so evident as in a solid structure. For railroad bridges the arch of solid concrete is superior to the reinforced, in that its greater weight and mass more readily absorb the vibrations and shocks due to the passage of heavy trainloads and engines. Concrete bridges require no floor renewals as steel bridges frequently do, and they will generally cost from 10 to 30 per cent less than stone. They are fire proof and have no steel, either in the form of principals or reinforcement, to rust. They can be widened at any time without tearing down the original bridges, as must be done with bridges of wood and steel.

Bridges of solid concrete are particularly suitable for permanent railroad structures. Many railroad companies are realizing their superior advantages and are replacing their steel bridges with new ones of masonry, and while these concrete bridges are frequently reinforced with steel, the main arches are in most cases, designed to resist only compressive stresses, with no need for steel in tension except to better unite the arch and to prevent cracking from change of temperature. Many iron and steel railroad bridges in America have been replaced two or three times by heavier steel ones during the past thirty or forty years, in order to renew worn out structures or to provide for heavier loads. When it is remembered that several masonry bridges in Europe that were built 2,000 years ago, are still standing and in use, it is evident economy for permanent roadways, to rebuild ordinary spans in masonry. Views of two old Roman bridges are shown on subsequent pages. Ponte Rotto at Rome, shown on page 73, was first completed in the year 142 B. C., and while it has been damaged several times by floods, owing to its unfortunate location, three arch spans still remain in good condition. The Bridge of Augustus at Rimini, supposed to have been built about 14 A. D., during the reign of Emperor Augustus, has five arch spans. The piers are very heavy and support semicircular arches. The bridge is finely ornamented, is still in good condition and in use at the present time. A view is shown on page 75.

Uncertainty of Masonry Arches.

As compared with steel frames, the design of masonry arches is uncertain. The hypotheses upon which the design is based are only approximate assumptions, and when constructed, the action of the arch under loads is unreliable. In the former case, with single truss systems and truss lines meeting in points, with working unit values closely known by long series of experiments in both tension and compression, the designing of such frames has become almost an exact science. It is different with masonry arches, as their conditions under loads are too little known to arrive at any exact method for proportioning them. Moreover, even if these conditions were more definitely known, the same incentive for reducing the quantities of material does not exist in masonry as in steel structures, because of the comparative cheapness of masonry. Some of the indefinite factors in the design of masonry arches are as follows:—

(1) The condition and amount of the external forces are not definitely known. For instance, in an arch with spandrel earth filling, the amount of the conjugate horizontal pressure of the earth against the extrados of the arch is comparatively unknown.

If the filling were a liquid, the external pressure would then be normal to the extrados and its amount would be definite. This condition does not ordinarily exist, and the nearest approach to liquid pressure is from spandrel filling of clean dry sand. It is well known that earth filling, which, when newly placed, will stand at no greater slope than one and one-half to one, will after it becomes set, support itself for a time, at any rate, with almost vertical faces. Hence, conjugate pressure which may have existed at first, while the arch was under construction, may vanish later. In the case of an arch under a deep embankment, it is plainly evident that such an arch does not support the entire weight of earth filling above it, as the earth to some extent arches itself. The case of a tunnel arch is an excellent example. Such an arch is proportioned to carry only a small part of the load above it, depending upon the nature of the overlying material. Further, where the masonry is continuous over the piers, especially where a large amount of backing is used, the material tends to cantilever itself from the piers, and thereby relieve the arch of much of its load, or if the amount of backing and filling above it be large, these materials may to a great extent arch themselves from pier to pier, and thereby relieve the real masonry arch. The external spandrel walls may also act as arches and carry a considerable load. The above remarks apply to bridge arches. In the case of arches in buildings, the condition of the external loads or forces is even more indefinite. Take, for example, the case of an arch carrying a wall load above it. It is customary to consider that the arch carries the entire weight of such a wall. The fact, however, is that an unbroken wall supports itself almost entirely, by acting as a masonry beam or by arching itself, and the only portion supported by the arch is a triangular piece of masonry directly above it. This is true for a wall without openings. When openings occur the above consideration will be effected, depending upon the location of the openings. If they occur in such positions as to evidently interfere with, and destroy the beam or arch-action of the superimposed masonry, then the entire weight of masonry may come on the arch. There are many bridge arches now standing that would doubtless fail, were they subjected to the entire weight of the materials above them. After striking center, the arch itself has settled, and much of the imposed load is transferred to the piers by the cantilever or arch action of the backing and fill, or the arch action of the spandrel walls.

(2) Another unknown factor in the design of masonry arches is the strength of masonry. Experiments have been made principally on small samples tested in machines with pressures normal to surface, all of which conditions are quite different to those of actual arches under loads. The material is then concentrated in bulk, with pressures inclined to bearing surfaces and with loads more or less of a vibratory nature.

(3) It is usually assumed by engineers and analysts, that the joints of block structures such as masonry arches will resist no tensile stress. This is a precaution on the side of safety, but may be far from true. With a rich quality of concrete, we know that properly formed points will actually resist considerable tension, provided they remain intact.

(4) The position of the line of resistance in the arch is not definitely known. This is largely due to the continuity of the arch at the center, and the square bearings at the piers or springs. To obviate this difficulty, some European engineers have built masonry arches with hinges at the crown and springs, thus fixing the position of the line of resistance at these points, but in America such provisions are not generally used.

(5) Imperfect workmanship in the cutting of the stones and the fitting of the joints is another factor causing the actual line of resistance to move from its supposed position to a different one, where the joints come to a firm bearing.

(6) The removal of the arch center and the settling of the arch to its permanent position, also effects to some extent the theoretical considerations.

It appears therefore that any effort at ultra refinement in arch design is a waste of energy, for the actual conditions existing in a completed structure may not even approximate those assumed.

Form.

The form or general outline is the first consideration in the design of a masonry arch. Semicircular and semi-elliptical arches, commonly known as full centered arches, spring from horizontal beds, while segmental arches spring from inclined beds called skewbacks. The old Roman arches were nearly all semicircular. In bridges and viaducts where piers are used, full centered arches or those which spring from horizontal beds, are preferable to segmental arches springing from inclined beds, for the reason that full centered arches produce a less overturning moment on the pier, and their attachment to the piers with horizontal beds is simpler than with inclined springs. The thrust on piers, however, depends upon the rise of arch, which is not necessarily the distance from spring to center intrados. The effective rise is the vertical height from spring to crown, measured on the linear arch or line of pressure and any minor curve joining the arch soffit to the pier, is not effective and must not be considered as part of the rise. Segmental arches have a shorter curve than elliptical for the same span, or for the same length of soffit the segmental arch results in a wider span. For small spans such as commonly used for culverts, segmental arches contain from 25 to 40 per cent less masonry than semicircular arches, though common practice makes the segmental arch ring 10 to 25 per cent thicker than the semicircular. For fluid pressure the proper form of arch is the semicircle. The effect of earth fill or other loads at the haunches, tends to raise the line of pressure to the approximate form of an ellipse, while the effect of a uniform load, such as the weight of earth fill and pavement above the crown, together with a uniform live load, tends to depress the line of pressure to the approximate form of a parabola. The combined effect of these two loadings is to bring the line of pressure more nearly to the segment of a circle. The most economical form is a linear arch of the given span for the required loading, in which the thickness is proportional to the thrust. In such an arch every part of the cross-section would be stressed alike. One authority recommends that the form of intrados for arches with earth filled haunches be midway between a circular segment and ellipse. Any variation from regular curves that is sufficient to be apparent to the eye, is a violation of a principle of design and should not be permitted. The many three and five centered flat arches already in existence are sufficient to clearly prove the utter failure of such forms to produce artistic or satisfying effects. If multi-centered flat arches must be used, they should be drawn from as many centers as possible. Three and five centered arches are suitable when the form approaches a semicircle.

An economical form of arch with cantilever brackets at the ends has lately been built over the Vermillion River at Wakeman, Ohio. The bridge has cross walls with open spandrels, a clear span of 145 feet, and end cantilever brackets 37 feet long. The method necessitates the use of reinforcing metal at the floor level for the purpose of tying the brackets to the main span. A somewhat similar plan was adopted in the Topeka bridge, but in the latter case the concrete cantilevers were for retaining walls only. The cantilevers were tied together with rods to prevent spreading from the pressure of the earth filling. In the case of arches such as culverts under high embankments, the segmental arch with its horizontal thrust is economical. The arch thrust resists and counteracts the earth pressure on the sidewalls from without.

Hinged Arches.

A practice that has long been followed in Europe, is to provide stone or metal hinges at the crown and springs. The use of such hinges locates definitely the position of the line of pressure at these points, and thereby removes one of the common uncertainties of masonry bridges. Hinges are particularly desirable where the nature of the soil is yielding or uncertain. Any lateral movement of the abutments causes the arch to sink at the crown when the centers are removed, and such sinking produces cracks that are unsightly and possibly dangerous. When hinges are used, the joints are filled in solid with cement mortar, after the centers are removed and the arch ring has assumed its final position. For additional loads, the entire area of both hinges and mortar filling will then be available for resisting arch thrusts.

Position of Springs.

The arch springs should be located as near to the foundation as conditions will permit. This will reduce the overturning effect on the pier to a minimum, and produce a more stable construction. Some of the conditions governing the position of the springs are as follows: Over streams the spring must be sufficiently high to allow ample water way, and clearance for the passage of boats or drift; over roads or highways the springs must be sufficiently high to provide proper head room and clearance for the passage of pedestrians and vehicles, and over railroads, for the passage of cars. In the last case, there must be a clear head room of at least 21 feet at a distance of five feet from the face of piers. This allows clearance for the largest box cars and additional space for trainmen on the roof.

Abutment Piers.

For long bridges or viaducts with a series of arches, abutment piers, or those of sufficient thickness to resist the pressure of a single arch, should be placed at frequent intervals. Where the spring line is located so near the foundation, that piers need not be excessively thick, it may be desirable to have all piers of the abutment type. Then, during the course of construction, the spans may be built independently and false work removed when desired, without reference to the adjoining spans, or after the completion of the bridge if one span should be destroyed by flood or other cause, the other spans would still remain intact. If all piers in an arch viaduct are of the ordinary type, to support vertical loads only, and one span should be destroyed, then the remaining spans would also fall, one after the other in succession, by the overturning of successive piers.

Height of Bridge.

In most cases, the height of the bridge or level of the roadway will be previously determined. In some cases, however, the floor grade may be varied more or less by grading the approaches to suit other conditions. It may be that money spent in raising the approaches and the level of the bridge floor, will be saved many times in the cost of the masonry.

Rise and Span.

The span is the clear distance between vertical faces of piers or abutments, and the rise is the height of crown above springs, measured on the line of pressure, and not on the arch intrados. Curves joining flat arches to piers are not part of the effective rise.

The length of span and rise of arch will be among the first considerations. In many cases, the natural conditions will determine one or both of these dimensions. If the bridge is short, a single span may be sufficient. If it spans a street or rapid stream, where piers are impracticable, the conditions will require only one span. In long viaducts, the dividing of such a structure into spans of proper length is an important matter. The economic span length depends chiefly upon the total height of structure above foundations. Generally, high structures require longer spans, and lower structures, shorter spans. For steel bridges with vertical reactions, the economic length of span for various heights is well known or may easily be determined, but with arches there are other considerations. The usual practice is as follows:—Place the springing lines on the piers down to the lowest point possible consistent with the necessary clearance, and after allowing for the thickness of the arch ring and filling at the crown, draw in spans, the length of which are from two to five times the rise of the arch, preference being given to spans of twice the rise or to semicircular arches. Certain other conditions, however, may determine the length of span. For example, in a long viaduct over railroad yards, it may be desired to span a certain number of tracks with each arch, or to have as few piers as possible to interfere with additional tracks or switches. In that case, the length of span may be fixed arbitrarily regardless of the rise or height of bridge.

In fixing the lengths of a series of arch spans, the Romans made those spans nearest to the center of the river, longer than the shore spans. The plan is still in general use, and it has the merit of causing the span at a distance from the shore observer, to appear at least as long as the nearer ones. When a uniform span length is used, the effect of perspective is to cause those spans near to the river center which should be of greater importance, to appear shorter than they really are.

To balance the pier thrust from unequal spans, the shorter one may have a smaller rise with greater earth filling and consequently greater loads.

Several of the large railroad companies have recently adopted standard segmental culvert arches having a rise of one-fifth the span. In many other bridges this proportion is exceeded, especially where natural or other conditions govern. Generally speaking it will be found cheaper to make long spans with few piers, provided sufficient rise is available.

Crown Thickness.

In the preliminary design it is necessary to know approximately the required crown thickness or depth of keystone, and also the amount of earth filling over the crown, to determine the remaining distance from crown to spring or the available height for the rise of arch. The crown thickness may be found approximately by reference to tables of existing arches, or from some reliable empirical formula. Trautwine's formula for such thickness is as follows, a development of the formula for various spans and rises being given in the Engineer's Pocket Manual.

$$\text{Depth of key in feet} = \sqrt{\frac{\text{Radius} + \text{half span}}{4}} + .2 \text{ ft.}$$

The above is for the first class cut stone work, either circular or elliptical. For second class masonry, increase the results from the above formula by one-eighth. for brick. by one third; for large elliptical arches some engineers increase also the above values by one-third.

Rankine's rule for crown thickness is:—

For single spans $\sqrt{.12 \text{ Radius}}$

For several spans $\sqrt{.17 \text{ Radius}}$

It becomes necessary therefore to determine the radius at the crown. This can be done graphically. The crown radius for an ellipse can be found as described later and shown in Figure 2. It is common practice with small segmental arches to make the arch ring from 10 to 25 per cent thicker than semicircular ones.

The crown thickness may also be found approximately by first determining the approximate crown thrust. This is easily computed by finding the center bending moments for all loads, the same as for a beam, and then dividing by the rise. or the approximate crown thrust may be found from Navier's formula, $T = pr$, where T is the crown thrust, p the average pressure per square unit on the arch. and r the radius of arch at crown. It will be noted that the proper value for the crown thrust is that one which produces equilibrium about the point of rupture, and not about the springs.

The experience of the writer in using Trautwine's tables of sizes and quantities for masonry arches is that Trautwine's figures are about one-third larger than the best practice now in use by the large railroad systems for the design of concrete arches.

Thickness of Crown Filling.

An assumed depth for this filling is required as noted above, in order to determine the available height for the rise of the arch. For highway bridges, a depth of filling including the pavement, of from one to two feet will be sufficient, but for railroad structures a greater depth is necessary in order to form a cushion for the ties and absorb and distribute the shock from passing trains. For this purpose a depth of from two to four feet, or ordinarily of two feet below the ties will be sufficient. To secure this cushion effect, the filling in some recent concrete railroad bridges has been as great as five feet.

Spandrels.

Bridge spandrels are either filled solid with earth held in place by side retaining walls, or the floor over the spandrels is supported on a series of interior walls and arches, which may or may not appear on the exterior. The solid earth filling is generally used for small spans and flat arches. But for large arches and especially semicircular ones, the open construction will be cheaper. In certain cases of comparatively flat arches, even where it would be more expensive than solid filling, the open spandrel construction may be desirable for the purpose of reducing the load on the foundations. This was the case with an elliptical arch bridge recently built by the Illinois Central Railroad Company over Big Muddy River, containing three spans of 140 feet each, with 30 feet rise. It was found that the open spandrel construction reduced the loading on the piles by about six tons per pile. Which one of these methods to use in any particular case, can be determined by making comparative designs and estimating the costs. In many cases, however, the choice can be made by inspection.

By building open chambers crosswise of the bridge and having the openings appear on the spandrel faces, a design is produced that presents a lighter appearance and at the same time shows plainly the plan of construction. When a heavier and more massive appearance is desired, then the side walls may be used and all spandrel openings closed. In large arches approaching the semicircular form, if open spandrels are used and the interior spandrel walls run parallel with the axis of the bridge, these walls then act as backing and produce the necessary conjugate thrusts on the haunches below the points of rupture. The need of providing for necessary conjugate thrusts is important and must not be overlooked. Cross spandrel walls and open chambers or arcades may be used above the point of rupture, but below that point the construction must be solid. This type of construction is well illustrated by the Connecticut Avenue bridge at Washington, shown on page 88.

An improved method of designing spandrels is illustrated in the Piney Creek Parabolic Arch bridge in Washington. The floor slabs are carried on an interior system of beams and columns supported on the arch ring, and the spandrels are enclosed with thin curtain walls. A design similar to this for a segmental arch was prepared by Mr. Thacher for the Bellefield bridge in Schenley Park, Pittsburg. This system is a very economical one and has the advantages of leaving the interior construction open at all times for inspection, and of producing a less amount of load in the spandrels for the arch to carry. The curtain walls are also thinner than retaining walls for earth filling and cost proportionately less, and the pavement may be laid at once without waiting for the filling to settle. When the pavement is laid, there will never be any liability of the road settling, as often does occur when pavement is laid on earth filling, even though such filling be well rammed and permitted to settle a long time before laying the roadway.

The use of the open spandrel construction with either cross walls or columns avoids any uncertainty in reference to horizontal conjugate pressure from spandrel filling, and also prevents water collecting and soaking into the arch masonry. When it is desired to secure a greater diversity in design, the face walls may be omitted and the interior arcade or colonnade construction artistically treated for the purpose of producing a more pleasing architectural effect. In comparing the relative costs of colonnade and arcade construction for spandrels, enclosed column construction will generally be found the cheaper, for the beams and columns may be left rough, and the spandrel curtain wall only will need a finished surface. Cross arcade construction has the economy of small dead load, but all open spandrel walls are exposed to view and may require finished surfaces or possibly architectural treatment. Open chambers may be enclosed at the top, either by means of arching or by using flat slabs of stone or reinforced concrete. The upper surface is then waterproofed by applying a layer of rich mortar and surfacing with neat cement, on top of which is poured a layer of tar or pitch. The surface may then be leveled with gravel and sand, and the pavement laid.

Another reason for selecting either the solid or the open spandrel type is for the purpose of adjusting the imposed loads on the arch to the form selected. This may be necessary to secure stability and will be considered later under the head of loading. In designing the side spandrel walls to retain earth filling the usual rules for retaining walls will apply. Practice is to make the thickness of such walls at the base 40% of the height. They should be firmly doweled or otherwise secured to the arch masonry.

Various Forms and How to Draw Them.

The forms adopted for the intrados of masonry arch bridges are generally circular, segmental, elliptical, or multi-centered. These four types can be reduced to two, circular and elliptical, for the segmental arch is merely a segment of a circle, and the multi-centered arch is merely an approximate ellipse. The two general forms are, therefore, the circular and the elliptical. Methods of drawing the ellipse and the multi-centered curve are as follows:

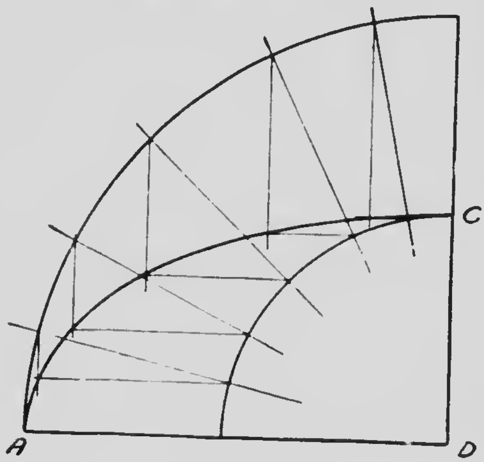


Fig. 1

Ellipse.

Let AD and CD be the semi-major and semi-minor axes of an ellipse at right angles to each other. Draw circular arcs with radii AD and CD, respectively. From points where a common radius intersects the two circular arcs, draw vertical and horizontal ordinates. The intersection of these ordinates gives points on the ellipse.

Multi-Centered Arch—Three Centers.

These curves are sometimes called basket-handled arches. The method of drawing a three-centered

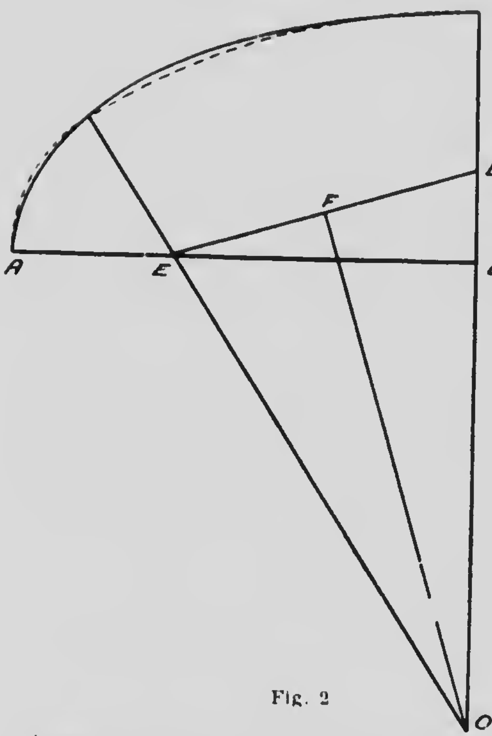


Fig. 2

arch is as follows:

Let AD and CD be the semi-major and semi-minor axes, respectively, of a true ellipse. The form of the true ellipse is first drawn by the method given above. This is shown in Figure 2 by the full line. The approximate form is then drawn as follows:

Assume any two equal distances CB and AE less than half of the semi-minor axis. Join BE and bisect the line BE at F. Through F draw a perpendicular to BE, intersecting the line CD at O. The two points O and E will be centers of two circular arcs which will form an approximate ellipse. By first selecting the position of the point E so the circular arc described from E as center will conform as closely as possible with the true ellipse, satisfactory curves will easily be found. The full line on Figure 2 shows the true ellipse and the dotted line the approximate.

Five-Centered Arch.

A method for drawing a five-centered arch is as follows:—

In order to check on the work, it is advisable to first draw the form of the true ellipse by the method given above. In Figure 3 the two curves so closely correspond that only one can be shown. On the transverse axis AO draw the rectangle AGCO, equal in height to the semi-minor axis OC of the ellipse, and draw the diagonal AC. From G draw a line GHD perpendicular to AC and intersecting the center line CO of the span produced at D. From O as center, with radius OC, draw the circular quadrant as shown. Describe the semicircle ARL and produce the line OC to its intersection with the semicircle at L. From O as center, describe the arc at M with radius equal to CL, and D as center de-

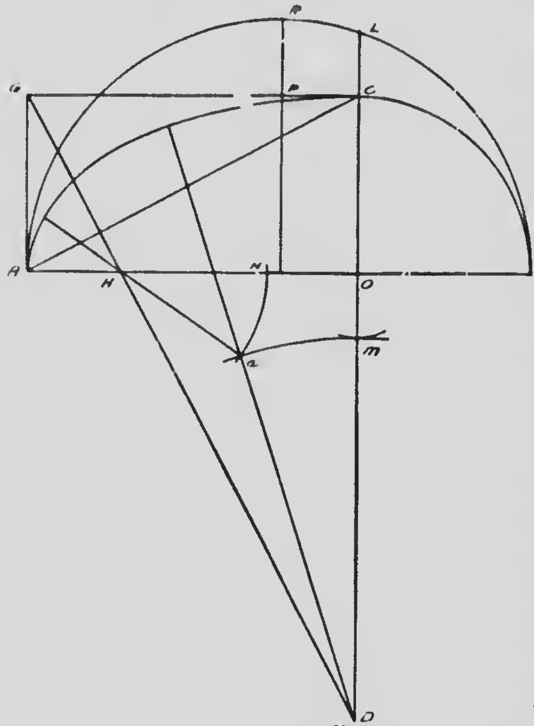


Fig. 3

scribe arc aM, with DM as radius. On the axis AO lay off AN equal to OL. Then from H as center, with radius HN, describe the arc Na, cutting Ma at a. The three points H, a and D, with corresponding ones in the other quadrant are the five desired centers from which to draw the approximate ellipse. This method of drawing a five-centered arch as approximate to an ellipse must not be confounded with the method given later for drawing a hydrostatic arch. The crown radius of the ellipse will be less than the corresponding radius of the hydrostatic arch.

Parabolic Arch.

The parabola is not frequently used in masonry bridges, but the formula for drawing it is given. It is as follows:—

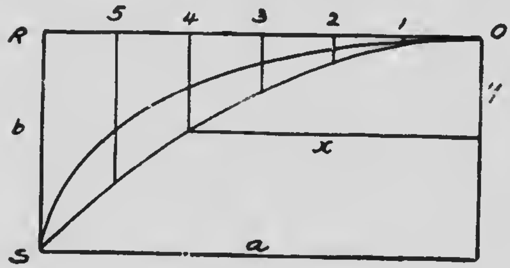


Fig. 4

The various letters refer to dimensions shown in the accompanying Figure 4.

$$y = \frac{x^2 b}{a^2}$$

The line OR is divided into any number of convenient equal parts, which are numbered 1, 2, 3, etc., beginning at the point nearest O. Then to find the value of y, for the various ordinates x, the numbers 1, 2, 3, etc., may be inserted in the above equation for values of x, and the total number, which in the illustration is 6, will be inserted for the value of *a*. The upper line in Figure 4 shows the corresponding form for a true ellipse.

A very simple graphical method of drawing the parabola is to lay off on the vertical line RS the same number of equal divisions as drawn on the horizontal axis OR, and from O draw radiating lines to the various division points on the vertical axis RS. From the various points on the horizontal line OR draw vertical lines intersecting the radiating lines from O. The points at which these vertical lines intersect the radiating lines are points on the required parabolic curve.

Hydrostatic and Geostatic Arches.

In selecting the most suitable form for the intrados of an arch, the following consideration of the above two forms of curves will be serviceable. The hydrostatic arch is the form of a linear arch under varying pressures which are always normal to the line of arch. This condition corresponds to that of an arch submerged below the surface of water. As the depth below the surface increases these normal pressures increase proportionately, and as the external pressures are always normal to the surface, the amount of pressure in the arch is constant, and is equal to the produce of the external pressure at the point by the radius of curvature. The equation is $T = pr$, and is known as Navier's Principle. Since the essential principle of the hydrostatic arch is that fluid pressure is normal to the surface, the thrusts at all points of the arch ring are, therefore, constant, and cannot vary without the application of oblique or tangential pressures. Since T is constant, r will vary directly as p. These radii may be found for varying depths below water level, and the corresponding curve plotted. It will be noted that the thrust T at the crown, is equal to the total horizontal pressure on the extrados of half the arch.

Ordinarily, however, arches are subjected to earth pressure rather than water. The external forces are, therefore, no longer normal to the extrados of the arch, but bear a relation thereto, depending on the nature of the overlying material. In the case of earth or gravel filling, having an angle of repose of one and one-half to one, it is known that the horizontal pressure exerted against vertical surfaces is about one-third of the weight of the material above the point under consideration. The formula is $H=\frac{wp}{3}$. The linear arch supporting a filling of clean dry sand would be the true form of the geostatic arch. If p is the horizontal intensity of force in the hydrostatic arch, and p' the corresponding force in the geostatic arch, then $p=Cp'$. It will be seen, therefore, that the geostatic arch bears the same relation to the hydrostatic arch as the ellipse does to the circle. A linear geostatic arch may, therefore, be drawn for any assumed value of C, such as 3, which experiments show to be about the right factor for earth or gravel filling. In drawing this linear arch all the vertical co-ordinates of the hydrostatic arch are retained, and conjugate pressures changed according to the formula $p = Cp'$. For arches under heavy banks of earth the geostatic arch can be drawn from the hydrostatic arch. If the height is fixed, the form of curve and proper width can be found to properly withstand the earth pressure. For bridges, these principles are useful chiefly for arches under high embankments.

In his book on Civil Engineering, page 420, Rankine gives the following approximate method for drawing the form of a hydrostatic curve about five centers by means of circular arcs. The two radii r' and r'' are first computed from the accompanying formula. This fixes two of the centers and the third is found at E as shown. The equations for radii are as follows:—

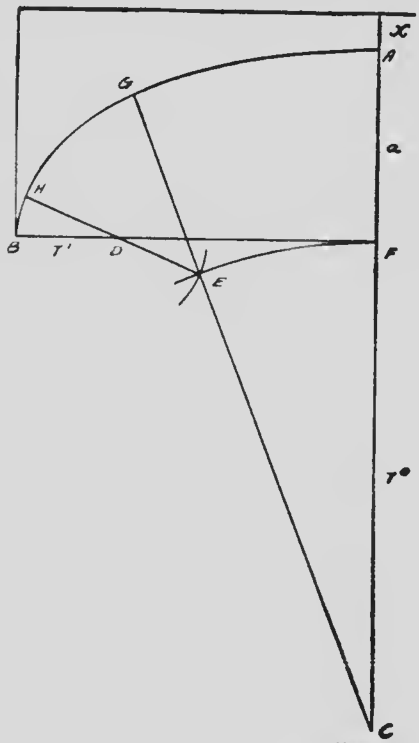


Fig. 5

$$r'' = \frac{a}{2}\left(1 + \frac{b^3}{a^3}\right)$$

$$r' = \frac{a}{2}\left(1 + \frac{a^3}{b^3}\right)$$

$$b = y + \frac{y^2}{30\,a}$$

$$BF = y$$

$$DE = AF - BD$$

$$x = a\left(\frac{a^3}{b^3 - a^3}\right)$$

In Figure 5, let FB be the half span and FA the rise of the proposed arch. Make $AC = r^\circ$, and $BD = r'$, the radius of curvature at the crown and springing as calculated from the above formulae. Then C will be one of the centers and D another. About D, with the radius DE, describe a circular arc, and about C, with radius CF, describe another circular arc. Let E be the point of intersection of these arcs. The points D, E and C will be the required centers.

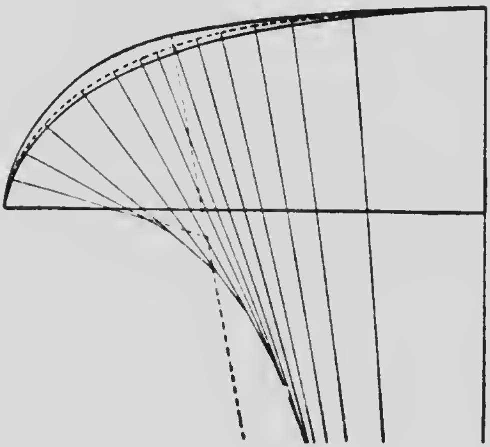


Fig. 6

Many semi-elliptic arches approach very nearly the form of a hydrostatic arch. A comparison between Rankine's approximate curve and the true one are shown in Figure 6. The upper or outside curve is the approximate curve as given by Rankine. The center curve is the true hydrostatic arch plotted from a succession of radii, and the inside curve is a true ellipse.

Selection of the Most Suitable Form.

Full centered arches, either circular or elliptical, produce the least overturning moment on the piers, and will generally require less pier masonry than segmental arches. If the arch thrusts against natural rock skewbacks or abutments, the amount of such thrust is then a matter of little importance as far as the abutment is concerned. The attachment of segmental arches to piers usually requires tilted beds to bring the joints at right angles to the line of pressure. This is a condition that does not occur in full centered arches. In flat ellipses the pier thrust is greater than with semicircular arches, the position of thrust approaching more nearly that of a segmental arch. It has already been shown that, for arch culverts carrying heavy earth banks, the segmental form of arch will be more effective and less expensive. It produces heavy thrusts on the abutments, which thrusts counteract the inward pressure of the earth on the side retaining walls. At the same time there is a shorter length of curved work to build than with a semi-circular form. The cost of segmental culverts has been shown to be only about 60% of the cost of the corresponding semicircular ones.

After drawing a trial linear arch or line of resistance for any particular case, the form of this trial curve will suggest the most suitable form for the intrados of the structure. For a bridge with spandrel filling and loads increasing from the center to the springs, the elliptical form or a corresponding multi-centered arch will probably lie nearest to the linear arch, while for an arch with open spandrels the condition of loading will be more nearly uniform, and the curve will be flatter at the haunches and approach the form of parabola. In such cases the segmental form would probably be used instead of the elliptical. The elliptical form requires less filling in the haunches than the segmental arch, and has, therefore, less weight to carry. At the same time it gives a greater amount of clearance underneath. A semicircular or Roman arch with a large rise generally requires the smallest piers, and in a high viaduct, where the piers are an important part of the total cost, this form will be economical. The exact line of resistance for an arch under a high embankment is the geostatic arch. It may, however, be assumed as an approximate ellipse. The form of the intrados under earth whose angle of repose is 30 degrees will then be determined by the equation:—

$$\frac{\text{Vertical axis}}{\text{Horizontal axis}} = \sqrt{3}$$

In designing culvert arches it will be advisable for the engineer to consult standard plans for such structures. Many considerations will appear that might not at first occur to the designer.

External Loads and Forces.

It has already been shown that both the amount and direction of the external forces acting on a masonry arch are indefinite. In an arch supporting a masonry wall it is usually assumed that the arch carries the entire weight of wall above it. This is on the side of safety, but is certainly not correct. The wall will, to a great extent, support itself, either acting as a beam or arch, and the probability is that the weight of only a small portion of the wall directly above the arch is all that is carried directly by it. Arches under high embankments certainly do not support the entire weight of earth above them. The earth corbels or arches itself, as is plainly seen in the case of a tunnel, where only a small portion above the crown is supported by the tunnel center. It is customary to consider that arch bridges with spandrel filling support the entire weight of such filling on the arch ring. The fact is, however, that the backing and fill either arch themselves, to some extent, from pier to pier, or if the backing is continuous over the pier, the backing itself will then form a cantilever and carry much of the spandrel loads.

The English engineer, Brunel, many years ago designed and built a semi-arch of brick, with hoop iron bond, 60 feet in length, which supported itself entirely by cantilever action. Since the introduction of reinforced concrete as a desirable material for arch construction, it has become common practice to build cantilever arms or brackets on the shore ends of arch spans, showing that the cantilever principle is just as sure to come into action when continuity over the piers exists, as it is that the arch thrust itself is in operation. A good illustration of this cantilever construction is shown in a bridge recently built over the Vermillion River at Wakeman, Ohio, and described in Engineering-Contracting, February 4, 1909. Somewhat similar cantilever arms were used for retaining walls at the ends of the reinforced concrete arch bridge at Topeka. Kansas.

Not only is the amount of vertical loading from the filling unknown. but the horizontal conjugate pressure on the masonry haunches is also indefinite. We know that nearly all semicircular arches, or those of similar form. after the centers are removed. will settle at the crown and recede laterally at the haunches. The effect of this settlement is to bring conjugate pressure on the backings. and. therefore. it is certain that pressure exists there. but the amount of such pressure is unknown. Semicircular arches require backing below the point of rupture to produce conjugate pressure equal in amount to the crown thrust. This must be secured. either from backing. fill or spandrel walls. If the point of rupture in segmental arches is at or near the skewback. the conjugate thrust then comes from the abutment. and little or no backing or corresponding walls will be required. While conjugate pressures are necessary for stability below the point of rupture, it has been demonstrated that conjugate tensions are necessary above that point, and to secure that result. rods have been used. The intensity of conjugate thrust from earth filling with an angle of repose of 30 degrees is one-third of the vertical. It is good practice to cut the voussoir stones on the extrados of the arch into steps with horizontal and vertical faces. so the pressures on these may be normal to the surfaces.

Scheffler's Theorem assumes that all external loading acts vertically. This is an error on the safe side and will require abutments slightly heavier than when conjugate horizontal forces are considered.

It has already been stated that elliptical arches have less fill or material above them, and consequently less weight to carry, than either segmental or parabolic arches.

In the case of arches supporting earth filling, the form of such filling will, to a large extent, determine the proportion of weight that bears upon the arch. A long bridge will carry the entire weight of material above it, while a culvert under a high bank will carry only a portion of the material above it. Sewer arches exist which would be unstable without earth pressure, showing clearly that conjugate earth pressure does exist.

Mathematical Theory of the Arch.

The theory of arches is very complex and intricate. Analysts have given much thought to the matter and many volumes have been written, when in reality, the complete determination of the force polygon, and the corresponding line of resistance in the arch, constitute all the calculations involved in the practical design of a masonry arch. All methods of computation are approximate only. The thickness of arch is first assumed by comparison with tables of existing arches or by the use of some empirical formula. Lines of resistance are then drawn for this arch, and if these lines do not fall within the middle third of the arch ring, the form is changed and a new line of resistance is drawn for the revised form. The calculations resolve themselves into a series of trials. No effort will be made here even to review the many theories of the arch. For such investigation the student is referred to the writings of mathematicians. Their conclusions only will be given in this book. The theory is based upon the assumption that joints will resist no tension.

Stability Requirements.

The requirements for complete stability in a masonry arch are three in number:

(1) There shall be no rotation of one part of the arch about another.

(2) There shall be no sliding of one surface upon another.

(3) The unit pressure shall be such that no crushing of the arch material shall occur.

To insure the first requirement it is necessary that the line of resistance shall lie entirely within the arch ring, and to insure further that the pressure shall be distributed across the entire section of the arch, and no tendency to opening of the joints occur, it is necessary that the line of resistance shall lie within the middle third of the arch ring. To avoid sliding of one joint upon another, all joints, including those in the arch and in the abutment, shall make angles not less than 70 degrees with the line of resistance. The friction coefficient for masonry joints is from 40% to 50%. To avoid crushing of the arch material, the cross-section of the arch shall be sufficient, so that the intensity of pressure at the outer edge shall not exceed a certain safe working unit. With these three requirements fulfilled, the stability of the arch is assured. If a line of resistance cannot be drawn within the middle third of the arch ring, then it is necessary to change either:—

(1) The thickness of the arch ring,

(2) The form of the arch, or

(3) The distribution of the loading.

Practice in the design and construction of concrete arches varies in reference to the absence or presence of joints in the arch ring. In large structures, where the entire concrete cannot be placed from one mixing, it is customary and sometimes necessary to provide joints in the arch ring, and as an additional precaution against sliding of such joints, they may be doweled or dovetailed together.

Ultimate Values.

The ultimate crushing values of the common arch materials are as follows:

Granite ...	5,000 to	18,000	pounds	per	square	inch
Limestone .	4,000 "	16,000	"	"	"	"
Sandstone .	3,000 "	10,000	"	"	"	"
Concrete ..	2,000 "	4,000	"	"	"	"
Brick	300 "	600	"	"	"	"

Working Units.

The working unit strength of these materials at the outer edge is taken at one-tenth of the ultimate, and as the maximum pressure at the outer edge when pressure at the inner edge is zero, is twice the mean or average pressure, this corresponds to using a mean unit pressure of only one-twentieth of the ultimate. The necessity for this high factor will be seen from the following considerations. Experimental data on the strength of masonry in bulk is comparatively small. Most experiments have been made on sample pieces of the material held properly in position with pressures applied normal to surfaces. Also the crushing strength of masonry in bulk is much less than that of the separate material of which it is composed, because of the presence of mortar joints. On the other hand, experiments were made on sample cubes of material, while in the arch the material is used in large mass, and is, therefore, stronger than cubes. Errors in workmanship and in fitting of joints may cause excessive pressure to occur on some parts of joints, and little or none at all on other parts. The entire system of external loads is, therefore, uncertain. Working units may safely be taken as follows:

Granite	500 to	1,500	pounds	per	square	inch
Limestone ...	300 "	1,000	"	"	"	"
Sandstone	200 "	800	"	"	"	"
Concrete	200 "	500	"	"	"	"
Brick	80 "	100	"	"	"	"

A maximum pressure of 400 pounds per square inch is good practice for concrete arch rings, and is suitable for a mixture of 1-2-4 well and carefully laid.

The above pressures refer to the maximum pressure at the outer edge and not to the mean or average pressure, which would be only one-half of the above. These units will give a factor of safety of ten in compression. The requirement that the line of resistance shall fall within the middle third of the joint produces a factor of safety against rotation of three, and the requirement that the angle between the face of joints and the line of resistance be not less than 70 degrees produces a factor of safety against sliding of from one and one-half to two.

Determination of Line of Resistance.

Ordinarily, the consideration of two cases of loading will be sufficient. (1) A uniform dead and live load over the entire structure, and (2) the entire dead load with a maximum live load over one-half of the span only. The absolute maximum stresses from partial loading may be obtained when the live load is applied to somewhat less than one-half the span, as .4 to .45 of the length, but for practical purposes it is sufficiently accurate to consider half the span loaded. In certain cases it may be necessary to consider the maximum dead load with a single concentrated live load at the center.

Find first the line of resistance for the maximum dead and live loads over the entire structure. An approximate thickness will have been assumed for the arch ring at the center, also the depth of the earth filling above as previously described, and an approximate form of arch will have been selected. If the bridge has spandrel filling, the first operation will be to divide the loaded area above the intrados into a number of vertical strips, to compute the weight of material in each of these strips and the live load on them. In order to simplify calculations, a portion of the bridge one foot in length at right angles to the paper will be considered. Each remaining portion will be a duplicate of this. It may be necessary to draw a separate line of resistance under the side spandrel walls, because the weight of wall masonry is greater than earth fill. The amount of conjugate pressure of the backing on the haunches is then considered. For gravel and earth the intensity of this pressure per square foot or other unit may be taken at one-third of the weight of filling and live load above the extrados at the strip under consideration. Then the product of this horizontal intensity and the area of the vertical projection of that portion of the extrados under the strip will give the amount of the conjugate thrust. This will be repeated for all other strips and a complete set of loadings found, which should all be written in their respective places.

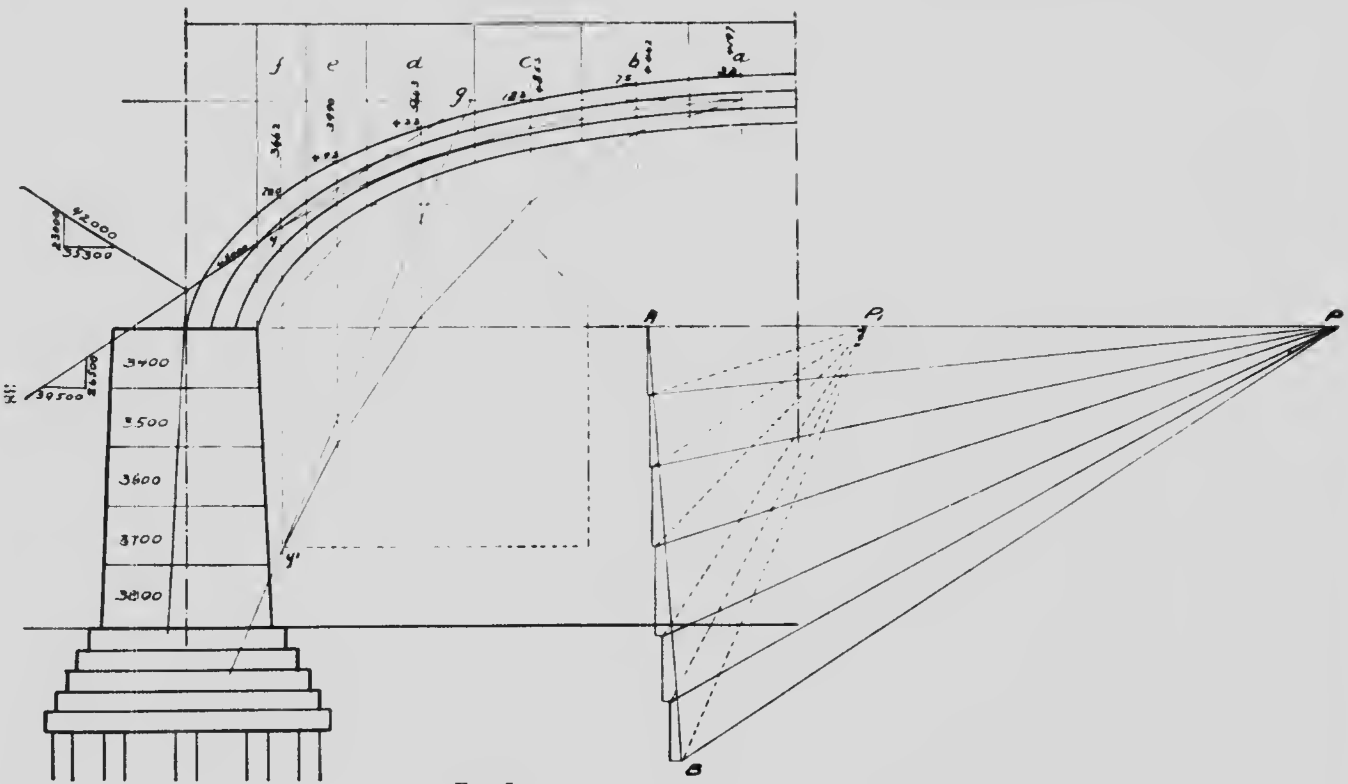

f
e
d
c
b
a
g
A
B
P
P1
y'
42000
35300
23000
39500
26500
3400
3500
3600
3700
3800

Fig. 7

Proceed next to construct a force polygon by drawing the various loadings to a convenient scale. As arches are generally symmetrical about the center and horizontal at that point, the crown thrust for uniform loadings will likewise be horizontal. The pole in the force polygon will, therefore, be on the same horizontal line with the upper end of the first load line at A. The amount of this crown thrust is unknown, and the pole distance can, therefore, be only assumed for the present. Take any pole, as that shown at P′ on Figure 7, and draw the corresponding force polygon. Draw also the corresponding line of resistance or funicular polygon in the arch ring, starting from any point within the middle third at the crown. The resulting funicular polygon is that shown at *ay′*. It is evident that the pole distance assumed was not the correct amount of the crown thrust, for the line of resistance or polygon falls entirely outside of the arch ring. Project the last line of the funicular polygon till it intersects the line of crown pressure produced at the point *g*. This gives the position of the resultant of the assumed loads, and its direction will be parallel to the line AB in the force polygon. The position of this resultant is constant, regardless of the force polygon. Therefore, the corresponding line of any other funicular polygon produced, such as that through *y*, will likewise intersect at the same point. Therefore, through *y* draw such a line, and from B in the force polygon draw BP, intersecting the horizontal through A at P. The distance AP measured to the same scale as the load line will represent the true amount of the crown thrust. The other lines radiating from P to the various points on the load line will truly represent the amount of thrust at the various points in the arch.

A check on the crown thrust may be made by finding the bending moment at the center for all the loads in the same way as for a beam, and dividing this moment by the rise of the arch. It will be remembered, however, that the rise is not necessarily the distance from spring to crown, for in flat arches, and especially in elliptical forms, the line of resistance does not fall as low as the springs. The correct rise of an arch is the rise of the line of resistance and not the rise of intrados from spring to crown.

It will be seen by inspection that a position of the point *y* was selected so the line of pressure would not pass outside of the middle third of the arch. It approaches nearest to the limit under the strip *d*. The point opposite to this limiting position is called the point of rupture, and is the point at which the arch first tends to open at the extrados. If the line of resistance from the assumed point *y* had fallen outside the middle third of the arch ring at *d*, a new point would then have been assumed so as to bring the line of resistance entirely within the middle third at the point of rupture. As this point y would approach very close to the middle third for an arch of uniform thickness from crown to spring, the ring is thickened at the haunch to keep the line of resistance well within the middle third. The line *ay*, which falls entirely within this limiting space, is, therefore, a true line of resistance for the maximum assumed dead and live loads. It was necessary to determine the crown thrust or pole distance by trial, because there are four unknown quantities, the two vertical and the two horizontal reactions of the arch, and to determine these there are only the three equations of equilibrium, $\Sigma x=0$, $\Sigma y=0$, $\Sigma m=0$. The line BP applied at the point y, represents truly in both direction and amount, the thrust of the arch on the abutment. This may be resolved into vertical and horizontal components as shown.

Numerous ingenious methods have been adopted for simplifying the computations. For instance, some writers prefer to construct what they call a reduced load contour. This consists in first finding the actual loads of arch ring, fill, live loads, etc., for each vertical strip, and reducing the height above the extrados to a corresponding height, provided the load was caused entirely from stone or material of the same nature as the arch ring. Plotting these various heights to scale above the intrados, and connecting the points so found, produces a line which is called the reduced load contour. Then by making the divisions two feet in width, and scaling the length of the two sides of each strip, the sum of the lengths scaled will represent the area of the enclosed strip. Sometimes the areas are plotted on the load line of the force polygon instead of the weights.

Practice varies somewhat in reference to the selecting of the proper point in the middle third of the arch crown from which to draw the line of resistance. When a hinge occurs at the crown there is then no uncertainty as to the correct position of the line of thrust. Some designers consider that the position of the line of resistance is such as to make the crown thrust a minimum without causing tension on any part of the section. To satisfy these conditions, the line would pass through the upper extremity of the middle third at the crown, and at the springs or at the points of rupture, the line of resistance would pass through the inner extremity of the middle third. Professor Church says that the true line of resistance is that one corresponding most nearly with the center line of the arch.

The intensity of the unit pressure on a surface may be found from the following formula:—

$$P = \frac{W}{L} + \frac{6Wd}{L^2}$$

where p is the maximum unit pressure at any part of a joint, W the total pressure, d the distance of the center of pressure from the center of the arch ring, and L the depth of the arch ring. The formula is general for all positions of *d*, provided the joints can resist tension. If they cannot resist tension, the formula is still general for the values of *d* up to one-sixth of L. If *d* exceeds this amount the maximum pressure is then given by the formula:—

$$p = \frac{2\ W}{3\ (\text{one half}\ L - d)}$$

The amount of crown thrust or pole distance may be found analytically by taking moments successively around the various load points in the arch. The crown thrust will be found a maximum when moments are taken about the load point opposite to the point of rupture. This is an analytical method of locating the point of rupture.

If the arch had hinges at the crown and springs, as are commonly built in Europe, the crown thrust could then be definitely figured. The presence of such hinges greatly facilitates the computations for partial loading, for then, not only the amount of the crown thrust, but also its direction, are unknown. It is no longer a horizontal thrust.

The above method of drawing a line of resistance for uniform loads applied to a pair of segmental arches is illustrated also on the left hand arch of Figure 10.

A modification of the above method of determining the crown thrust and drawing the line of resistance is shown in Figure 8. The space above the arch ring is divided as before into ten equal divisions and the total load on each calculated and indicated in the proper places. Beginning at the point R, which is the upper extremity of the middle third at the crown, the loads for half the arch are meas-

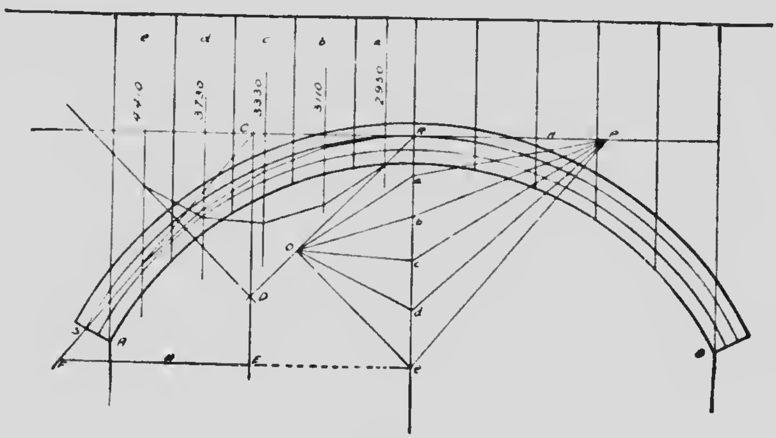


Fig. 8

ured off to scale on a vertical load line Re. From R and e draw lines at 45 degrees with the vertical intersecting at O, and from O draw lines to the points a, b, c and d. Construct a polygon with sides parallel to the lines Oa, Ob, Oc, Od and Oe and extend the two extreme lines of this polygon to their intersection at D. Through D draw the vertical CE, intersecting the horizontal line R at C. The line CE marks the center of gravity of the loads on the five arch divisions. Through C draw the line CS so that the line of resistance, when drawn, will lie within the middle third of the arch ring. After drawing the line of resistance, if it should be found that any part of it falls without the middle third, a new position must then be assumed for the point S. Through *e* draw the horizontal line EF, intersecting CS prolonged at F. The line FC will represent truly to scale the amount of the crown thrust. From R lay off on a horizontal line through R, the distance RP, equal to FE, and join P with the points *a*, *b*, *c*, *d* and *e*. From R draw the line of resistance with sides parallel to the lines P*a*, P*b*, etc. If any part of this line of resistance falls outside of the middle third of the arch ring, a new position must then be assumed for the point S, and another line of resistance drawn, falling entirely within the middle third. If no such line of resistance can be drawn, then either the form of the arch or its thickness must be changed until a line of resistance can be drawn lying entirely within the middle third.

Line of Resistance—Partial Loading.

Consider next the case of a maximum live load over half the span, acting in conjunction with

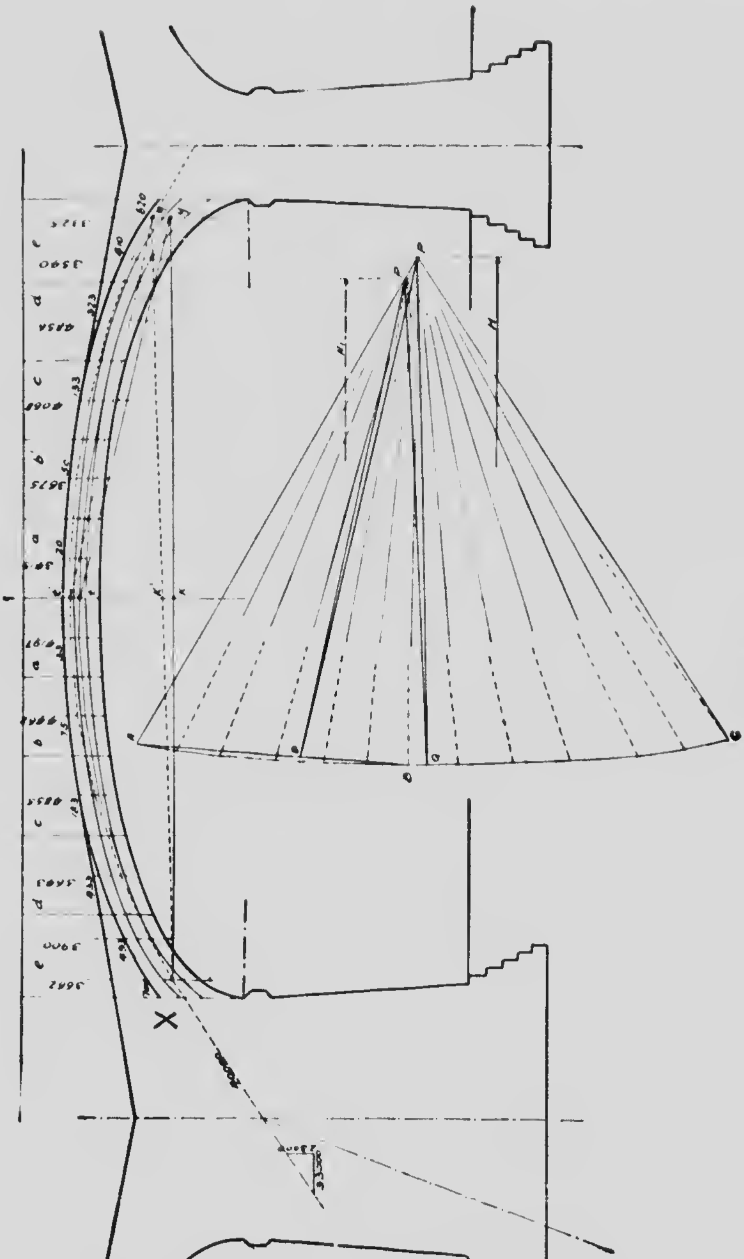


Fig. 9

the maximum dead load. Both halves of the arch must then be considered. As before, the portion of the bridge above the intrados is divided into vertical strips, and the vertical and conjugate loadings written down in their respective places. A load line, ABC, is drawn, and any trial pole, P′, assumed. With this position of pole, the funicular polygon shown in dotted lines is drawn. By using a little care, the point *x* may be selected, so the curve on the left will fall within the middle third, or tangent to it. It will be seen that this line of resistance shown dotted, falls outside of the middle third in two places and intersects the outer vertical through *c′* at *y*. This curve cuts the center line of arch at *t′*. See if it is possible to draw another line of resistance, so that it will cut the center of the span at the point *t* and pass through the point *y*. From P′ draw a line parallel to *t′ y′* intersecting AB at D, and from D draw another line DP parallel to *ty*. The new pole will lie on the line DP. Also through P′ draw a line parallel to *xy′* intersecting the load line in Q, and from Q draw another line QP parallel to *xy*, intersecting the line DP at P. The point will be the correct position of the pole, in order to have the line of resistance pass through the three points, *x*, *t* and *y*. The distance H in the force polygon may be verified analytically as follows:—

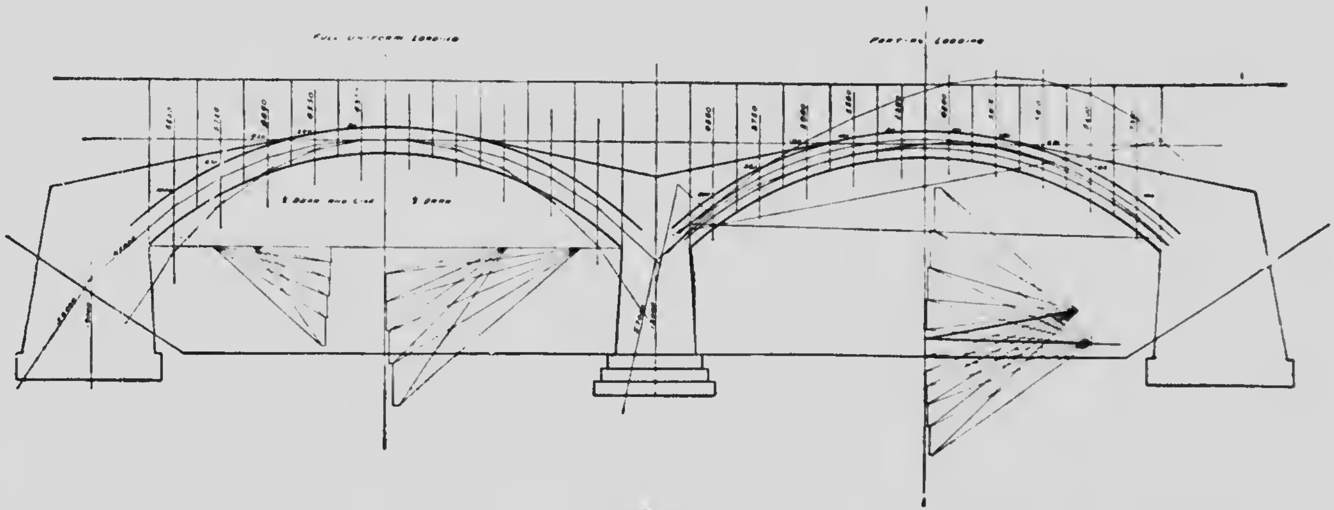


Fig. 10

The accompanying stress sheet shows a design for a two-span concrete arch bridge made by the author of this book, and built in the year 1907, to carry a double track railroad over an irrigation canal in Idaho. The canal is 8 feet deep, and 70 feet wide at the bottom with side earth slopes. The spans are segmental, and have a clear width of 35 feet with 7 feet rise. The depth of earth filling from the base of rail to the arch extrados at crown is 3.5 feet, while the depth from base of rails to the bottom of canal is 20.5 feet. The bridge contains 1150 cubic yards of concrete, and the cost including centers and temporary shoo-fly trestle to carry travel during construction was $11,000.

$$H' \times t'k' = H \times tk.$$

From this equation the value of H may be found, and the point P will lie on the line QP at a distance H from the load line. The line of resistance *xty* is tangent to the line of middle third in the strip *d*. The point where lines become tangent might have been taken as the required point through which, with *x* and *t*, it was desired to pass a line of resistance. The corresponding line would have been found in a manner similar to that described. It will be seen that the line *xty* lies entirely within the middle third of the arch, and the arch as drawn is, therefore, stable. If it had been found impossible to draw a line of resistance within the limits of the middle third, it would have been necessary to change either (1) the form of the arch; (2) the thickness of the arch; or (3) the distribution of the arch loading. A similar method applied to segmental arches is shown in Figure 10. In this case the bridge was designed to carry a double line of railroad, with tracks 15 feet apart on centers. It was assumed that the ties and earth filling distribute the weight of each track and the live load thereon evenly over one-half the width of the bridge. This assumption may not be true, but it is as reasonable an approximation as can be made. The live load was assumed equal to Cooper's standard E 50, and for 35-foot spans is equivalent to a uniform live load of 10,000 pounds per lineal foot, which was considered evenly distributed over a width of 15 feet, amounting to 667 pounds per lineal foot in width of bridge. For partial loading, the equivalent uniform live load on half the span was assumed at 11,500 pounds per foot of track.

Point of Rupture.

The point of rupture is that point of the arch ring at the haunches where the joints tend to open at the extrados, or where the line of resistance lies closest to the inner edge of the arch. By some writers this point is considered the real springing point of the arch, and any part of the arch below the point of rupture is considered as part of the pier or abutment. Its position can best be determined graphically when drawing the resistance line, and, as far as the arch itself is concerned, the line of resistance is required only above the point of rupture. It is, however, continued further for determining the stability of the pier.

The following empirical rule gives approximately the required thickness for circular segmental arch rings at the point of rupture. In the following equation t = crown thickness, d = required thickness at point of rupture, when

$$\frac{\text{rise}}{\text{span}} > = \tfrac{1}{4} \text{ then } d = 2.00\,t$$

$$\text{"} \quad = \tfrac{1}{6} \text{ then } d = 1.40\,t$$

$$\text{"} \quad = \tfrac{1}{8} \text{ then } d = 1.24\,t$$

$$\text{"} \quad = \tfrac{1}{10} \text{ then } d = 1.15\,t$$

$$\text{"} \quad = \tfrac{1}{12} \text{ then } d = 1.10\,t$$

In reference to the necessary thickness of the arch ring at various points between the crown and springs the vertical projection of every section cutting the arch ring normal to the line of resistance must be at least as great as the vertical depth of arch ring at the crown.

The position of the point of rupture generally occurs at about that point of the arch where the normal to the line of pressure makes an angle of 45 degrees with the horizontal. It may be said that it never falls lower than an angle of 30 degrees with the horizontal and generally between 35 and 45 degrees with the horizontal.

Determination of Arch Thickness.

The amounts of pressure at the various points of the arch have been determined. It will be seen that these pressures increase from crown to spring in proportion to the rise of the arch. In some extreme arches the thrust at the spring may be three or four times the thrust at the crown. The relative position of the center of arch and the line of resistance must be examined and suitable unit pressures selected for the various points. If the line of resistance is at either limit of the middle third, the mean unit pressure will then be one-half of the maximum at the outer edge. This is the usual assumption. Then the area obtained by dividing the total pressures by the working units will be the required area of material at various points of the arch. Most authorities on the subject recommend liberal sizes, not only because the usual arch material is not expensive, but also on account of the uncertainty of so many conditions in connection with the whole matter.

Backing.

Reference has already been made to the point of rupture. It is that point on the extrados of the arch where the joints tend to open, and it occurs opposite that point where the line of pressure approaches nearest to the intrados. It is known in the failure of flat arches that the joints open at the intrados of the crown, and extrados at the two points of rupture, and the haunches recede laterally, allowing the central part of the arch to fall. In order to resist and counteract this lateral movement of the haunches and apply horizontal conjugate thrust thereto, that part of the extrados from the point of rupture down to the pier is filled generally with backing of rubble masonry or concrete laid in horizontal layers. Semicircular arches require backing sufficient to produce conjugate pressures equal to the crown thrust. Segmental arches which have a horizontal thrust component at the spring requires less backing than semicircular ones.

Waterproofing and Drainage.

Previous mention has already been made of waterproofing. This is necessary to prevent water soaking into the joints and freezing, thereby tending to disintegrate the masonry. Waterproofing is necessary also to prevent drainage water leaking through the arch and discoloring or otherwise disfiguring the structure. To prevent such leakage it is customary to cover the upper surface of the arch and backing with a layer of bituminous concrete or clay puddle. Clay should contain enough sand to prevent the clay from cracking when dry. Waterproofing may be accomplished by applying a layer of rich mortar and surfacing it with neat cement, on top of which is poured a coating of tar, pitch or asphaltum. The upper surface of the backing must have sufficient slope to carry drainage water to the gutter, where it may be discharged through pipes built into either the arch soffit or the side spandrel walls.

Intermediate Piers.

In making preliminary designs of piers, use may be made of empirical formula to determine approximate sizes. Rankine's rule is to make the thickness of piers at spring from one-sixth to one-seventh of the span or arch for intermediate piers, and one-fourth of the span for abutment piers. Intermediate piers must be of sufficient area to resist crushing from the maximum loads, and in proportioning the base of pier the weight of the pier itself must be added to the imposed loads. Intermediate piers must also have sufficient stability to resist the overturning effect of unbalanced thrusts on the adjoining spans. Such unbalanced thrusts will occur if the adjoining spans are of different lengths, or if one only, is subject to live load. For such conditions the center of pressure shall fall within the middle third of pier base. Piers must be given sufficient spread at the base, so the pressure on the foundation will not exceed a safe unit. To neutralize the effect of unequal thrust on the piers from spans of different lengths, the shorter span may have a less rise with a correspondingly greater amount of filling. This will tend to produce a thrust from the smaller span sufficiently large to equal that from the longer one. Another method is to incline the shorter span upward so the thrust will act on the pier at a point somewhat higher than the corresponding thrust from the longer span. In writing on this subject, Rankine says: "Each pier of a series should have sufficient stability to resist the thrust which acts upon it, when one only of the arches which spring from it is loaded with a traveling load. That thrust may be roughly computed by multiplying the traveling load per lineal foot by the radius of curvature of the intrados at its crown in feet." The mathematical investigation of piers is shown in Figures 7, 9 and 10.

Abutment Piers.

Bridges having a series of spans should have abutment piers at intervals in order that the possible failure of one span would not cause the entire structure to fail. Abutment piers are useful also in allowing false work centers to be removed from some of the spans, without waiting for the completion of the entire structure. When spring lines can be located close to the foundations, it may be advantageous to make all piers, abutment piers. This was the case in the long masonry viaduct recently built at Santa Ana in California, on the line of the San Pedro, Los Angeles & Salt Lake Railroad. (See Engineering Record, September 9, 1905.) When it is impracticable to make all piers abutment piers, it will then be well to have every third or fifth one of the type. Such piers may be designed with a factor of safety against overturning of from one and one-half to two. It will be noticed that the point of intersection of the arch thrust with the load line through the center of gravity of the piers, falls lower in the abutment pier than in the intermediate ones, owing to the greater width of pier. This is an advantage and will bring the resultant pressure nearer to the center line of the pier base. Trautwine's approximate formula for the thickness of abutment piers at the springing is to make the thickness equal to one-fifth of the crown radius plus one-tenth of the rise, plus two feet.

Abutments.

In proportioning abutment piers, it is not necessary to keep the resultant pressure within the middle third of the base, if the maximum pressure at the outer edge does not exceed the allowable unit pressure. Trautwine's empirical rule for the thickness of abutments at the springs is the same as was given above for abutment piers. This approximate size will assist in establishing the correct or final one and the rule gives a thickness intended to be sufficient without depending upon the existence of earth pressure from behind. Abutments sustaining high banks of loose material, must be proportioned, not only for the arch thrust, but also as retaining walls.

There is frequently more masonry in the abutments of a bridge than in the span itself. For this reason it is desirable to consider carefully any opportunities for saving material in the abutments. Placing the arch spring down near the ground, greatly reduces the overturning moment on the abutments and causes a considerable saving of material. In bridges with several arch spans, even though the spring lines on the piers must be high to secure a clearance underneath the bridge, the springs at the end abutments may sometimes be kept down lower than the corresponding springs on the pier, or if abutments must be high, it may be economical to use ribbed abutments, cored out and reinforced with metal bars, if necessary.

The use of pavement ties of either wood or metal, will cause the arch thrusts to counteract each other, and thereby greatly reduce the size of abutments. This expedient has not been used to any great extent until recent years, and even now is used chiefly for bridges of reinforced concrete.

A wide and shallow waterway is more effective than a narrow but higher one of the same area. Figure 11 shows some possible abutment forms. At A and C are shown abutments where the concrete in front of dotted line, not only is of no service or benefit, but actually decreases the area of waterway and at the same time adds to the cost of the structure. It will be seen, however, that the abutment A is one of the most common forms used in nearly

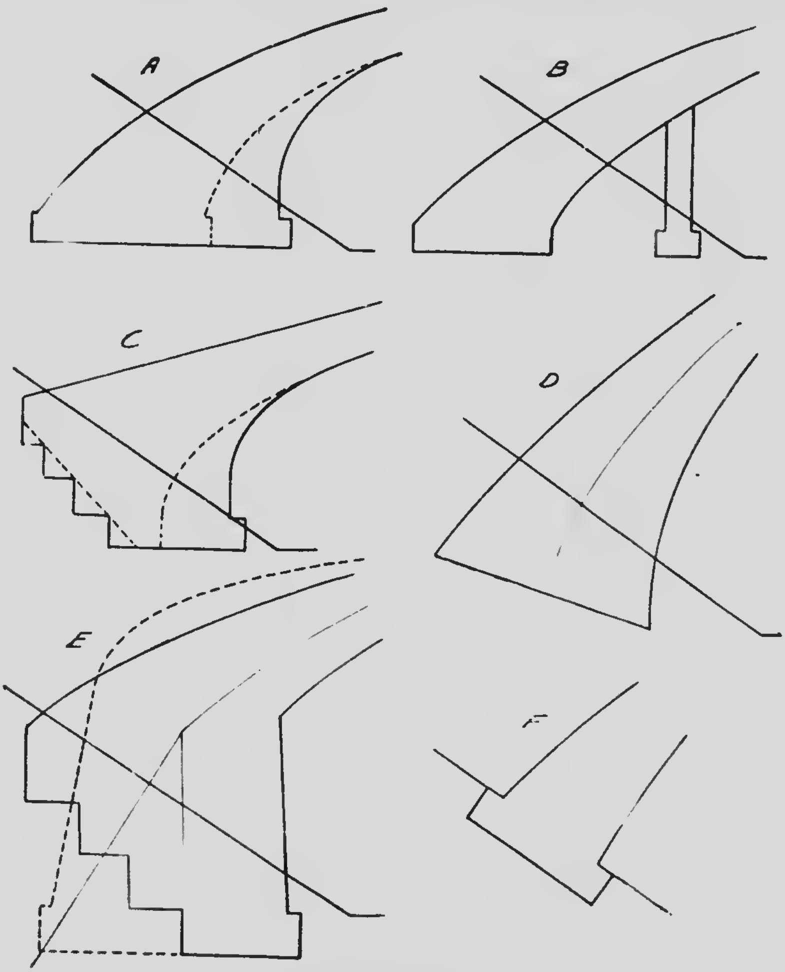


Fig. 11
ARCH ABUTMENTS.

all old arch bridges. If for any sufficient reason, vertical sides are desirable or necessary, it will be economy to build independent side walls, as shown at B, rather than waste material by making the whole abutment solid.

At E are shown old and new methods of construction. The dotted lines showing an abutment built on level foundation is the method given by Trautwine and the one generally used until recent years. It will be seen, however, that the forms shown at E in full lines is equally effective in transmitting thrusts to the soil, and requires somewhat less material. If vertical sides are not required, some additional material may be saved by using the method shown by dotted line at C. D is suitable for arches with considerable rise on hard soil or loose rock, and F shows a form of abutment in which the arch thrusts against solid rock.

In designing abutments, it is safer to discard the effect of conjugate earth pressure on the arch extrados. The abutments will then be somewhat heavier, but the error will be on the side of safety. Rankine says that the thickness of abutments is often from one-third to one-fifth of the radius of curvature at the crown. Flaring wing walls, 25 feet in height or less, rigidly connected to the abutment face, will ordinarily be safe with a base equal in width to one-fifth of the height. This is only half the thickness usually given to retaining walls, and is less because of the angular connection to the abutment face.

Foundations.

Piers and abutments must have sufficient spread at the base, so the load on the foundation will not exceed a safe unit. For soil, this will not ordinarily exceed from two to four tons per square foot at the outer edge of the pier, where pressure is the greatest. If piles are used, the same precaution will be taken. Sloping piles have occasionally been used in arch foundations for resisting the arch thrust, but they are more difficult to drive than plumb piles. The Jamestown Exposition bridge, Figure 28, has 26 plumb and 126 batter piles under each abutment. The maximum allowable load on piles should not exceed from 15 to 25 tons each, depending upon the penetration of the pile at the last blow of the hammer. Allowance must be made for the resultant pressure on the base falling outside of the center. It need not necessarily be confined to the middle third, provided the pressure on the foundations at the outer edge is not excessive.

In his treatise on Masonry Construction, Professor Baker gives the following values for safe bearing power of soils:

	Tons per square foot.
Rock equal to best ashlar	25 to 30
Rock equal to best brick masonry	15 to 20
Rock equal to poor brick masonry	5 to 10
Clay, dry thick beds	4 to 6
Clay—moderately dry thick beds	2 to 4
Clay—soft	1 to 2
Gravel and coarse sand well cemented	8 to 10
Sand—compact and well cemented	4 to 6
Sand—clean and dry	2 to 4
Quicksand, alluvial soil, etc	½ to 1

Expansion.

It is well to provide for possible expansion, so cracks will not appear in the finished surface. In the case of the Connecticut Avenue Bridge at Washington, shown on page 88, one-half inch expansion joints are provided throughout the entire height of the spandrels, from spring to the floor over the piers and across the roadway. These arches are 150 feet in length and semicircular. After the completion of the concrete arch bridge over Big Muddy River on the Illinois Central Railroad (See Engineering News, November 12, 1903) an examination was made during a period of several months, and almost no expansion whatever was discovered.

Surface Finish.

Various methods have been adopted for procuring satisfactory surface finish on concrete structures. Among these methods may be mentioned cement washing, tooling, sand blasting, rough casting or slap dashing, scrubbing, cold-water painting, and acid treating. The Connecticut Avenue Bridge at Washington has corners and moldings made of concrete blocks, and to remove form marks the body and flat face work were bush hammered.

The Walnut Lane Bridge at Philadelphia has a rough surface finish similar to pebble dash, but of coarser grain. The surface shows stone chips not larger than three-eighths of an inch in diameter, formed by washing the concrete face before the cement had hardened. A more expensive method of securing a finished surface is to build all exposed surfaces of cut-stone work, or a combination of stone and brick, using concrete for the body of the work only. The Green Island Concrete Bridge at Niagara Falls has surfacing on the spandrels and piers of cut stone, and other bridges have been similarly built at Indianapolis and elsewhere. Many bridges generally known as stone masonry bridges are stone only on the surface, with the body of piers, arches and backing composed entirely of concrete. The Rockville stone arch bridge built by the Pennsylvania Railroad Company over the Susquehanna River is of this construction. It has stone facing throughout, including the soffits, spandrels and piers. In building an ornamental concrete foot bridge over two lines of railroad at Como Park, St. Paul, to avoid the appearance of form marking on the finished surface of the bridge, the entire surface of the lagging and moulds was lathed and finished with fine plaster. In the National Zoological Park, Washington, is a concrete bridge faced on the spandrels and parapets with natural boulders, which extend down six inches or more below the concrete soffit. In San Francisco are several concrete bridges with rustic surface finish, made to represent natural boulders, but really formed of moulded concrete. These boulder and rustic surfaces are appropriate for certain wooded parks or rural places, but are not suitable for general adoption.

Engineering-Contracting for January 6, 1909, contains illustrations of concrete surface effects secured

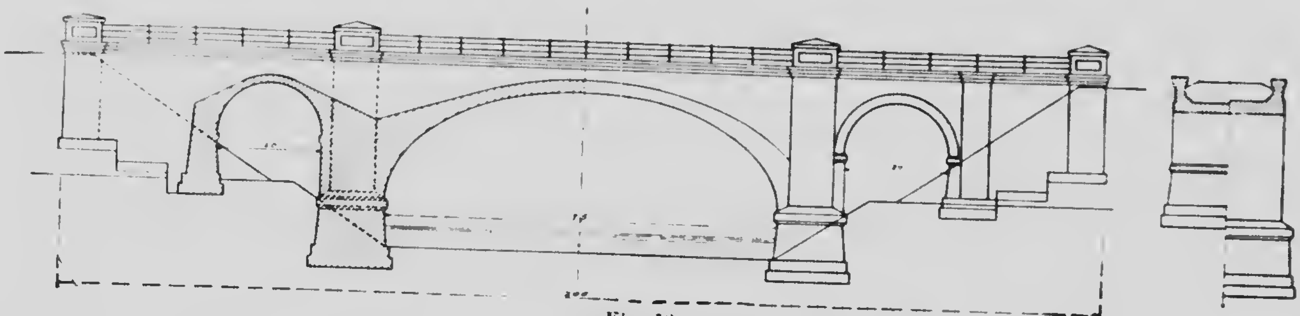


Fig. 12

Design for a Concrete Railroad Bridge.

The accompanying view shows a design for a concrete railroad bridge made by the author in the year 1906, to carry a single line of railway across a stream. As the location was remote from any large towns, the design was made without ornamentation. Comparative estimates were also made for bridges of other types and forms, and while a steel bridge would cost somewhat less than the design shown, the concrete arch was preferred on account of its being a permanent structure. The estimated cost for the concrete bridge was $17,000.

by various methods on laboratory samples. It will be understood, however, that better results would be obtained under these conditions than could be expected on larger surfaces where one of its chief difficulties is to produce uniform effects.

Stony Brook Bridge in the Boston Fenways has granite trimmings with spreckled brick facing, while the arch soffits are lined with glazed brick of varying patterns and colors.

There is a very artistic three-span arch bridge over the river at Des Moines, Iowa, that has vitrified brick facing. The spans are each 100 feet in length and elliptical in form. The brick facing with trimmings of a lighter color presents a very pleasing appearance.

Another method of preventing form marks from appearing on the concrete surface is to cover the lagging with a layer of fine clay and overlay the same with building paper.

Cost of Concrete Arch Bridges.

The cost of concrete bridges varies with local requirements and conditions. The following original formula gives the cost of solid concrete arch bridges for both railroads and highways. The formula is

$$C = F\left(\frac{HW}{100}\right)$$

where C is the cost in dollars per square foot of roadway, H the general height of the bridge at the center, W the total width and F a variable factor given by the following table:

When A is	200,	then F is	1.5				
” ”	500,	” ”	1.0				
” ”	1000,	” ”	.65				
” ”	1500,	” ”	.48				
” ”	2000,	” ”	.42				
” ”	2500,	” ”	.36				
” ”	3000,	” ”	.32				
” ”	3500,	” ”	.285				
” ”	4000,	” ”	.262	and	F′ is	.96	
” ”	5000,	” ”	.224	”	”	.95	
” ”	6000,	” ”	.20	”	”	.94	
” ”	7000,	” ”	.18	”	”	.93	
” ”	8000,	” ”	.164	”	”	.92	
” ”	9000,	” ”	.152	”	”	.91	
” ”	10000,	” ”	.141	”	”	.88	
” ”	11000,	” ”	.133	”	”	.86	
” ”	12000,	” ”	.125	”	”	.85	
” ”	13000,	” ”	.119	”	”	.82	
” ”	14000,	” ”	.113	”	”	.80	

As the height of the bridge multiplied by its width gives the cross sectional area, the function HW may be represented by the letter A. Factors F refer to arch bridges with complete soffit slabs, while factors F′ refer to arch bridges with partial soffit slabs, such as used in the Walnut Lane bridge in Philadelphia, and the Detroit Ave. bridge in Cleveland.

The cost of concrete bridges is affected more by natural conditions and the selection of the economic forms than by the live load to which these bridges are subjected. This is shown by the above formula applying equally to concrete arch bridges for both railroads and highways.

The weight of concrete and other materials is greater than the imposed live load and the live loads are not, therefore, the chief considerations in determining the ultimate cost.

The formula clearly shows that concrete arch bridges vary in cost in proportion to the product of their weight and width. Bridges with a small cross sectional area cost as low a price as $2.50 per square foot of floor surface, while large monumental bridges may cost as high as $16.00 per square foot.

The formula also clearly shows the great economy in using partial in place of complete soffit slabs, and this economy may be still further increased by the use of ribbed arch designs. Ribbed arches are not, however, generally suitable for construction in solid concrete and the treatment of this style of arch will therefore be taken up later, with the design of arches in reinforced concrete.

Table No. 1, giving details of concrete bridges, gives also the total cost of these structures.

Design for a Concrete Arch, 60 Feet Center to Center of Intermediate Piers. Clear Span 53 Feet. Rise 10 Feet.

The bridge consists of a series of arches to carry a street over a number of railroad tracks. The span was arbitrarily fixed at 60 feet center to center of intermediate piers, or 53 feet in the clear. This provides clearance for four lines of tracks, 13 feet apart on centers. For a low structure of this height, shorter spans might have been more economical, but this length was selected that the clearance way for the tracks would not be too greatly obstructed with piers. The headroom underneath is shown on Figure 7, and is the height generally required by railroad specifications, being 21 feet from the top of rail in the center of track nearest to the pier. The elliptical form was selected for the reason that, with the given clearance, it allows the springing line to fall lower than any other form and in this case is 15 feet above the ground. As the viaduct is a long one, it was desirable to keep the entire height and the corresponding cost down to the lowest possible amount. A minimum rise of one-fifth the span was therefore selected, amounting to 10 feet from spring to crown. The rise is the semi-minor axis of the ellipse and not the effective rise of the line of pressure, which is used later in determining the crown thrust and pier reactions. The approximate rule for the thickness of intermediate piers is to make the thickness of such piers one-sixth to one-seventh of the length of span. This would produce a thickness of pier from 7 to 8 feet at the spring and 7 feet was selected for a trial. To determine an approximate crown thickness, Rankine's rule was used. For a series of arches, it is $\sqrt{.17 \text{ Radius}}$. This requires that the radius be known. Lay out an ellipse graphically by the method of five centers, and the radius is found to be 72 feet. Rankine's rule, as above, gives a thickness of 3.5, while Trautwine's rule for the approximate thickness is given in his book, page 617, and is 2.2 feet. Try a thickness of 2.5 feet. The grading of the bridge up to a higher level in order to secure a greater rise for the arch was considered, but as this increased the quantities of material in the superstructure, and would effect a saving only in the abutment piers, the plan was not adopted. A thickness of crown filling of 2.5 feet was assumed from the extrados of the arch to the pavement surface.

The entire portion of the bridge above the intrados was then divided into strips, and the weight for each of these strips calculated, on the assumption that earth filling weighs 100 pounds per cubic foot, and masonry 160 pounds per cubic foot. A live load of 150 pounds per square foot was assumed on the roadway. The weight was computed for each strip and noted on Figure 7 in their respective places. The amount of conjugate thrust was then found by taking the intensity of such thrust at one-third the weight of earth and live load above it. These were also noted in their proper places. Center lines were then drawn through each strip, and a load diagram constructed by drawing in order the various vertical and horizontal loads from A to B, as shown in Figure 7. A trial pole P′ was selected and lines drawn connecting each of the load points on *a*B with P′. The corresponding funicular polygon *ay* was drawn with lines parallel to the lines in the force polygon AP′, BP′, etc. This is evidently not the correct position of the pole, for the resulting funicular polygon lies almost entirely outside of the arch. By prolonging the last string of the funicular polygon to its intersection at *g* with the horizontal through the arch center from *a*, we find the point of application of the resultant of all the imposed loads, which is at *g*. The direction of the resultant pressure would be parallel to AB. As the position of this point is constant for any other position of pole, we may draw through *y* a line *yg*. This will represent the direction of the actual pressure of the arch on the abutment. Through B in the force polygon draw a line parallel to *yg* intersecting the horizontal through A at P. The point P will be the correct position of the pole, and the distance AP measured to the same scale as the line AB, will represent truly the amount of the crown thrust. In this case it is 42,000 pounds. This investigation is for a portion of the bridge one foot in length at right angles to the diagram. Pressures at the various points in the arch correspond to the lengths of lines in the force polygon. At the pier for full loading, the pressure is 48,000 pounds.

Uneven Loading.

Lines of resistance were next drawn for unsymmetrical loading as shown in Figure 9. This has already been quite fully described under the head of Partial Loads.

Required Area in Arch.

Using a maximum pressure of 400 pounds per square inch as a safe working unit on concrete at the outer edge, or 200 pounds per square inch mean pressure, the required area in the arch is $\frac{42000}{200}$ or 210 square inches. This requires a depth of arch of 18 inches. We have already assumed a depth of 30 inches so the arch is secure against crushing. With a depth of 30 inches, the mean pressure on the concrete is only 116 pounds per square inch, instead of the 200 pounds which is proposed.

Intermediate Piers.

First figure the required size of pier to sustain the total load in compression. The total weight from the arches and the live load is 54,000 pounds. Assume the material of the pier to be concrete, with an allowable unit pressure on the outer edge of 400 pounds per square inch. For a mean working pressure assume half of this amount, or 200 pounds per square inch. The required area in the pier at springing to sustain direct loads is therefore $\frac{54000}{200}$ or 270 square inches. As the assumed width of pier at the top was 7 feet, the case of full uniform loading is evidently not the governing consideration.

Consider next the case of equal spans thrusting on the pier, one with full dead and live load and the other with dead load only. The thrusts in these two cases are 48,000 and 42,000 pounds respectively. The total load on the pier at the level of the ground is therefore as follows:—

	Pounds.
From fully loaded span	26,000
From partly loaded span	23,000
Weight of pier	18,000
Total	67,000

By combining this load with the arch thrust, we find the resultant pressure, the line of which intersects the base at ground level one foot from the center of the pier, which is well within the middle third. This result may very easily be checked analytically. The width of pier at base is 14 feet, which was found as follows:—

The total pressure on the soil is:—

	Pounds.
From bridge	54,000
From pier	27,000
Total	81,000

Allowing a mean pressure of 6,000 pounds per square foot on the soil, the required width of pier is $\frac{81000}{6000}$ or 13.5 feet. If the soil will not sustain 6,000 pounds per square foot, which, allowing for uneven pressure, equals 4 to 5 tons per square foot at the outer edge, piles will then be required.

Abutment Piers.

In proportioning the abutment pier, stability is the chief consideration. It must be stable against the thrust of arch from one side only. This arch thrust intersects the center of the pier at a distance of 14 feet above the ground. The overturning moment from this thrust is therefore 33000×14, foot pounds. Using a factor of one and one-half against overturning, the necessary moment of stability is 33,000×14×1½, or 69,300 foot pounds. Next proceed to find the half width of pier base at ground level. Calling this half width x, the required moment of stability in foot pounds is

$23000x+(2x\times22\times160)x=69{,}300$ foot pounds. In the above, 22 is the total height of the pier from the top of the ground to the top of the backing, and 160 is the weight of the pier material per cubic foot. From the above we obtain a quadratic equation, and solving, we find the value of x to be 8.45 feet. This would be for a pier with vertical sides. For sloping sides, take a half width at the base of 9 feet, as shown in Figure 9. This size of pier is then amply stable against overturning.

Coring out the haunches by means of interior spandrel walls, would evidently be no economy in so flat an arch. The cost of such walls and arching would be greater than the saving in the arch ring from the reduced dead load and the less amount of filling.

Illustrations of Concrete and Masonry Bridges.

The foregoing table gives a list of arch bridges, the main arches of which are built of solid concrete without metal reinforcement. In one of these, however—the railroad bridge over the Vermillion River at Danville—reinforcement was actually used in the main arch, but was adopted only for the purpose of better uniting the concrete and preventing cracks from change of temperature. In several of the other bridges, noted in the table, metal reinforcement was used in spandrel arches, or other minor parts, but as already stated, the main arches have been designed with no provision for tension in any part of the arch section, and consequently no need for reinforcing metal to resist direct stresses.

The table is not intended to be comprehensive or complete, but gives some details of a few of the largest concrete spans, the main arches of which are designed without reinforcement. In reference to the Hudson Memorial Bridge, noted in this table, and illustrated on page 77, the design calls for a large amount of metal reinforcement, not for the purpose of resisting any tensile stresses in the arch, but rather to supplement the concrete in resisting direct compression. This is a new principle in arch construction, not previously used.

Illustrations and descriptions of two old Roman bridges are also given for the purpose of calling attention to the superiority and permanence of masonry bridges over those of any other known type or material. They have existed for centuries, and such bridges should endure after metal bridges have disappeared.

Fig. 13
PONTE ROTTO, ROME

Ponte Rotto, Rome.

As it stands to-day, this old bridge has three stone arch spans, and a suspension bridge, spanning the gap where other arches originally stood. The present bridge stands on the site of the old Pons Aemilius, built B. C. 178-142, which was the first stone bridge over the Tiber at Rome. The three remaining arches date from Julius III, and are richly ornamented. Two arches were carried away by a flood in 1598, and have never been replaced. The bridge seems to be unfortunately located, as it has been carried away at least four times, the first time in A. D. 280. It was erected by Caius Flavius, and is probably the first appearance of the arch in bridge construction. It has semicircular arches and a level roadway. The two end arches were shorter than the three intermediate ones. It is called also Pons Palatinus, Senators' Bridge, and Pons Lapidens. The bridge is similar in construction to the other old stone bridges of Rome, and is built of peperino and tufa, faced with blocks of travertine anchored into the body of the masonry. It will be seen from the illustration that the spandrels and parapets are highly ornamented with carved panel work and each of the piers above the arches and foundations are penetrated with smaller arch openings. The panel work has disappeared from the left shore span and plainly reveals the plan of construction. It will be seen that the arch ring is built of

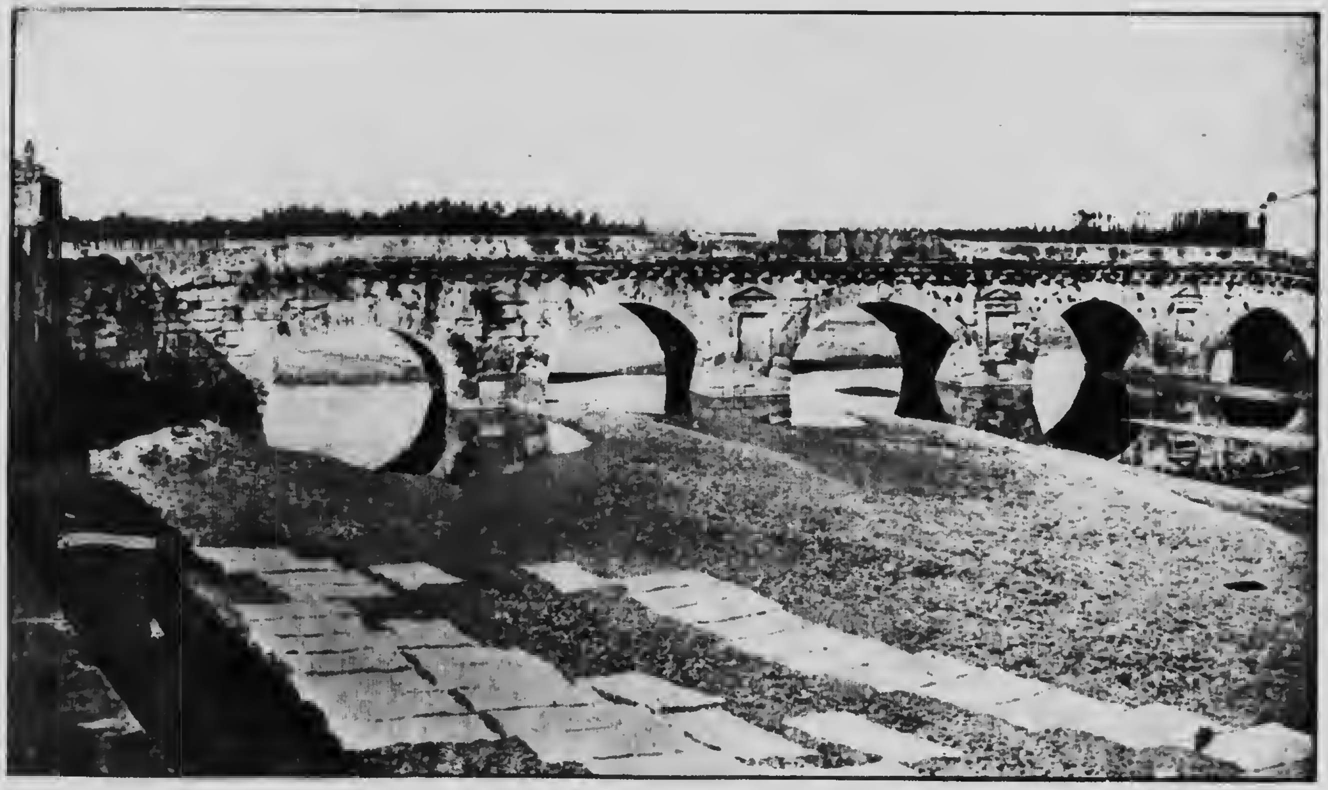


Fig. 14
BRIDGE OF AUGUSTUS, RIMINI, ITALY

different material and differently laid than the filling above it, and that numerous openings occur in the backing which were doubtless used for the purpose of anchoring the ornamental facing to the body of the structure. It is well known that, in the construction of bridges and aqueducts built by the Romans and others in early times, a large amount of concrete was used.

Bridge of Augustus at Rimini.

The old Roman bridge crossing the River Marachia at Rimini, is supposed to have been built during the reign of Emperor Augustus, about 14 A. D. It has five arch spans, with very heavy piers. The details that still remain show that originally the bridge was very ornamental. There are niches at the piers, and the heavy stone cornice is carried on numerous brackets. The arches are all semicircular, the end ones having a span of 23 feet, while the three intermediate ones have spans of 28 feet.

Henry Hudson Memorial Bridge.

(Reinforced Concrete Design.)

It is proposed to erect on an extension of Riverside Drive in the City of New York, a Memorial bridge over Spuyten Duyvil Creek, to commemorate the explorations and discoveries of Henry Hudson. The design accepted by the Municipal Art Commis-

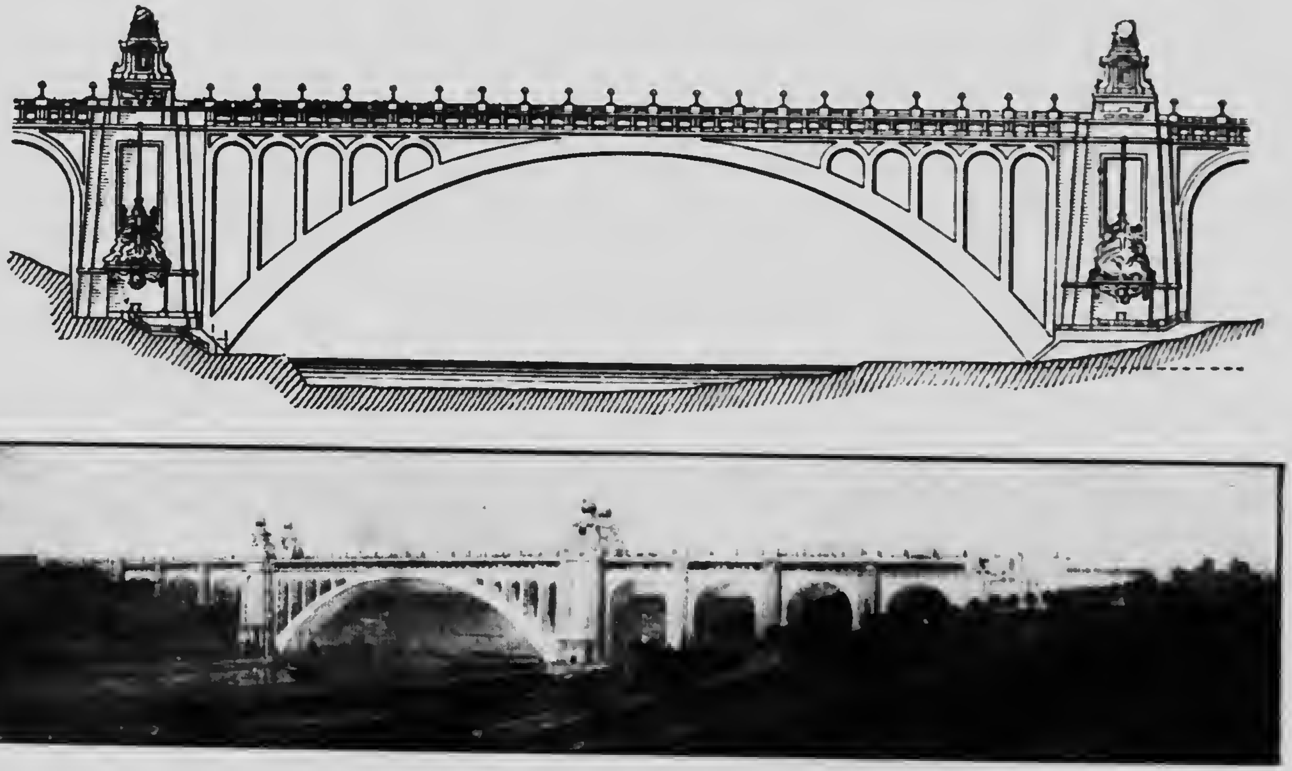
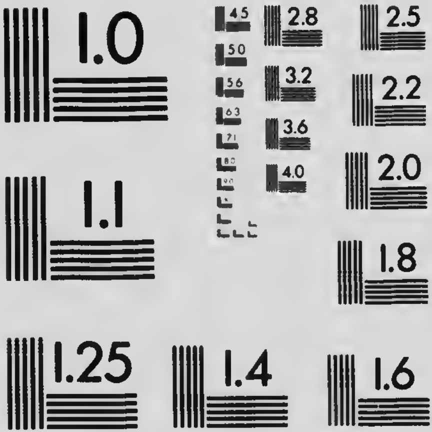


Fig. 15
PROPOSED HUDSON MEMORIAL BRIDGE, NEW YORK CITY

MICROCOPY RESOLUTION TEST CHART

(ANSI and ISO TEST CHART No. 2)



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sion of the City of New York is herewith shown. Previous designs showing the principal span framed in steel were rejected as being inappropriate for a great memorial bridge. There will be one span with a clear length of 703 feet. and seven other semicircular arch spans with clear lengths of 108 feet. The total length of the structure will be 2.840 feet. The main arch span will have a rise of 177 feet. and will contain a large amount of steel. used. not as concrete reinforcement ordinarily is. to resist tensile stresses. but rather to assist in resisting the compressive stresses in the concrete. and thereby reduce the amount of masonry. The arch will have a crown thickness of 15 feet. There will be two decks. the upper one carrying a 70-foot roadway and two 15-foot sidewalks. while the lower deck will be 70 feet in width. and will carry four lines of electric railway. It is the intention to omit the construction of the lower deck at the present time. The design provides for a clear headroom of 183 feet under the main arch. The main piers will be 180 feet in width. The estimated cost is $3.800.000. The illustration shows the bridge as it will appear to an observer looking out over the Hudson River. with the Palisades in the distance. The design was made by the bridge department of the City of New York. at which time C. M. Ingersoll was Chief Engineer. L. S. Moisseiff Engineer in Charge. Wm. H. Burr, Consulting Engineer, and Whitney

Warren, Architect. The next longest masonry arches of the world are as follows:

	Feet. span.
Stone arch bridge over Adda River	230
Stone arch bridge at Luxemburg, Germany	278
Stone arch bridge at Plauen, Germany	295
Concrete arch bridge at Gruenwald	230
Concrete arch bridge at Walnut Lane, Philadelphia	233
Stein-Teufen bridge, Switzerland	259
Concrete arch bridge at Rocky River, Cleveland	280
Aukland, New Zealand	320

Aukland, New Zealand, Bridge.

A reinforced concrete arch bridge is being built on the North Island, at Aukland, New Zealand, with a clear span of 320 feet—the longest in existence. Several longer ones have been projected, one over the Mississippi River at Fort Snelling, Minn., with two spans of 350 feet, but none built. The Aukland bridge has, besides the 320-foot center span, two 35-foot and four 70-foot spans, with a total length of 910 feet. It is 40 feet wide, and the roadway is 147 feet above the valley. The two arch rings are hinged at the springs and center. It was commenced in February, 1908, and the contract calls for completion in two years. It adjoins a residential district, and at one end are the graves of New Zealand pioneers.

Monroe Street Bridge, Spokane, Wash.

In the city of Spokane, Wash., plans are prepared for building a four-span concrete bridge to carry Monroe street at a height of 140 feet above the Spokane River. The main arch has a clear span of 281 feet, and is divided into two ribs, 16 feet wide and 6 feet thick at the crown. It will have open spandrels and overhanging sidewalks, with Dutch towers at the ends for public lavatories. The bridge will replace the old steel cantilever built 17 years ago. It will have a 50-foot roadway and two 9-foot sidewalks, making a total width of 71 feet and a total length of 791 feet. The main arch will be segmental and the remaining ones semicircular. The deck will be carried on solid cross spandrel walls, 20 feet apart. The ground on the north side of the river is naturally suited for an arch bridge, but on the south side the plan proposes an abutment carried down to 140 feet below street level, consisting of four parallel walls, each 4 feet in thickness, joined by numerous cross struts and braces. See Fig. 16. J. C. Ralston, City Engineer.

Rocky River Bridge, Cleveland, Ohio.

A concrete arch bridge with the longest masonry span in America is now being built over Rocky River on Detroit avenue, at Cleveland, Ohio. It will have a central span of 280 feet and five approach spans of 44 feet each. It will carry a 40-foot roadway and two sidewalks 8 feet wide each. The total width over railings will be 60 feet and the total length 708 feet. The main span consists of two sep-

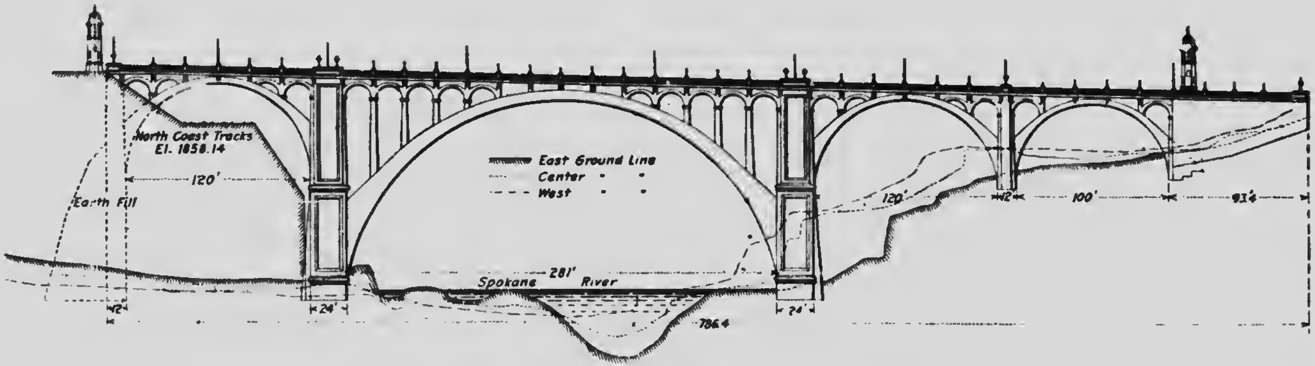


Fig. 16

ELEVATION OF CONCRETE BRIDGE ACROSS MONROE ST., SPOKANE, WASHINGTON

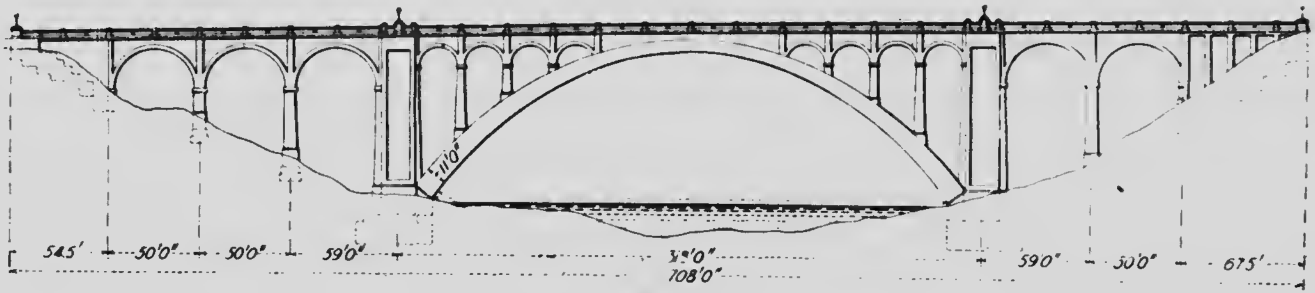


Fig 18

ROCKY RIVER BRIDGE

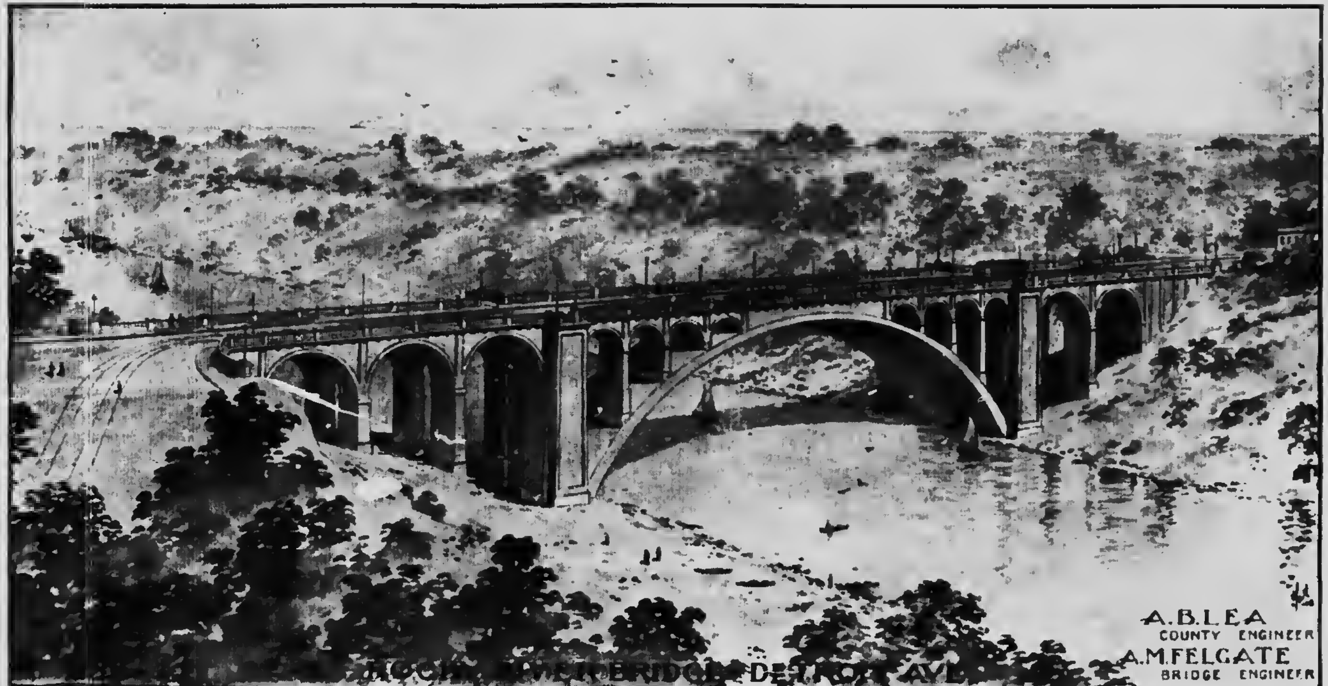


Fig. 17
ROCKY RIVER BRIDGE, CLEVELAND, OHIO

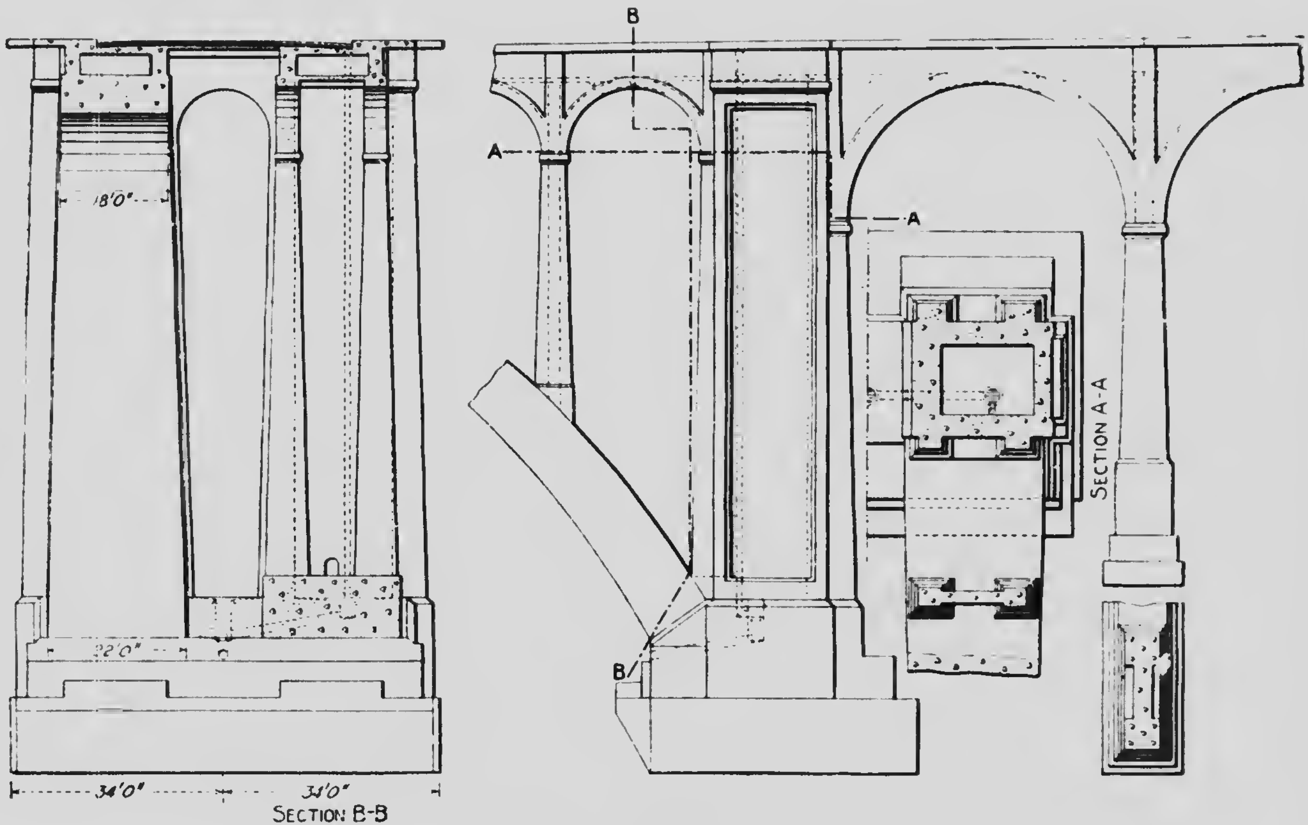


Fig. 19
ROCKY RIVER BRIDGE

arate arch rings 18 feet wide at the crown, and placed 16 feet apart. On these arches the deck is to be carried on cross-spandrel walls. The roadway level is 94 feet above the surface of low water and the pavement will be of brick, with two lines of track for heavy suburban cars. Beneath the floor are to be two subway chambers, 3 feet by 11 feet for the placing of pipes and wires. The main arch rings will contain no steel reinforcement, as the calculations show that no tension can at any time occur in any part of the arch. The sidewalks project out over the face walls about five feet, and are supported on brackets. The entire structure will be built of concrete. It will be quite similar to and 47 feet longer than the Walnut Lane Bridge at Philadelphia. The only longer masonry arch span in existence is the one at Planen, in Germany, with a span of 296 feet, built of hard slate. Other projected long-span bridges are that over the Neckar River at Manheim, with a span [illegible] feet, and the Hudson Memorial Bridge in N[illegible] City, with a span of 703 feet. The Rocky River Bridge was designed under the direction of A. B. Lea, County Engineer, by A. M. Felgate, Bridge Engineer. It is under construction by Schillinger Brothers, contractors of Chicago. Wilbur J. Watson, Engineer.

Walnut Lane Bridge, Philadelphia.

Walnut Lane crosses the Wissahickon valley on a new concrete bridge at a height of 147 feet above the river bed. At the time of completion it was the

Fig. 20
WALNUT LANE BRIDGE, PHILADELPHIA, PA.

longest concrete masonry bridge, having a clear span of 233 feet. It consists of two separate arch rings, 18 feet wide at the crown, increasing to 21 feet 6 inches at the springs. At the crown the two rings are separated by a space of 16 feet. The double rib construction is similar to that used in the stone arch bridge at Luxemburg, Germany, having a span of 275 feet. The main arch is an approximate ellipse, has a rise of 73 feet, and carries 10 cross walls which support the floor system. There are also five semicircular approach arches with clear spans of 53 feet. The bridge connects Germantown and Roxborough, two residential suburbs of Philadelphia. It has a 40-foot roadway, and two 10-foot sidewalks. The entire structure is solid concrete, not reinforced, excepting in certain minor details. The surface finish is rough, somewhat similar to pebble dash, but of coarser grain. The exposed surface shows stone chips of not over three-eighths inch in size, formed by washing before the cement had hardened. The total length of bridge over all is 585 feet, and the cost $259,000. George S. Webster, Chief Engineer, Bureau of Surveys. H. H. Quimby, Bridge Engineer. Reilly & Riddle, Contractors.

Connecticut Avenue Bridge, Washington.

Connecticut Avenue, one of the chief thoroughfares of Washington, is carried over Rock Creek valley near its junction with the Potomac on a new concrete arch bridge, about three miles from the

Fig. 21
CONNECTICUT AVENUE BRIDGE, WASHINGTON, D. C.

Capitol building. The roadway is 120 feet above the valley below, and is carried by five semicircular arches of 150-foot span, and two end arches of 82-foot span. It has a 35-foot roadway, and two sidewalks 8 feet wide each, making a total width of 52 feet, a clear length between abutments of 1,068 feet, and a total length of 1,341 feet. It was commenced in 1889, and completed in 1908. The main arches are hingeless with no reinforcing, but the spandrel arches have steel reinforcement. As the bridge is located in a fine residential district, its aesthetic appearance was a matter of considerable importance. The face rings of the arch, pier corners, mouldings and all trimmings below the granite coping, are moulded concrete blocks. The remaining part of the exposed concrete surface is bush hammered, for the purpose of presenting a more uniform and pleasing appearance. The cost of the falsework was about $50,000, but on this there was a salvage of about $15,000. The cost of framing the falsework was $9 per thousand feet of lumber. Moulded cement blocks cost $15 per cubic yard. The total cost of the structure complete was $850,000, equal to $639 per lineal foot, or $12.30 per square foot of floor surface. It is built from a modification of the prize design submitted by the late George S. Morrison. The original competitive designs estimated to cost from $370,000 to $1,100,000 were published in Engineering News January 27, 1898. It was built under the direction of Col. John Biddle, Engineer Commissioner of the District of Columbia. W. J.

Fig. 22
BIG MUDDY RIVER BRIDGE, ILLINOIS

Douglas, Bridge Engineer. E. P. Casey Consulting Architect.

Big Muddy River Bridge, Illinois.

Two tracks of the Illinois Central Railroad are carried over Big Muddy River near Grand Tower, Illinois, on a new three-span concrete arch bridge. It was built in 1903 to replace an old steel bridge, and for this reason the piers remain in their original location. The bridge has three clear openings of 140 feet, and a total length of 463 feet between faces of abutments. It is 32 feet wide, contains 12,000 cubic yards of concrete, and cost complete $125,000. The arches are true ellipses with semi-minor axes of 30 feet. The old piers were 9 to 10 feet in thickness, and the new ones, which were built around the old ones, are 22 feet thick. The main arches are solid concrete, the only reinforcing being in the spandrel arches supporting the floor, and this was used for convenience in erection. As built, with spandrel arches and openings, the cost was somewhat greater than if it had been filled. The designer explains that open spandrels were used for the purpose of reducing the load on the foundations. Big Muddy River Bridge was designed by H. W. Parkhurst, Engineer for the Illinois Central Railroad Company.

Santa Ana Bridge, California.

This structure carries the new line of the San Pedro, Los Angeles and Salt Lake Railroad, over Santa Ana River, near Riverside, California. The

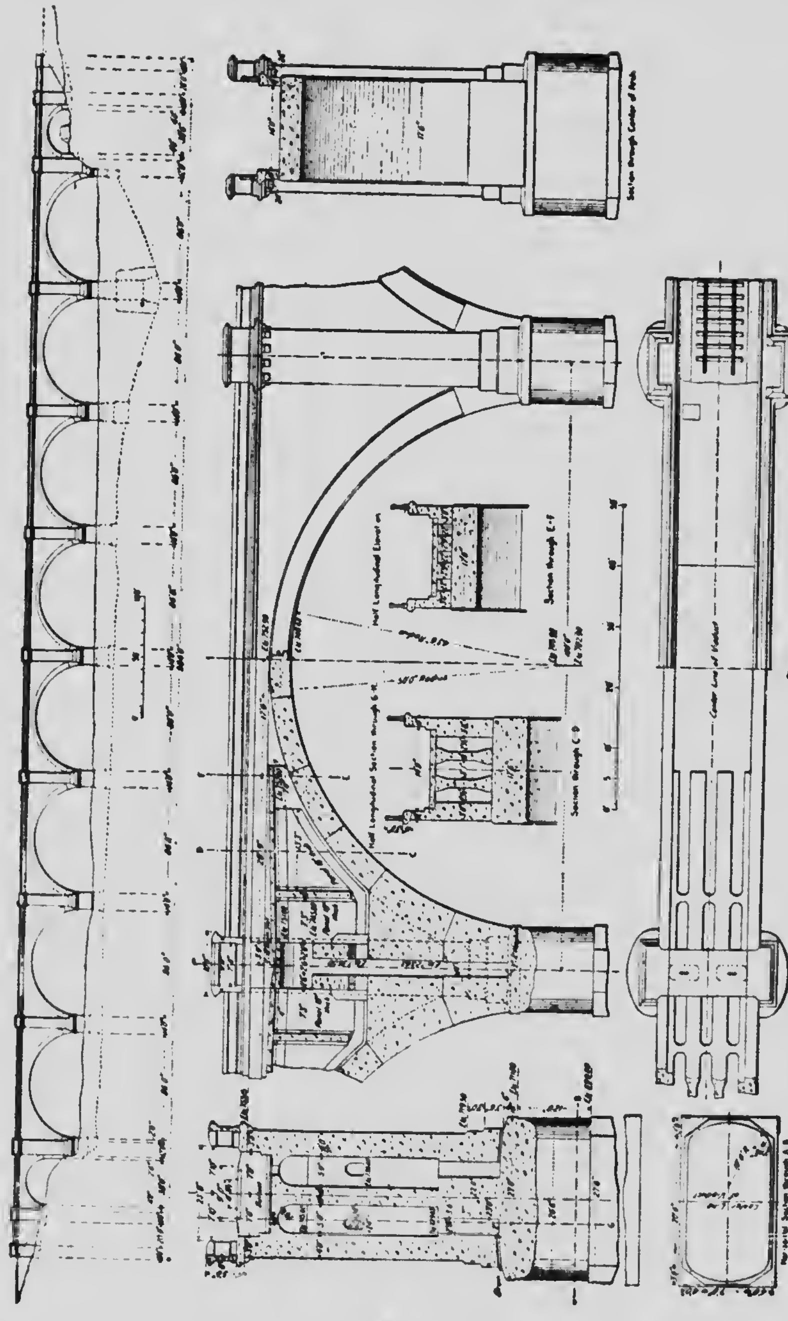


Fig. 23
SANTA ANA BRIDGE, RIVERSIDE, CALIFORNIA

Fig. 24
SANTA ANA BRIDGE, RIVERSIDE, CALIFORNIA

bridge has a total length of 984 feet, and the deck is 55 feet above the water. It was built during the years 1902 to 1904 under the direction of Henry Hawgood, who was then Chief Engineer for the above railroad company. It contains eight semicircular arches of 86 feet clear span, and two end spans of 38 feet. The piers are 14 feet in thickness, making the distance on centers of main piers 100 feet. It is made of solid concrete without reinforcement, contains 12,500 cubic yards of concrete and cost $185,300. The thickness of arch at crown is 3 feet 6 inches. and the width across soffit is 17 feet and 6 inches.

A letter from Mr. Hawgood to the author in reference to this bridge states as follows:—"The Santa Ana viaduct has given entire satisfaction from an operating standpoint. There has been no cost for maintenance during the five years it has been in service, whereas a steel bridge would certainly have involved some expense during the same period. In positions such as the Santa Ana Viaduct where there is no limitation as to headroom, I consider the simple concrete structure without reinforcement a better structure than one reinforced. The greater weight of concrete required forms a much heavier mass to take up the impact of heavy high speed trains. The absence of vibration is very marked. It is a parallel condition to a heavy anvil under a steam hammer—the heavier the anvil, the longer it will last."

TABLE I

LIST OF CONCRETE BRIDGES

Number	LOCATION.	Over.	No. of Spans.	Length of Span, ft.	Rise, ft.	Total Length, ft.
1	Hudson Mem., New York........	Spuyten Duyvil.	1	703	177	2840
2	" " " "	" "	7	108	.....	.....
3	Auckland, N. Z...........................		1	320	.....	910
4	" "		2	35	.....	.....
5	" "		4	70	.....	.....
6	Detroit Avenue, Cleveland	Rocky River...	1	280	80	708
7	" " "	" " ...	5	44	22	.....
8	Walnut Lane, Philadelphia	Wissahickon. ...	1	233	73	585
9	" " "	"	5	53	26.5	.....
10	Gruenwald, Bavaria...........	Isar..........	2	230	42	720
11	Ulm, Germany..................	Railway Yards .	1	210	.....	.....
12	Kempton, Germany............	Iller River......	1	211	87	500
13	" "	" "	3	68	.....	.....
14	Lautrach, "	" "	1	187	32	280
15	Neckarhausen, Germany..	Neckar	1	165	13.5	.....
16	Munderkingen, Wurtemburg. . ..	Danube........	1	164	16.4	.....
17	Connecticut Ave., Washington. ...	Rock Creek.....	5	150	75	1341
18	" " " . ..	" "	2	82	41	.....
19	Portland, Pennsylvania	Delaware River .	5	150	40	1450
20	" "	" "	2	120	.....	1450
21	" "	" "	2	30	.....	1450
22	Vauxhall, London.............	Thames........	1	144.6	18.5	.....
23	Grand Tower, Illinois...........	Big Muddy River	3	140	30	483
24	Inzighofen, Germany............	Danube........	1	141	14	150
25	Edmondson Ave., Baltimore	Gwynns River ..	1	139	44	542
26	" " "	" " ..	3	60	30	542
27	Borrodale, Scotland............	Borrodale Burn	1	127.6	22.5	.....
28	" "	" "	2	20	.....	.....

TABLE I—Continued

LIST OF CONCRETE BRIDGES

Width, ft.	Height, ft.	Form of Curve.	Date.	Highway or Railroad	Cost, $.	Engineer.	Reference. N., Eng. News R., " Record	Number.
80	183	....	....	H.	$3,860,000	Moisseiff	N., Nov. 21, '07	1
"	"	....	....	"	....	"	R., Dec. 28, '07	2
45	147	....	1908	H.	....	....	....	3
"	...	....	"	"	....	....	....	4
"	...	....	"	"	....	....	....	5
60	94	Seg.	1909	H.	208,300	Felgate	....	6
60	94	C.	"	"	....	"	....	7
60	147	E.	1906	H.	262,000	Webster	N., Jan. 31, '07	8
60	147	C.	"	"	....	"	N., Jan. 31, '07	9
26	....	....	1904	H.	65,000	Morsch	N., Feb. 23, '05	10
46	....	....	1905	H.	45,000	....	N., Mar. 15, '06	11
54	...	C.	1906	R. R.	....	....	....	12
....	....	....	....	....	....	....	....	13
13.8	40	Seg.	1906	R. R.	21,600	....	....	14
15.8	....	....	1903	H.	....	Leibbrand	N., May 2, '07	15
28	....	....	1893	H.	21,420	Leibbrand	....	16
52	120	C.	1907	H.	850,000	Morrison	N., Mar. 26, '08	17
52	...	C.	....	....	....	....	....	18
34	65	El.	1908	R. R.	....	Bush	R., Aug. 15, '08	19
34	65	....	1908	R. R.	....	Bush	R., Aug. 15, '08	20
34	65	....	1908	R. R.	....	Bush	R., Aug. 15, '08	21
....	....	....	1899	....	....	Binnie	....	22
33	45	E.	1902	R. R.	12[illegible]	Parkhurst	N., Nov. 12, '03	23
12.5	20	Seg.	1895	H.	6,650	Leibbrand	N., Sept. 17, '96	24
60	70	....	1909	H.	183,300	....	R., June 19, '09	25
60	70	C.	"	....	....	....	....	26
....	....	....	1898	R. R.	....	Simpson	N., Feb. 9, '99	27
....	....	....	....	....	....	....	....	28

TABLE I—Continued

LIST OF CONCRETE BRIDGES

Number.	LOCATION.	Over.	No. of Spans.	Length of Span, ft.	Rise, ft	Total Length, ft.
29	Sixteenth St., Washington.	Piney Creek. . . .	1	125	39	272
30	Kirchheim, Wurtemburg.	Neckar.	4	94.6	19	450
31	Hainsburg, New Jersey.	Paulins Kill.	5	120	60	1100
32	" " "	" "	2	100	. . . . .	1100
33	Miltenburg, Germany	Main.	2	112	17.7	733
34	" "	"	2	107	. . . . .	. . . . .
35	" "	"	2	102	. . . . .	. . .
36	Pittsburg, Pennsylvania.	Silver Lake. . . .	1	100	50	600
37	" "	" "	5	80	40	. . . . .
38	Thebes, Illinois.	Mississippi.	1	100	50	. . . . .
39	" "	"	11	65	32.5	. . . . .
40	Danville, Illinois.	Vermillion.	1	100	40	330
41	" "	"	2	80	30	. . . . .
42	Mechanicsville, New York.	Anthony Kill. . .	2	100	. . . . .	. . . . .
43	" " "	" " . . .	1	50	. . . . .	. . . . .
44	Imnau, Bavaria.	Eyach.	1	98	9.8	110
45	Wyoming Ave., Philadelphia.	Frankford Creek.	2	98	28	200
46	Brookside Park, Cleveland.	Big Creek.	1	92	9	125
47	Riverside, California.	Santa Ana.	8	86	43	984
48	" "	" "	2	38	19	. . . . .
49	Boulevard, Philadelphia.	Tacony Creek. . .	3	80	14	350
50	Long Key, Florida.	Atlantic.	180	50	25	10500
51	Mannheim	Neckar.	1	365	. . . . .	. . . . .
52	Larimer Ave., Pittsburg.	Beechwood Boul	1	300	. . . . .	. . . . .
53	Spokane	Spokane	1	281	115	791
54	"	"	2	120	60	. . . . .
55	"	"	1	100	50	. . . . .
56	Almendares, Cuba	. . . .		190	. . . . .	. . . . .

TABLE I—Continued

LIST OF CONCRETE BRIDGES

Width, ft.	Height, ft.	Form of Curve.	Date.	Highway or Railroad.	Cost, $.	Engineer.	Reference. N., Eng. News R., " Record	Number.
25	50	Par.	1906	H.	50,000	Douglas	R., Jan. 26, '07	29
19	40	Seg.	1898	H.	46,600	....	N., Mar. 29, '00	30
34	115	C.	1908	R. R.	....	Bush	R., Aug. 15, '08	31
31	115	.....	....	.....	....	....	....	32
23	22	Seg.	1899	H.	101,000	Fleischman	N., July 25, '01	33
....	....	.....	....	.....	....	....	....	34
....	....	.....	....	.....	....	....	....	35
51	70	C.	1905	R. R.	....	Brown	R., May 6, '05	36
51	70	C.	....	.....	....	....	....	37
28	....	C.	1903	R. R.	....	Nobel	N., Nov. 20, '02	38
28	....	C.	....	.....	....	....	....	39
33	90	.....	1905	R. R.	....	Duane	R., Mar. 3, '03	40
33	....	.....	....	.....	....	....	....	41
....	....	.....	....	El. R	....	....	N., Nov. 5, '03	42
....	....	.....	....	.....	....	....	....	43
82	15	Seg.	1896	H.	4,285	Leibbrand	N., Feb. 16, '99	44
80	32	.....	1908	H.	102,000	Quimby	R., Feb. 27, '09	45
12.7	12	El.	1905	H.	....	Zesiger	N., May 10, '06	46
17.6	55	C.	1904	R. R.	185,000	Hawgood	R., Sept. 9, '05	47
....	....	C.	1904	R. R.	....	....	....	48
100	30	Seg.	1908	H.	100,000	Webster	R., Mar. 13, '09	49
15	30	C.	1904	R. R.	....	Carter	N., Oct. 19, '05	50
....	....	.....	....	.....	....	....	Proposed	51
....	....	.....	1909	H.	....	Whited	Projected	52
71	142	C.	"	"	....	Ralston	....	53
"	....	.....	"	"	....	....	....	54
"	....	.....	"	"	....	....	....	55
....	....	.....	....	.....	....	....	....	56

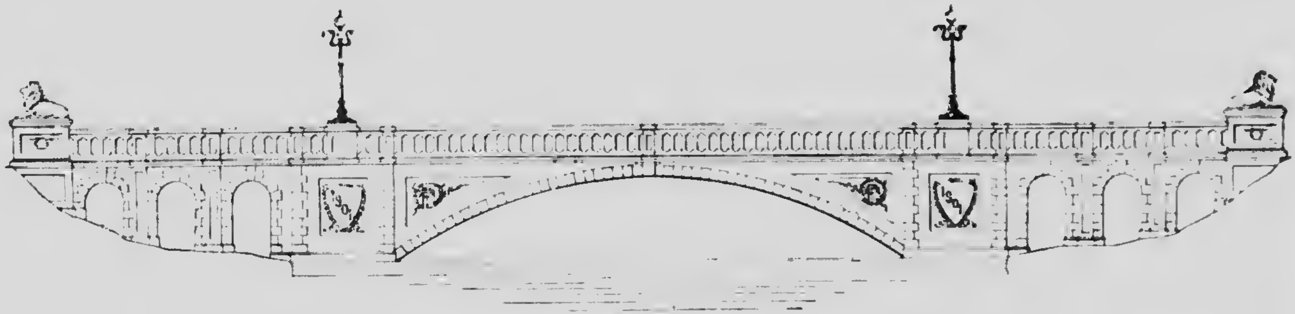


Fig. 25.

Design for a Concrete Highway Bridge.

The above shows a design for an ornamental park bridge made by the author some years ago. The arch ring and all corners and mouldings are shown of concrete blocks, while the balustrade is of artificial stone. The two piers at either side of the main arch project out past the face of the arch and are ornamented with shields bearing the date of the design. Over the piers at the roadway the balustrade is offset two feet, forming retreats from the sidewalks in which seats are provided under the electric lamps. At the ends of the bridge are figures of reclining lions mounted on ornamental concrete pedestals. On the spandrels are ornamental panels with monograms. The minor arches at the ends could be carried out either as false arches filled in with walls, or if required open for foot paths, could be built in that way.

PART II.

Reinforced Concrete Arch Bridges.

Reinforced concrete arch bridges as usually built, are a combination of arch and beam, and contain most of the properties of both types, the arch or beam properties predominating according as they have a large or small rise in proportion to their span. Flat arches act more like beams, regardless of theory.

Reinforced concrete was first considered merely a cheap substitute for stone, but its own merits are now recognized and it is used in a manner according with its properties.

A principle of architectural design demands that imitation of one material by the use of another shall not be made, and, therefore, in designing concrete bridges, there should be no effort to imitate stone, but to treat the design simply and truthfully, keeping all lines in harmony with the material used.

The extent to which concrete and reinforced concrete are now being used in preference to stone or steel, may be judged from the fact that, during the year 1908, there was at least twenty times more cement manufactured and sold than in the corresponding period, ten years previous. As methods of design and construction become generally understood and as workmen become more accustomed to handling concrete, there will be a still greater number of bridges built of this material. Long

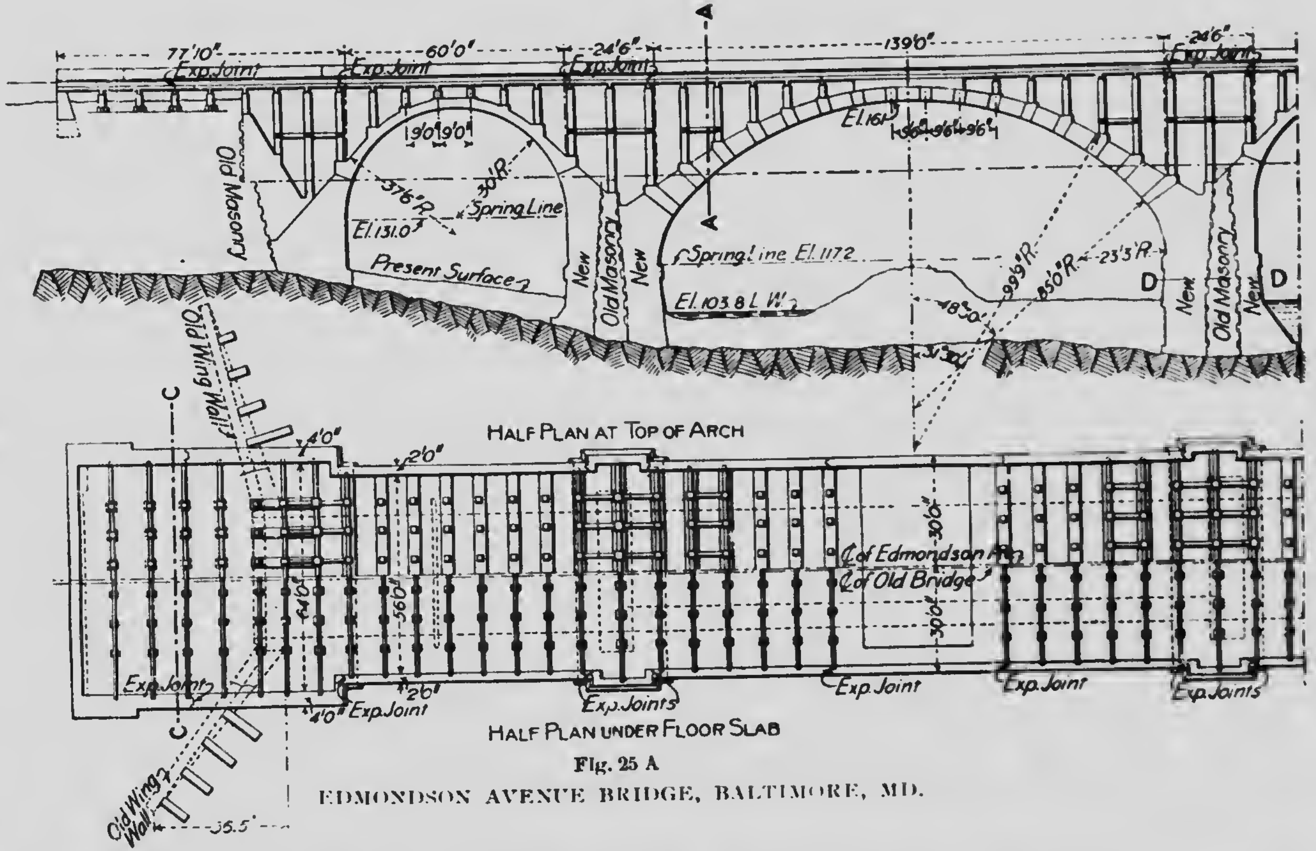


Fig. 25 A

EDMONDSON AVENUE BRIDGE, BALTIMORE, MD.

spans exceeding three to four hundred feet, will probably continue to be framed in metal, but there is reason to believe that all ordinary town and county bridges and the majority of railroad bridges will be built as permanent structures.

Reinforced concrete is a good combination of materials. Concrete has a high compressive strength, but is weak in tension. Steel rods imbedded in concrete have a high tensile strength, but are weak in compression. The steel, therefore, strengthens the concrete, and the concrete stiffens the steel, the strength of one thus supplementing the weakness of the other.

Since the beginning of the competitive practice in bridge building, many bridges have been built which are deficient in both strength and design. There is no doubt that competition is responsible for many economic features in steel bridge design and has helped to a great extent in developing economic methods. It was found about the year 1900, that steel bridges were being built entirely too light and competition was responsible for the condition. Previous to that time, the various bridge companies were accustomed to submit competitive plans, and generally the lowest bid and consequently the weakest bridge was the one accepted. From that date the policy began to change, and instead of calling for competitive designs, a competent engineer was employed to prepare plans and competitive prices were then received on his plans. The policy of employing an engineer whose principal motive was to produce an economic design, has resulted in a much better class of bridges than under the old competitive system.

Concrete bridges are now in the same stage of development as were steel bridges ten years ago. Many concrete bridges have been and are still being built, which are lacking in architectural design and some are lacking in strength. The principal reason for these defects is that reinforced concrete bridges are obliged to compete with structures of wood and steel. When towns and other municipalities realize the chances they are taking in accepting competitive designs, the method of securing an acceptable one will then be changed and a competent engineer will be employed to prepare the plans. Competitive prices will then be received on these plans, but competition will cause no reduction of the cost by weakening any parts of the bridge. At the present time, stone and concrete bridges exist, having factors of safety varying from three to one hundred and fifty, and there is, therefore, a very evident need for better and more rational methods of design.

Historical Outline.

Since the early days of stone bridge building, rods and bands of hoop iron have been used near the extrados of the arch from the piers and abutments, to or slightly beyond the point of rupture. It was found when the temporary arch centers were removed, that the arch settled at the crown and there was a tendency for the masonry joints to open at the extrados haunches. To prevent these joints from opening, iron rods have long been used. There was then no general effort made to strengthen the masonry arch, excepting as stated above. Concrete arches are reinforced with metal not only at the extrados from the piers to the points of rupture, but are also strengthened at all places where there is any possibility of tension in the arch ring. Jean Monier first began using reinforced concrete in Germany in the year 1867, by making large flower pots and urns of cement and concrete with a single layer of wire netting embedded therein. Monier was a gardener, but he foresaw a successful future for this combination, and in the next ten years he built a number of tanks, bins and other small structures of the composite material, and secured patents from the German Government on his invention. Introduction of this construction in Germany was slow, and it was not until 1894 that the Monier patents were introduced in the United States. This system of reinforced concrete contained a single layer of wire mesh with wires of the same size in both directions. Professor Melan realized the weakness of the Monier system and patented another and improved method of reinforcing arches, by which curved steel ribs were placed lengthwise of the arch and imbedded in the concrete two or three feet apart. In his first designs, curved I beams were used and are still used under his patents for small spans. For larger spans with a greater thickness of arch ring, he proposed a system of light latticed girders spaced from three to five feet apart, which system is still in use. These patents were introduced in the United States by Herr von Emperger in the year 1893, and under these patents many of America's best concrete bridges are built. In the year 1894, when American engineers began to seriously consider building and replacing old bridges in the new type, it was estimated that Europe had not less than two hundred of these bridges built mostly on the Monier system. A bridge which is believed to be the first of reinforced concrete in the United States, was built in Golden Gate Park, San Francisco, in 1889. It has a 20-foot span, 4 feet 3 inches rise, and a width of 64 feet. It is an ornamental bridge with curved wing walls built with imitation rough stone finish. A second one in the same park and of similar design was built in 1891. In 1895 a 70-foot span arch was built by Herr von Emperger, carrying a driveway over Park Avenue in Eden Park, Cincinnati. The bridge is located in the park at a place much frequented, and an effort was made to make it both strong and beautiful. The balustrade is highly ornamental and the spandrel walls are decorated with panels. The intrados of the arch is much flatter than appears necessary and certainly a greater rise would have presented a more pleasing effect.

During the first ten years after the introduction of the Melan patents in the United States, there were not more than a hundred reinforced concrete bridges built. The fact that a more general introduction of this system was not made, was probably due to the lack of more definite knowledge and data in reference to the action and behavior of this construction under live loads. European engineers were likewise embarrassed by lack of knowledge, so much so, that during the years 1890 to 1895, the Austrian Government undertook extensive experiments on full-sized concrete arches. The result of these experiments was entirely satisfactory, and complete reports of the investigations were published in many of the engineering journals of America and Europe. From the completion of these experiments in 1895 to the present time, the building of bridges in concrete and reinforced concrete has been on the increase, and there are now more than a thousand of these bridges in the United States. Previous to these experiments, no satisfactory progress was made either here or abroad.

At first it was customary to use reinforcing steel in the arch ring only, but later structures and most of those now being built have metal reinforcement throughout. Masonry bridges and buildings are still existing that have stood for many centuries, while steel bridges built less than forty years ago, have already worn or rusted out and have been replaced. Two of these bridges have already been illustrated in Part I of this book, and there are positive records of many others quite as ancient which are still in existence. Pont du Gard, an old Roman aqueduct bringing water to the city of Nimes, France, is supposed to have been built about the time of Augustus in the year 19 B. C. The Aqueduct of Vejus, consisting of a series of high arches, and the dome of the Pantheon at Rome, with a span of 140 feet, are at least 1,800 years old and all of these structures are even now in a fairly good condition. These and many others quite as old are built of coarse concrete masonry.

Several American railroad companies, after repeatedly renewing their metal bridges to support increased loads and rolling stock, have at last resorted to building their bridges in masonry, knowing that when properly built, they will remain as permanent structures for centuries.

Advantages of Reinforced Concrete.

The general advantages of masonry as compared to steel framing have already been referred to on page 1. These advantages referred particularly to plain concrete rather than to reinforced concrete bridges. It was stated there, that arch bridges of solid concrete were superior to all others, and particularly superior to arches where tension occurs in any part of the arch ring. In pointing out the commendable qualities of solid concrete, it is not intended to deny the merits of reinforced concrete. On the other hand, reinforced concrete arches have some decided advantages over solid concrete. Some of these advantages are as follows:

(1) Working units for reinforced concrete may be higher than for plain concrete.

(2) Higher units produce a thinner arch ring, and consequently less dead load and lighter abutments.

(3) Flat arches may be safely used, which would be impossible in solid concrete.

(4) Because of their lighter weight, it is practicable to build spans of much greater length.

(5) All cracks of every description can be avoided in reinforced concrete arches.

(6) They have the strength of steel with the solidity and substantial appearance of stone.

Bridges of both plain and reinforced concrete have also the following merits:—

(1) They have no noise or vibration and are not only cheaper but more durable than stone.

(2) Concrete bridges with solid decks permit the use of ordinary ties for railroad tracks, which cannot be used on steel bridges with open decks.

(3) The floors of concrete street bridges over railroad tracks are not damaged by the action of gas and fumes from locomotives, as is the framing of these bridges when built in steel.

(4) Concrete bridges require but very little skilled labor.

(5) A concrete arch bridge so designed that tension cannot occur at any time or under any condition of loading, is the most permanent bridge of all. If no tension occurs, cracks will not form to permit moisture to reach and corrode the reinforcing steel, and when

the metal is permanently protected and secure from the atmosphere and moisture, it should endure for centuries.

Deck bridges are in nearly all cases preferable to those where the travel is carried between lines of side trussing and beneath systems of overhead bracing. Such truss and bracing systems are a danger and menace to travel, particularly on crowded thoroughfares, and obstruct the space required for vehicles. Trussing and bracing are also an obstruction to observation and the clearance required through the bridge prevents the use of lateral bracing necessary to stiffen the frame. Concrete arch bridges, when deck structures, are free from the disadvantages mentioned above. Through bridges should never under any condition be used for important locations unless the underneath clearance or structural requirements positively prohibit the use of a deck bridge.

For all ordinary locations and length of span, there appears, therefore, to be no good or sufficient reason for building unsightly frame structures when more permanent and artistic ones can be made at the same cost.

Adhesion and Bond.

Rich cement concrete in which iron or steel is embedded has an adhesion thereto of from 500 to 600 pounds per square inch of exposed surface. Adhesion of concrete to metal occurs only when the metal is thoroughly embedded and the concrete has opportunity to surround and grip the bars. If a metal bar is placed simply in contact with soft concrete there will be but little adhesion. For the purpose of illustration, if steel plates are placed on edge and concrete filled in between, but not under or above them, after the concrete has hardened it will be a comparatively easy matter to loosen the concrete and break the adhesion. This weakness is due to the fact that the concrete is simply in contact with the metal but does not grip or surround it. In contrast to this condition, if a bar be thoroughly embedded and surrounded with rich concrete, it will adhere so securely to the rod, that a pull of from 500 to 600 pounds for every square inch in contact will be required to extricate the rod from its bed. In order to develop the full strength of the rod up to its elastic limit, it is necessary that the embedded length must at least equal twenty to twenty-five times the diameter of the rod. This is on the assumption of perfect adhesion between the metal and concrete. The mixture as ordinarily used, instead of fine mortar, contains more or less voids, which may be considered equal to 50% of the entire surface in contact. To allow for watersoaking, a still further reduction of 50% must be made. In ordinary work as found in actual structures, the adhesion between the concrete and metal, instead of being from 500 to 600 pounds per square inch, as for fine test samples, would, therefore, not exceed from 125 to 150 pounds per square inch. By using a factor of safety of five a working adhesive unit will not exceed from 30 to 40 pounds per square inch of surface in contact. The length, therefore, that rods must be embedded in ordinary concrete to develop their full strength up to the elastic limit is about four times twenty-five, or one hundred times the diameter of the rod.

It has been positively proven by numerous experiments that concrete adheres as securely to smooth rods as it does to rough ones. Frequent and continued shocks and vibrations tend to destroy the union between the two materials, and experiments show that continuous watersoaking from six to twelve months reduces the adhesion by about 100%. Poor workmanship in placing and ramming the concrete is also probable and for these reasons, it is desirable to use reinforcing rods that are roughened or twisted, so the bar may have a direct mechanical grip on the concrete in addition to its adhesion. When this roughening of the bars is secured without decreasing their cross-sectional area the entire area of the bar is then available for tension and no strength is lost by the expedient. Roughening the bars can, therefore, do no harm and it may be a source of extra strength. Assuming that the rough rods cost more than plain ones, the consideration in making a choice between the two, is simply whether the extra expense for rough rods is warranted by the additional strength that they may give. While watersoaking decreases the adhesion between the two materials, the upper concrete surfaces are usually waterproofed, and the probability is, that instead of weakening from watersoaking, the strength of the concrete and its adhesion to the steel will increase. The conclusion, however, is that rough rods are preferable. They cost but little more, can do no harm and may be a benefit.

Metal Reinforcement.

Reinforcing steel in concrete bridges is introduced for any or all of the following reasons:—

(1) To resist tensile stresses due to bending moments,

(2) To prevent cracks occurring from change of temperature.

(3) To form a temporary working platform at the roadway level.

There is no sufficient reason from a scientific standpoint for the use of high tension bars or rods for concrete reinforcement. After years of investigation and experiment, brittle metal was discarded for structural use and the only reason for a return to the use of high tension bars now, is a commercial one and not scientific. It is well known that in re-rolling bars to produce surface roughening, the tensile strength of the metal is increased Instead of admitting the inferior quality of their products, interested parties have endeavored to explain that this increase in tensile strength, and corresponding decrease in ductility is a benefit.

Medium steel with an elastic limit of 32,000 pounds per square inch, or soft steel with a corresponding elastic limit of 28,000 pounds, are the proper grades of metal for all ordinary concrete reinforcement. These may safely be stressed up to half their elastic limit under working loads. If, for any sufficient reason a high tension metal is desirable, then some grade of wire is preferable to bars. It is difficult, however, to secure good contact between wire mesh and concrete, for the small openings in the mesh make it difficult to tamp the two materials well together. If a mesh must be used, then a large mesh is preferable to a smaller one. In nearly all positions, whether tensile stresses are liable to occur or not, the presence of metal in concrete will add to its strength and permanence. Only in such places where there is insufficient space for its insertion, will it be a detriment. The rule generally is "when in doubt, use reinforcement".

The old Monier system of arch reinforcement, consisting of a single layer of wire mesh with wires of the same size in each direction, is evidently wrong in principle. The amount of metal required crosswise and longitudinally of the arch is not necessarily the same, for the area in each case must be suited to its need. For resisting bending moments in the arch ring, when the line of pressure falls outside of the middle third, the size of rods will depend on the magnitude of the bending moments.

It was customary at first to reinforce only the arch ring, but now all parts of reinforced concrete bridges, excepting perhaps the balustrade and other ornamental features, are provided with metal for the purpose of better uniting the whole into a solid monolith. It is particularly desirable that reinforcement be placed at all points where local loads are liable under any circumstances to produce bending or tension. Where cross spandrel walls bear upon the arch ring, these walls should not only be well anchored to the arch, but additional metal may be required beneath these concentrated loads. The best practice at the present time in reinforcing concrete arch rings is to use two complete systems, one at the extrados and the other at the intrados of the arch. Some designers prefer to reinforce the extrados only from the springs to, or a little beyond the point of rupture, omitting the metal at the extrados crown. The saving by this omission is not great and generally is not sufficient to warrant it.

At all points where light walls or sections join to heavier concrete masses, heavy reinforcement should be used. In setting and drying, concrete acts much in the same way as cast iron, and unless the light sections are well tied to the heavier ones, cracks at the junction will occur. This is illustrated where ring walls join to the abutments. If for any reason, it is impracticable to anchor the wing walls to the abutment face, it is then preferable to leave an open joint, for otherwise an irregular crack will occur, showing weakness either in the design or in the construction.

As the amount of adhesion between stee and concrete depends directly upon the amount of steel surface in contact with the concrete, it is preferable for securing the greatest bond, to use a larger number of small bars rather than a smaller number of larger ones. It is desirable also to have the cracks in the concrete as small as possible, so water will not enter the cracks and corrode the metal. Upon this feature the duration of a concrete structure depends. If water is allowed to soak into the cracks and corrode the reinforcing metal, it will then be only a few years until the strength of the member will be destroyed by rust. It is necessary, therefore, that sufficient reinforcing metal be used in order that cracks will not be excessive. Several leading designers of reinforced concrete are now specifying that tension in the concrete shall be considered, and enough metal used so the tension in the concrete will not exceed a safe unit, which is usually placed at about 50 pounds per square inch on the cross-sectional area of the concrete in tension. The object in this is to prevent cracks from forming and to exclude all moisture from the metal. This is doubtless the ideal condition, for when perfectly embedded and protected from moisture, steel is known to be indefinitely preserved. When insufficient steel is used, large cracks will form on the tension side and the bridge is then no more a permanent one than an ordinary steel bridge, or not even as permanent. When a steel bridge is exposed to moisture the steel can be examined and painted, whereas in a reinforced concrete bridge, the steel is concealed from view, cannot be inspected, and its collapse is the first warning given that the metal reinforcement has been destroyed. The best results are, therefore, secured by allowing no cracks whatever, but if cracks must form, to have these cracks so small that water cannot enter them. It is better to have a large number of very small cracks than a small number of large ones.

A requirement upon which the strength of reinforced concrete directly depends, is the amount of contact between the two composing materials. Every effort should be made to have this contact as perfect and complete as possible. In deciding upon a working unit for adhesion of concrete to steel, it is customary to consider that imperfect workmanship in ordinary structures will cause only about one-half of the exposed metal surface to be actually gripped by the cement. If a higher degree of workmanship be secured, then the strength of the structure will be increased accordingly. It is considered that watersoaking still further decreases the adhesion by another 100%. Therefore, if perfect adhesion on rich samples between the two materials is from 500 to 600 pounds per square inch, the ultimate adhesion in actual structures cannot be taken greater than from 125 to 150 pounds per square inch. To develop the full tensile strength of bars embedded in concrete, it is easy, therefore, to compute the length that these bars must be embedded. Using an ultimate adhesive unit for ordinary structures of 150 pounds per square inch, one inch square bars would be gripped to the extent of 600 pounds per lineal inch of bar. Therefore, to secure the full elastic strength of the bar up to 32,000, the rod must be embedded a number of inches, equal to 32,000 divided by 6,000, or 53 inches. Where arch rings join to piers and abutments, it is customary to run the reinforcing steel well into the piers to develop the full strength of the metal.

Experiments show that adhesion to steel is much greater before the steel is painted than afterward. A slight coating of rust has been found to add to, rather than to detract from, the adhesive strength. Loose scales or flakes of rust must not be permitted, but a slight rusting is no disadvantage. Experiments have been made on rusted steel imbedded in rich cement, and after a period of several months when the steel was removed and the cement broken away, it was found that the steel appeared clean and free from even the slight rusting that existed when it was first imbedded.

Light reinforcing frames are frequently used in the spandrels of reinforced concrete bridges, not only to strengthen the concrete, but also to provide a temporary working platform at the roadway level. This plan is illustrated by the Illinois Central Railroad Company's bridge over Big Muddy River near Grand Tower, Illinois. Bridges built by Herr Wunsch in Germany were mostly of this type. The metal in such cases must have sufficient strength to act as compressive members. In the Big Muddy River bridge, the engineer used old rails for the spandrel frames, and when completed, these were encased by the concrete spandrel columns.

Reinforcing Systems.

The principal reason for the existence of the many patented systems for concrete reinforcement is the patent royalty secured therefrom. There are a few essential requirements, and where these are fulfilled, the reinforcement is satisfactory. Chief among these requirements are:—

(1) The metal shall be rough or have a mechanical union with the concrete,

(2) Reinforced beams shall have stirrups for transmitting shear components from the main tension members into the web of the beam.

In connection with the latter requirement, it is preferable that the stirrups be rigidly connected to the tension member, in order to secure a positive transmittal of the shear components.

The various reinforcing systems may be roughly classified under two headings.

(1) Slab Reinforcement,

(2) Beam Reinforcement.

Under the first heading are included the various kinds of expanded metal. Light rods are suitable for slabs, as are also twisted bars and plain flats with rivet heads thereon. For beam reinforcement, the opportunity for patented systems is greater, and a large number are now on the market. Among these may be mentioned Twisted rods, Corrugated bars, Diamond bars, Thacher bars, Cup bars, Twisted Lug bars, etc. All of these are rods and bars without provision for stirrup connections. In addition to these, there is quite a variety of patented bars on the market, either in the form of truss frames or with stirrup connections. In this latter class may be placed the Kahn bar, the Cummings Girder Frame, the Unit Reinforcing Frame, the Luten Truss, the Monolith Frame, the General Fireproofing Company's Girder Frame and others.

For slab reinforcement, a coarse wire with its high tensile strength and corresponding high elastic limit, is economical. It does not have the disadvantage of high tension bars, for while bars are brittle and lack ductility, wire is elastic and has always been and probably will continue to be a desirable tensile metal. It bends easily, will not crack in handling and gives a large external contact area in proportion to its section. Certain kinds of wire mesh have the principal strands in one direction, united by a lighter weave at right angles to them. This type of wire mesh is made with the principal wires in various sizes and is well suited for reinforcing bridge floors. Where floor panels are square and floor beams in both directions, it is then economical to use a wire mesh with wires of the same size in each direction. Most of the various expanded metal systems, while they have a lower tensile strength, have sufficient stiffness to support their own weight during construction, and are rougher and have a greater mechanical bond than wire. An excellent example, showing the various methods of reinforcement for concrete bridges is a ribbed design, for Grand Avenue Viaduct in Milwaukee, shown in Figure 27 and more fully described in the Engineering News, February 14, 1907.

As the shearing stress in curved arch slabs is quite small, there is but little need for metal in the web. The Melan system has continuous lines of double angle bars at the extrados and the intrados of the arch, connected by light lattice work, and these are manufactured complete in the structural shop and shipped to the bridge site ready for erection. These frames are blocked up vertically on the arch centers from three to five feet apart crosswise of the bridge, and they are connected at intervals with bars or frames which take the place of expansion rods. These shop-riveted frames considerably simplify the work of field erection and avoid the complexity and confusion which is liable to occur when a large number of disconnected small bars are used, but much of the web material and the shop labor of riveting is unnecessary for resisting stresses. In some of the designs, Mr. Thacher has used plain flat bars adjacent to the extrados and intrados placed about two feet apart. These bars are roughened by having rivets driven at frequent intervals, rivet heads projecting to form the mechanical bond.

The Kahn bar with light connected diagonals, is well suited for arch reinforcement, as the web members securely tie the reinforcing bar into the body of the arch, but any system of rough bars or rods which are completely imbedded in and surrounded with concrete and which have the necessary cross-sectional area, regardless of whether they have a web connection or not, are suitable for concrete arch reinforcement.

Concrete Composition.

It is customary with some engineers to specify several degrees of richness for the concrete in a single bridge. Mixtures varying from one part of cement with two of sand and three of gravel and stone, varying through several different grades to corresponding mixtures of 1, 5 and 10, are all specified in the same bridge, the richer concrete for the spandrel or arch ring and the poorer for the abutment foundation. The policy is generally unwarranted. Anyone who has observed the ordinary methods used, and the way in which concrete goes into structures, should realize that exact methods which can reasonably be applied to single truss systems, and specifications for various grades of metal, are not appropriate or suitable for use in the design of concrete bridges. Generally it is quite sufficient to specify only one or two kinds of concrete mixtures, the richer for the superstructure and the poorer grade, if another, for the foundation. Examination of test records on the strength of concrete mixtures, varying from 1, 2 and 3 to 1, 3 and 6, does not show enough variance in strength to warrant a change of working unit. Therefore, instead of several mixtures with only slight variations, it is better to specify a single mixture. It is frequently cheaper for the contractor to put in all mixtures of the richer grade, than to make numerous changes. A more important consideration than the quality of the concrete, is the securing of contact between the concrete and the metal. In proportion as this is well or poorly done, the permanency of the bridge depends.

Loads.

The principal loads on masonry arches are the dead weight of the arch itself and the superimposed material above it. It is better to consider only vertical loads as acting on ordinary earth filled flat arches, for the conjugate horizontal forces are small and may be neglected. The amount of horizontal thrust from earth filling is indefinite, for the earth will recede more or less horizontally, allowing the arch to settle at the crown. Therefore, neglecting these horizontal earth pressures is an assumption on the side of safety. It must be noted, however, that the above statements apply only to flat arches when the proportion of rise to span is small. When the arch has a greater rise equal to or approaching half the span, the conditions are greatly changed, for below the point of rupture the horizontal thrusts are so great that solid masonry filling is required.

The side retaining walls of earth filled arches frequently act as arch ribs and carry a large proportion of the weight of the earth filling. The distribution of lo[illegible] earth filled arches is uncertain and the pro[illegible]on borne separately by the arch ring and the side walls acting as arch ribs, is uncertain. To avoid this uncertainty some engineers are now designing the side retaining walls with one or more expansion joints in each wall, to prevent these side walls from having any arch action. The entire dead weight and imposed loads must then be supported by the arch ring. There is no doubt that the side retaining walls are capable of supporting large loads as arch ribs, but it is important to know definitely which members of a structure are in action. Any type of construction in which the action of stresses is indefinite, is in many ways undesirable. The condition is similar to that of multiple systems for metal truss bridges. Multiple systems are no doubt economical, but it is usually impossible to know what proportion of the load is carried by each system. This lack of definite knowledge is often the cause of failure, and it is desirable in the design of masonry as well as steel structures to have the condition of loads as nearly fixed as possible. For this reason many arches are designed with cross-spandrel walls eliminating entirely any possibility of external horizontal pressure on the arch ring.

The weight of earth filling varies according to its nature from 100 to 120 pounds per cubic foot, and the weight of concrete from 130 to 160 pounds per cubic foot, depending upon the density of the stone. Other loads such as that of pavement, railing, water pipes, etc., must be taken according to their actual weights. Approximate general rules for moving live loads are as follows:—

(a) Light carriage travel is equivalent to 100 pounds per square foot.

(b) Heavy carriage travel is equivalent to 200 pounds per square foot.

(c) Electric railroad travel is equivalent to 500 pounds per square foot.

(d) Steam railroad travel is equivalent to 1,000 pounds per square foot.

There is usually sufficient earth filling above the arch ring to distribute any concentrated loads, and particularly for railroad bridges where the ties and rails assist in spreading the load out over a greater area. It is usually safe, therefore, to consider all live loads as uniformly distributed. These rules apply only to earth filled arches, for the loads on arch rings which have open cross-spandrel chambers or arcades occur beneath the spandrel walls, and are plainly concentrated loads. The system of loads should be carefully considered for each case, and the designer should be satisfied in reference to the safety of his assumptions, for local loads might easily occur which would require special provision.

The bending moments on arch rings for moving loads are a maximum when the uniform live load covers from two-fifths to three-fifths of the span, but it is usually considered as covering one-half of the span.

The weight of loaded electric cars varies from 1,000 to 3,000 pounds per lineal foot of track, one-half of this load being borne on each rail. The weight of ordinary light electric cars fully loaded will not exceed 1,000 pounds per lineal foot, but it is now customary to proportion the better class of street railroad bridges to carry loaded freight cars which it is often convenient to switch over electric railroad tracks. The additional cost of proportioning bridges for this extra load is comparatively small. The electric railroad companies themselves so often require large quantities of coal delivered at their power plants, that they are usually willing to pay the extra cost of a bridge over which their tracks run, in order to have coal cars delivered directly to their plants.

Temperature stresses in masonry arch rings are frequently as large or even larger than the bending stresses from partial live loads. Masonry bridges are not subject to so great a range of temperature as metal bridges, for masonry is a poorer conductor of heat than metal and the intrados of an arch is not exposed to the direct rays of the sun, neither is the extrados or any part of the arch ring excepting the ends appearing at the spandrel. For this reason it is safe to assume a maximum temperature range of from 50 to 60 degrees between the highest and the lowest temperatures of the arch material. Temperature stresses may be entirely eliminated by the use of hinges at the springs and crown, but the practice with American engineers is to spend more money in making the foundations secure, and thereby avoid the need of hinges. The money that would be spent on building hinges is put into the foundations.

As temperature rises, the arch expands and rises at the crown, but when the temperature falls, the arch contracts and it must necessarily fall at the crown. This rise and fall of the arch, due to atmospheric conditions, is the cause of temperature stresses.

Addition must be made to the live loads to provide for the effect of impact. The amount of this impact is determined from the formula

$$\text{Impact load} = \frac{L^2}{L+D}$$

where L is the live load and D the total dead load per horizontal square foot on the arch.

Units—Ultimate and Working.

Permissible working units for plain concrete arches have already been given in Part I. Reinforced concrete arches may have higher values, owing partly to the fact that the reinforcing steel will resist some compression and also because reinforced masonry is a more secure monolith. Concrete has an ultimate compressive stress of from 2,000 to 2,800 pounds per square inch. A working unit for plain concrete in compression was given at 400 pounds per square inch; for reinforced concrete it is safe to assume 500 pounds per square inch for combined, direct and live load bending stresses. For combined, direct, bending and temperature stresses, it is safe to assume a working unit of from 600 to 700 pounds per square inch.

American engineers generally are accustomed to using much lower working units in concrete than are used by European engineers. There is probably sufficient reason for these lower units, for the quality of work done in America is not so fine as is produced in France and Germany. In designing the Grand Avenue bridge, now being built in Milwaukee, the concrete working units used were 500 pounds per square inch, and 600 pounds including temperature stresses. Perfect adhesion of rich concrete to steel varies from 500 to 600 pounds per square inch. It has already been shown under the heading "Adhesion", that 30 pounds per square inch of exposed surface is a safe and usual working adhesive unit.

The ultimate shearing strength of concrete is 400 pounds per square inch and a safe working unit is 50 pounds per square inch.

A safe working stress for steel in compression is one-half its elastic strength, or 14,000 pounds per square inch for soft steel and 16,000 pounds per square inch for medium steel. The ultimate tensile strength of good concrete is 200 pounds per square inch, and for the purpose of preventing cracks forming on the tension side of beams or members subject to bending, provision may be made for tension in the concrete, not exceeding 50 pounds per square inch. The object in this is plainly to prevent cracks from forming, which would admit water or moisture and expose the metal to the danger of corrosion. The provision is a safe one, but as the modulus of elasticity for steel is not more than twenty times greater than for concrete, the steel in the tension side of the beam would then be stressed to only twenty times the tension allowed on the concrete, or 20 times 50, which is 1,000 pounds per square inch, instead of 16,000 pounds per square inch.

Some engineers propose a method of proportioning concrete sections by the use of ultimate units applied to three or four times the actual loads. This method is inconsistent. Bridge engineers have long been accustomed to using safe working units for structures which are only a fraction of the ultimate values, using different working values where necessary for the dead and the live loads. The same system used in designing steel bridges should be applied also to concrete bridges, and all sections proportioned according to safe working units after addition has been made to the live loads for impact. It is evident that when a tension unit of 16,000 pounds per square inch is used for dead load stresses and a corresponding tension unit of only 8,000 pounds for live load stresses, that provision is made by these varying units for impact amounting to 100%. It is simpler and more accurate to follow the method of the more recent steel bridge specifications and apply impact addition, using the same unit stresses for both dead and live loads.

Theory of Arches.

The exact theory of arches is very complex. Several comprehensive books have been written on the subject and the theory will be referred to only briefly here. For a full discussion and explanation of the various theories, the reader is referred to any of the mathematical treatises on the elastic arch. The subject has been treated generally by two methods, the analytical and the graphical. Most, if not all writers and designers using the analytical method follow the theory as developed and explained by Professor Charles E. Green in his book entitled "Trusses and Arches", while exponents of the graphical method use the one outlined by Professor William Cain in his "Theory of Elastic Arches".

The complexity of the subject is responsible to a great extent for the lack of a more general introduction of reinforced concrete arches. They are really a combination of arch and beam. Plain concrete arches have already been discussed in Part I, and reinforced concrete beams are considered in Part III. The reinforced concrete arch is proportioned to act both in direct compression and as a beam, to resist bending stresses from uneven loading on the arch ring.

The arch is distinguished from the beam by having horizontal or inclined thrusts at the springs, in addition to the vertical reaction of the abutments. Arches are classified under three headings according as they are fixed or hinged.

(1) Arches with no hinges,

(2) Arches with two hinges at the springs.

(3) Arches with hinges at the two springs and hinge at the crown.

They are classified also under two general heads into (1) Slab Arches and (2) Ribbed Arches.

The stress conditions in the arch vary greatly, depending upon the presence or absence of hinges. The space allotted to this book will not permit of more than a very brief review of the principles involved. The principal part of the computation for a reinforced concrete arch consists in finding

(a) the horizontal thrust.

(b) the end reactions and

(c) the bending moments at various points in the span.

After these have been found, it is then a comparatively easy matter to proportion the metal and concrete to resist the stresses. The method consists in drawing the correct line of pressure for the given arch and loading, and determining its proper position in the arch ring. When this has been done, it is easy to find the bending moment at any point of the arch.

Most of the uncertainties of masonry arches which have been enumerated in Part I apply equally to reinforced arches. The elastic theory applies not only to arches in which bending moments are resisted by the arch ring, but may be used also for arches of solid concrete with no tension in any part of the arch ring or where the line of pressure lies in all cases within the middle third of its depth. The theory is applicable both for two-hinged and for fixed end arches.

rches with fixed ends have five unknown quantities.

(a) equal horizontal thrusts at either end.

(b) two vertical end reactions and

(c) two bending moments at the springs.

Where there are no hinges in the arch, the reactions are not transferred to the abutments in accordance with the law of the lever. Since there are five unknown quantities, there must in addition to the two equations of equilibrium, $\Sigma x = 0$ and $\Sigma y = 0$ be three more equations found. These are determined from the conditions of equilibrium for fixed end arches, which are as follows:—

(1) The angle of inclination that the springs make with each other must not change.

(2) The relative elevation of the two end abutments must not change, and

(3) The length of span must not change.

These are mathematically expressed by the formulae:—

$$\Sigma_B^A n M = 0, \Sigma_B^A n Mx = 0, \Sigma_B^A n My = 0.$$

In the above formulae, M is the general value of the bending moments, n the length of a short portion of the arch ring, and x and y, the horizontal and vertical coordinates to the center of n, measured from the origin at the springs A or B. Fixed end arches have high temperature stresses, two to four times greater than for two-hinged arches.

The abutment reactions for arches with either two or three hinges, follow the law of the lever, which greatly simplifies the mathematical calculations.

Two-hinged arches have only three sets of unknown forces.

(1) The horizontal reaction and

(2) The two vertical end reactions.

As there are hinges at the end and a condition of continuity cannot exist there, the two additional unknown quantities, the two unknown bending moments at the springs do not now exist. This is the theoretical assumption, but it is not exact, for even with pin bearings at the end, there is a large amount of friction on the pins and the bending moments will not entirely disappear. The assumption, however, for two-hinged arches is that there are only three sets of unknown forces. Therefore, in addition to the two usual equations of equilibrium $\Sigma x = 0$ and $\Sigma y = 0$, there is only one other equation required, and this can be found from the condition that the length of span must not change. The span length should not change or no sliding of either abutment should occur in order that the arch ring between the springs shall remain intact.

The third equation required for the solution of the two-hinged arch is, therefore, expressed as follows:—

$$\Sigma_B^A n\,My = 0$$

Hinged arches are not frequently built in America, but some designers for the purpose of simplifying calculations, consider the arch ring as hinged at the springs.

The condition of stress in three-hinged arches is definite, for the moments both at the springs and crown are zero, and the position of the line of pressure is, therefore, fixed at these three points. The equations of equilibrium for three-hinged arches are, therefore:—

$$\Sigma x = 0,\ \Sigma y = 0,\ \Sigma M = 0,$$

The thrusts, bending moments and shears may be found most easily by Professor Cain's graphical method, after which the section may be most easily proportioned analytically. It has already been stated that the graphical method consists in drawing the correct line of pressure for the given arch and loading and determining its proper position in the arch ring.

The following method is used for determining the form of arch and the thickness of the arch ring for uniform loading. It avoids the usual trial method given in Part I for solid concrete arches. The position of the springs must first be assumed as well as an approximate crown thickness and the depth of earth filling above it. The remaining height from spring to crown intrados will be the rise of the arch. The method depends upon the equation, M = HT, where

M is the bending moment,

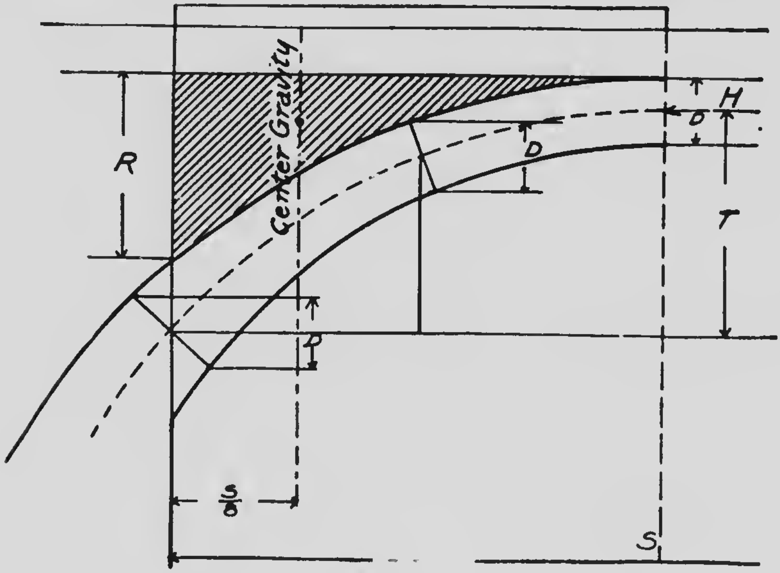
H the crown thrust or pole distance of the force polygon, and

T the vertical ordinate to the pressure curve at the point where the moment is taken.

The bending moment at the center is the same as for a simple beam and dividing this moment by the arch rise gives the crown thrust or pole distance H. The bending moment at any other point of the arch is equal to the pole distance H multiplied by the vertical intercept at that point in the funicular polygon. The moments are, therefore, computed for as many points as desired and dividing these moments by the pole distance H, which has already been found, gives the required ordinates T to the funicular polygon, which is the line of pressure for the full assumed loading. The pressure curve is then plotted from the ordinates found and this will give a curve for uniform loads.

The height T referred to above is the distance to the line of pressure measured from a horizontal line through the point of rupture, which is not necessarily at the abutment face. The correct crown thrust cannot be obtained by using a distance T to any point below the point of rupture. When the point of rupture falls within the abutment face, the span length must be taken as the distance between the points of rupture, and not the clear distance between abutments.

For full dead and live loads, the line of pressure should wherever possible, lie within the middle third of the arch ring, and reinforcement used only for resisting bending stresses due to partial live loads. In Figure 26, the weight of the arch ring may be assumed at its mean thickness at the quarter point, and the arch ring weight assumed approximately as a uniform load. The weight of earth filling, pavement and other material between the



Fi[illegible]

extrados and roadway level, as well as the uniform live load, is also uniform, and the center bending moment for these uniform loads is expressed by the equation:

$$M = \frac{W S^2}{8}$$

For a parabolic arch, the spandrel area shown hatched in Figure 26 is equal to $\frac{S\,R}{6}$. The center of gravity of this area is equal to one-eighth of the span length from the abutment face. Therefore, the bending moment at the center from spandrel filling is equal to $\frac{25\ R\ S^2}{12}$. The total moment is, therefore, equal to the sum of moments from uniform loads and from the spandrel filling. Dividing the center moment by the rise gives the crown thrust or pole distance H for the force polygon. This is a very convenient analytical method for determining the correct arch form for any system or arrangement of loads. A combination of the analytical with the graphical method will simplify computation, as some results, like finding the crown thrust, may be determined most easily by the analytical process.

In practice, it is usually sufficient to find the sum of all moments and thrusts at three different points—the center, the quarter points and springs.

The thickness of arch ring at other points below the crown must be such that the vertical heights D, shall not be less than at the crown.

The bending moment at any point of the arch ring from partial loading is equal to the pole distance or horizontal thrust at the center, multiplied by the vertical intercept between the neutral plane and the line of pressure at the point considered. The correct position of the line of pressure for partial loading will already have been drawn upon the arch ring, and the vertical intercept may be scaled and will be positive or negative according as the pressure curve lies above or below the neutral axis of the arch.

The determination of the thrusts and moments may be simplified by considering the arch as a parabola. This is approximately true when the rise is small in comparison to the span.

The stability of an arch is secured when it will resist the stresses resulting from thrust and bending from any system of loads, when the line of pressure is drawn in such a position as to produce the least possible bending moment, or when the line of pressure is drawn the nearest possible to the center line of the arch.

General Design.

The introduction of bridges of combined metal and concrete has thrown open a wide field for improvement in design. So long as it was necessary to build bridges of stone, the art showed no great improvement over the work of the ancients. In recent years, however, the increased production of cement with its decreased cost, as well as the invention of improved stone-crushing machinery and appliances for mixing concrete, have tended to make larger structures possible, even in solid masonry. The greatest progress in the art has been made since the completion of the Austrian experiments in 1895. Reinforced concrete has made it possible to discard old, conventional forms and to introduce new and lighter types of bridges supported by arch ribs, carrying open spandrel framing to support the roadway. The enormous reduction in the dead weight of the superstructure has caused a proportionately large saving in the foundations. A large number of improved methods of design have already been tried successfully and there is prospect of additional progress in the future. With the new material designers are following to some extent the outlines used for metal bridges, so there are now numerous examples of bridges built in concrete-steel, not only in the form of light ribbed arches, but also as solid and ribbed cantilevers, girders, trusses, etc. The new material is, in fact, being used according as its own properties will permit.

The general subject of arch bridge design is divided into four parts,

(1) The parapet or deck,

(2) The spandrels,

(3) The arch ring and

(4) Temporary arch centers.

In beginning the general design, the final object should at all times be kept in view. The first and chief object in building all bridges is to construct and support a platform at the proper elevation, of sufficient capacity to safely and securely conduct travel over certain openings. A second object which is too often neglected, is the desirability of making the bridge pleasing in appearance, in harmony with its surroundings and a credit to its builders.

When once started, the design should be continued in logical sequence. The width of bridge and the kind of pavement required, should be selected with the necessary filling beneath the pavement to support the roadway or the railroad ties. After deciding upon the kind of deck required, the most economical method of supporting this deck must be determined. It may be carried on solid earth filling or on a series of walls or columns, and these may be continued to the ground in the form of a trestle, provided the height from deck to ground is small. If the height be great, these walls or columns may then be supported on other ribs or frames, such as arches or trusses, and the loads from these may in turn be transmitted to the ground through walls or piers of the most economical form. There is no good reason why the spandrel columns of a concrete bridge cannot be supported in other ways, excepting on slab or ribbed arches. Trussed frames or girders are possible forms, though they would not be as pleasing in appearance as a continuous arch. It is possible that arches with double ribs or drums separated by systems of framing may be used, following the outline of a double-braced metal arch. If the design is developed in successive steps, beginning with the roadway platform, and transmitting the loads continuously in the most economical manner through various kinds of framing into the foundations, the result will be both scientific in construction and satisfying to the engineer. It is a deplorable fact that the design of many bridges is begun by first locating the foundations and developing the design upward from the ground, instead of from the deck downward. This one error accounts for the absence of economy in many structures.

The old empirical rules for masonry arches, which required more masonry in the abutments than in the arch, are unscientific and useless for reinforced concrete. All through bridges are objectionable. They are a menace and an obstruction to travel, are lacking in lateral stiffness, and the trusses or framing interfere with the river view, which is generally and should always be an interesting feature of a river bridge.

If a bridge has several spans and one span has movable bascule leaves or other kind of draw, the outline of the draw span should conform and harmonize with the rest of the bridge and its presence should be indicated by piers or towers at either side of the opening. The underneath outline for double bascule leaves in a single span may easily be made in the form of a continuous arch, corresponding to the intrados curves of other spans in the bridge.

Unsymmetrical arch spans may be used at the ends of viaducts crossing deep ravines. They cause a large saving in the abutments by permitting higher springs at the abutments than at the piers. The half shore span adjoining the pier may be made with intrados curve to correspond with the next adjoining span, thus producing symmetry about the pier center. As the end arch span lacks symmetry in the arch, it is necessary for appearance, that the design shall be symmetrical about the pier.

The Kissinger Bridge, twelve miles southeast from Wabash, Indiana, is of unusual design. It has a 16-foot concrete roadway slab balanced on a single center concrete web 12 inches in thickness, supported on a segmental concrete arch, 8 feet in width. It is a single span highway bridge, with 60-foot opening and was built in 1907.

All town or city bridges should have open chambers beneath the floor for pipes and wires. They may either have removable iron covers, or be paved over, with manholes or entrances provided at either end.

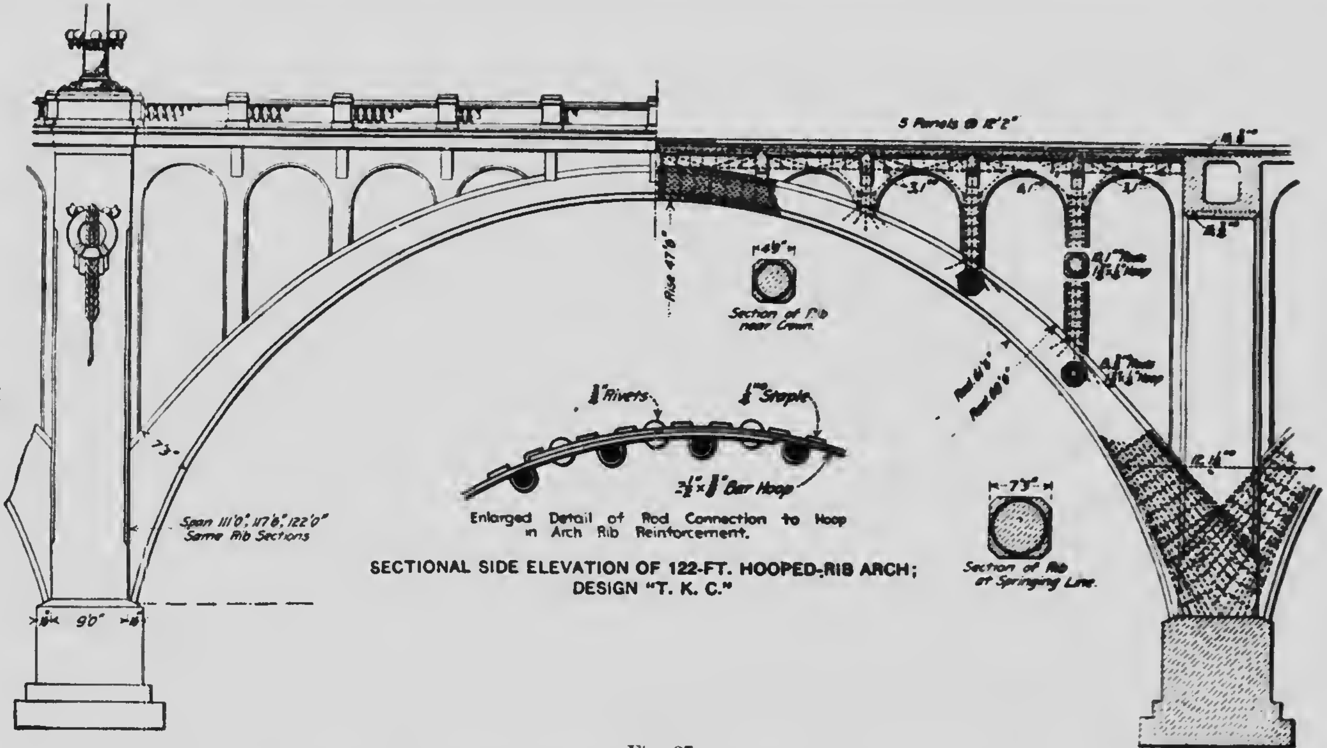
Hinged Arches.

There is a difference of opinion with regard to the use of hinged or fixed arches for masonry bridges. Hinges, by which is meant the insertion of heavy stone or metal blocks at or near the center line of the arch, remove one of the principal uncertainties of arch construction, by fixing the position of the line of pressure at the springs. The presence of a hinge at the crown tends to considerably reduce the rigidity and increase deflection, and is not always to be recommended. Hinges may be introduced at the springs in such a manner as to insure absolutely within small limits the position of the line of pressure there. Fixed ends tend to greatly increase the amount of temperature stresses and they have no advantages over hinged ends. After the centers are removed and the arch ring has come to or nearly to its final position, the open joints at the hinges should then be filled solid with cement, so the entire cross-section at the hinges will be available for full loading. The presence of hinges or the assumption of their presence at the springs, simplifies the computations and removes one of the chief uncertainties of concrete arch design. The American practice has been to avoid any extra expenditure on hinges, but to put it into the foundations, insuring their stability against movement. There are numerous unfortunate cases where the foundations have been insufficient. Several spans of a bridge over the Illinois River at Peoria were recently destroyed, owing to the undermining of foundations. Hinges are desirable chiefly where it is known that the soil is yielding and the abutments are liable to recede laterally, allowing the arch to fall at the crown, and cause unsightly and possibly dangerous cracks. A method employed by certain German engineers is to place hinges at the point of rupture. This was done in a bridge built at Kempten, Bavaria, over the Iller River, and described in the Engineering News, May 2, 1907.

Ribbed Arches.

The principal economy in reinforced concrete bridges comes from the use of ribbed arches. Most of the surplus material, both in the structure itself, and in the spandrel filling, may then be eliminated, and as weight of superstructure decreases, the cost of foundations decreases in proportion. The use of ribs instead of slabs, is a more scientific type of construction and allows the strongest supporting members to be placed exactly where required. Ribbed concrete arches are purely a product of this new material and are possible in concrete only when properly reinforced with metal. Concrete ribbed bridges are built mostly in the form of arches, though other forms, as cantilevers, have also been used with varying degrees of success. Many bridges designed as arches have cantilever action also, or when the rise is small in proportion to the span, the stresses are chiefly the result of bending, and regardless of theory the span acts then more as a beam than as an arch. The uncertainty in reference to cantilever or beam action of arches can be removed by building an open vertical joint between the arches over the piers, the presence of which will positively prevent any cantilever action. While such a joint removes a serious uncertainty of design, it is very doubtful whether or not this expedient is desirable, for the cantilever action frequently adds as much strength to the bridge as does the arch and when properly designed and built to resist both sets of stresses, the presence of canti[illegible]r action adds greatly to its strength and permanence.

The Walnut Lane bridge at Philadelphia, and the Rocky River and Piney Creek bridges now under construction, illustrate to some extent the saving which may be accomplished by the use of ribbed



SECTIONAL SIDE ELEVATION OF 122-FT. HOOPED-RIB ARCH;
DESIGN "T. K. C."

Fig. 27.

GRAND AVENUE BRIDGE DESIGN, MILWAUKEE, WISCONSIN in place of slab arches, and yet all of these three bridges are only partially ribbed. They each consist of a pair of twin arch rings separated by a distance of from 10 to 20 feet, which space between the rings is spanned by simple floor construction. The saving in the arch ring by this expedient is from 25% to 30% of the cost of the ring, which saving would be still further increased by using entire ribbed designs. The Luxemburg stone arch bridge in Germany with a span of 275 feet and completed in the year 1903, is of the same type. An unusual example of ribbed arch design prepared by Mr. Turner of Minneapolis is shown in Figure 27. It is one of several designs submitted for the Grand Avenue viaduct in Milwaukee. The main compression members are octagonal and are hooped.

The use of ribs instead of slabs makes it possible to place members of the proper strength where required, as for example under lines of street railway track, where heavier ribs are usually required than under other parts of the roadway. Sidewalks may be bracketed from the outer ribs and properly tied into or across the floor and the whole design executed in a more scientific and economical manner.

The principal objection to the use of ribs is the extra cost of the required wooden forms, which of course is much greater than for plain arched slabs. Notwithstanding this objection, important concrete arches of the future will possibly be built with ribs, particularly when the proportion of the rise to span is large.

Intrados Form.

A low flat opening is the best form for the passage of water. A rectangular opening for culverts with the height greater than the width will cost less than when the width is the greater of the two dimensions. This is clearly shown by the culvert design given in Tables VII, VIII, IX and X of Part IV, but the decreased cost is secured at the expense of efficiency.

Intrados [illegible]ms should be as nearly as possible exact mathematical curves, but if these cannot be secured, they should then approach so nearly to the exact curves that the lack of regularity may not be detected by the eye. Three and five centered flat arches as approximation to the ellipse, are usually unsatisfactory because the breaks in the curve can be det[illegible] If a flat ellipse is desired, the curve shoul[illegible] exact ellipse and not an approximation. [illegible] which are too flat are not artistic. A rise of [illegible] one-fourth to one-sixth of the span will give [illegible]er appearance. Natural conditions or grade lines will frequently prevent even this amount of rise, and it must then be determined by stability requirements, which should not be less than from one-eighth to one-tenth of the span. The steel arches of the bridge across the Mississippi River at St. Louis have a rise of one-eleventh of the span and there is at Steyr, Austria, a reinf[illegible] concrete bridge of 139-foot span, the rise of [illegible] is only one-sixteenth of the opening.

Earth filling in the haunches tends to m[illegible] line of pressure approach the form of an ellipse, while the uniform loads including the weight of arch ring, filling above the extrados, pavement and full live load tends to depress the line of pressure to the approximate form of a parabola. The combined effect of these two tendencies is to produce a curve approximating a circular segment. The resulting curve will lie nearer to the ellipse or to the parabola, according as the effect of haunch filling or uniform load predominates.

The trial method of determining the intrados curve is no longer necessary, for a direct method has been given. Under the head of "Theory of Arches", a method has been explained for determining the amount of crown thrust by dividing the center bending moment by the rise. The simple beam moment at any other point is equal to the crown thrust or pole distance H multiplied by the vertical ordinate in the funicular polygon, which is the intercept between the closing line and the pressure curve. Therefore, dividing this bending moment by the crown thrust or pole distance, gives the proper ordinate or rise for the center line of the arch at the point considered. This method makes it possible, after having first assumed the approximate form, to determine directly without trial, the exact intrados curve for uniform loading. When the exact linear arch has been found, the bridge will present a better appearance if a regular curve be drawn, such as a segment or ellipse, even though the use of a regular curve makes the arch somewhat thicker in certain parts than is required. After having drawn the correct linear arch, the thickness of the ring for uniform loads should be proportioned directly to the thrusts.

The computations are much simplified if the curve be considered a parabola, and this assumption is approximately true when the rise is small in comparison with the span. Parabolic and segmental arches require little metal reinforcing, while elliptical and other flat arches may require a greater amount.

Some designers prefer to use an intrados curve, lying half way between a segment and an ellipse and found by bisecting the vertical intercepts between these two latter curves. Mr. Burr's Potomac Memorial Design No. 3 has an elliptical intrados, with a rise of one-fourth the span, and a segmental extrados.

Spandrels.

The principles already given for the spandrel design of solid concrete arches, apply also to arches of reinforced concrete. If side spandrel walls are used, provision should be made for expansion or these side walls will crack. A dovetailed expansion joint is the most satisfactory one, for sufficient space can be allowed in it for expansion, while the two wall sections are held securely together. If an expansion joint is not provided, an open crack is liable to develop between the spandrel wall and the arch, and if an effort be made to prevent such an opening by clamping the spandrel with metal ties to the arch ring. the stress in the arch then becomes indeterminate, as a portion of the load will be carried by the arch action of the spandrel wall.

Joints in continuous walls should occur at intervals not exceeding 20 to 25 feet. It has been found by experience that temperature cracks occur in solid walls at about these intervals and if artificial joints be formed, the developing of unsightly and irregular cracks will be avoided.

All exposed flat concrete surfaces should be paneled to avoid monotony. It is difficult to build plain surfaces perfectly straight or plumb, and the use of panels with pilasters and belt courses assists to conceal irregularities and imperfections in flat surfaces, that otherwise might be quite apparent.

Open spandrel arches in the haunches produce a light and artistic appearance, but they are not practicable for flat arches.

Spandrel walls may be built either as curtains to obscure the open chamber framing, or as retaining walls to support earth filling. As retaining walls they may be built either as solid gravity walls, or as lighter reinforced walls with counterforts. In any case it is better that the centers be removed and the arch allowed to settle before building the spandrel walls.

Piers and Abutments

On the stability of the foundations the strength of the whole superstructure depends. The piers and abutments include all of the structure from the ground up to the point of rupture. The total angle included between normals to the points of rupture, never exceeds 120 degrees and is usually from 90 to 110 degrees, the real theory of arches applying only to material between these limits. The part below the points of rupture must be designed in connection with the substructure.

The greatest economy in the design of abutments is secured by using low springs. If higher springs are desired, they can be secured by false side walls as explained and illustrated in Part I. Great saving can be effected in high abutments by coring out the rear and transferring the thrust to the soil through vertical walls bearing on a foundation slab of reinforced concrete. Abutment wings may be built as cantilevers from the arch, extending into the embankment only far enough to hold the slope. They contain much less masonry than the old style of gravity retaining wing walls. Cantilever wing walls should be tied together with rods beneath the roadway, to resist the outward thrust of filling. This method was adopted in the Topeka bridge.

The recent failure of the Peoria bridge over the Illinois River, has called attention to the need of having absolutely secure foundations. The Peoria bridge was destroyed, not because of any lack in the design of the superstructure, but because of the undermining of its foundations.

Flaring gravity wing walls are more economical than straight ones of the same type and better direct water to the opening, but straight wings usually present a better appearance.

River piers require cut-waters at the upper end which should be capped with stone or steel, well anchored into the masonry.

Some bridge piers have been given a different batter on the two sides for resisting the unequal thrust on the sides from spans of different lengths. The piers must have sufficient thickness to resist the uneven thrust caused by full live loading on one span and no live load on the other. Piers must be designed, not by empirical rule, but according to the stresses that they actually have to resist.

The presence of reinforcing rods for resisting temperature stresses in piers, is desirable though not necessary. Piers are usually well protected from the direct rays of the sun, and rods are more useful to unite the mass into a solid monolith than for resisting temperature stresses.

The design of piers for reinforced concrete bridges does not differ greatly from the design of piers for masonry bridges, and most of the discussion of this subject for Concrete Bridges, applies equally here.

Cost of Reinforced Concrete Bridges.

There are numerous considerations that affect the cost of reinforced concrete bridges, among which are the nature of the soil, the nearness or accessibility of materials, presence or absence of switching facilities, the design of the bridge whether solid filled or open spandrel, the height, width, finish, paving, wings, etc. They will, however, rarely if ever cost more than bridges of solid concrete. An original formula for the cost of solid concrete bridges has been given in Part I, but for convenience it is repeated here. It is as follows:—

$$C = F \frac{HW}{100}$$

Where C is the cost of the bridge in dollars per square foot of roadway,

W, the total width of deck in feet,

H, the height of deck above valley or river bottom, and

F, a variable factor the value of which is as given below,

The function HW, or the product of height by width, is the cross-sectional area, and may be represented by the letter A. Factors F, are for bridges with solid slab arches, while factors F′ are for bridges with partial slabs, like the Walnut Lane bridge at Philadelphia, or the Rocky River bridge at Cleveland.

Values of Factors F, and F′.

When A is	200,	then F is	1.5
"	500,	"	1.0
"	1000,	"	.65
"	1500,	"	.48
"	2000,	"	.42
"	2500,	"	.36
"	3000,	"	.32
"	3500,	"	.285

When A is	4000,	then F is	.262	and F′ is	.96
"	5000,	"	.224	"	.95
"	6000,	"	.200	"	.94
"	7000,	"	.180	"	.93
"	8000,	"	.164	"	.92
"	9000,	"	.152	"	.91
"	10000,	"	.141	"	.88
"	11000,	"	.133	"	.86
"	12000,	"	.125	"	.85

This formula will give costs that should rarely if ever be exceeded. Generally, however, economically designed reinforced concrete bridges should cost from 25% to 50% less than the costs given by the formula for bridges in solid concrete. In a few cases, the cost of bridges in reinforced concrete have exceeded that given by the formula, but these cases are rare. Where the height does not exceed 15 to 20 feet, the cost will usually vary from $2.00 to $4.00 per square foot of floor surface, while for greater heights it may be twice these amounts.

The total cost, as well as the cost per square foot of deck for a miscellaneous lot of reinforced concrete bridges is given in Table No. II. The square foot cost is based upon the total length of bridge over parapets or foundations, and not upon the length of opening. If based on the latter length, the costs per square foot would then be greater.

The cost of 18 concrete arch highway bridges, built by the city of Philadelphia, is reported in Engineering Record January 23, 1909. The report states that the bridges were mostly single span with ornamental balustrade, washed granolithic surfaces and paved decks. The costs based upon the total length of bridge vary from $1.73 to $7.39 per square foot, or an average of $3.50 per square foot, while the costs based upon the width multiplied by the clear length of opening vary from $3.10 to $9.74, or an average of $6.25 per square foot. The total cost based upon the yardage of concrete in the structure varies from $8.50 to $11.25 per cubic yard. The report states further that if large spalls or stones were embedded in the concrete to save cement and mixing, the cost would then be reduced by about 20%.

Compared with steel, reinforced concrete bridges usually cost about the same as steel bridges with solid floors. The report referred to above states that those built in Philadelphia proved to be cheaper in first cost than plate girder bridges by about 25%, but if maintenance expense is considered, the saving is still greater.

Comparative estimates for the Memorial Bridge at Washington, one design for which is given in the frontispiece, showed that the reinforced concrete designs cost 45% more than corresponding designs in steel.

A bridge over the Hudson River at Sandy Hill, N. Y., consisting of 15 ribbed arch spans of 60 feet each, cost only $2.30 per square foot and a steel bridge for the same loads would have cost as much.

Bids received for a bridge over the Mississippi River at Fort Snelling Minn., consisting of two arch spans 350 feet in length each, showed that the bridge could be built in either steel or reinforced concrete at about the same cost.

A concrete design for the Richmond trestle shown in Figure 40 is reported to have been accepted in preference to steel, simply because it was the cheaper.

Estimating.

It is customary to estimate the total cost of floor slabs, including concrete, metal reinforcement and forms, at 25 cents per square foot of floor for the slab only. This figure is made up as follows:—

Concrete, 6 inches	12 cents
Metal	5 cents
Wood forms	8 cents
Total	25 cents

The cost of forms varies considerably, and for floor slabs may cost from 8 to 20 cents per square foot of floor. If the slabs are estimated separately, then it is necessary to estimate also the cost of floor beams and spandrel columns. It is usual to estimate the cost of forms for beams and columns of ordinary size, not exceeding about one and a half foot in cross-section, at 50 cents per lineal foot. To this must be added the cost of the concrete and steel in the member. The total cost per lineal foot of girder or columns would then be as follows:—

Concrete 1 cu. feet	25 cents
Steel	15 cents
Forms	50 cents
Total	90 cents

TABLE II

APPROXIMATE ESTIMATING PRICES

	Price delivered.	Price in Place.
Earth filling	....	$0.50 to $1.00 per yd.
Excavating, ordinary	....	.50 cu. ft.
" under water (including cost of cofferdam)	....	4.00 cu. ft.
Wood piling	....	.35 lin. ft.
Sheet piling	....	40.00 per M.
Concrete piling	....	1.25 per ft.
Concrete in foundations	....	6.00 " yd.
" in arch rings	....	8.00 " "
" including steel reinforcement	....	12.00 " "
Concrete including steel reinforcement and centers	....	18.00 " "
Steel reinforcement, riveted work	....	70.00 per ton
" rods, plain	....	30.00 " "
" patented rods	....	50.00 " "
Brick, common	$6.00 to $10.00 per M.	20.00 per M.
" face	30.00 " "	45.00 " "
" moulded	50.00 " "	70.00 " "
" enameled	70.00 " "	100.00 " "
Concrete blocks, 10 inches thick	.25 cu. ft.	.30 cu. ft.
Sand	.75 to 1.25 " yd.	....
Gravel	1.25 " "	....
Cement, Portland	1.35 per barrel	....
" non-staining	3.25 " "	....
Crushed limestone	1.20 per yd.	....
" granite	3.00 to 3.50 " "	....
Bedford limestone	1.30 per ft.	1.60 per ft.
Carthage limestone	2.00 " "	2.30 " "
Kasota or Mankato stone	2.50 " "	2.80 " "
Granite	2.50 to 3.00 " "	3.30 " "
Bedford ashlar facing, 4 to 8 inches thick	....	1.00 sq. ft.
Bedford stone carving	....	4.00 " "
Concrete floor slabs (concrete, steel, forms)	....	.25 " "
Concrete girders and columns (concrete, steel and forms)	....	1.00 lin. ft.
Concrete columns, spiral wound	....	1.70 " "

TABLE II—Continued

APPROXIMATE ESTIMATING PRICES

	Price delivered.	Price in Place.
Bridge pavements, wood block..		$1.50 sq. yd.
" " granolithic walks..		1.50 " "
" " brick........		2.50 " "
" " asphalt		3.50 "
" " stone block.		3.00 " "
" " granite block.		4.50 " "
Railing, three lines pipe.........		1.00 per ft
" plain iron lattice		2.00 " "
" fancy iron lattice.......		5.00 " "
" artificial stone		6.00 " "
Balusters, turned Bedford stone.....		1.00 each
Hand rail and base rail.		.60 per ft.
Stone coping		2.00 " "
Intermediate rail posts		8.00 to 12.00 each
End newels..............		10.00 " 100.00 "
Lamp posts		20.00 " 100.00 "
Trolley poles		15.00 " 75.00 "
Lumber in cofferdams		40.00 per M.
" " arch centers..........	$22.00	32.00 " "
" " forms.............		.08 sq. ft.
Beam and column forms ..		.50 lin. ft.
Metal lath and plaster, interior		.50 per sq. yd.
" " " " exterior ..		.90 " "
Expanded metal No. 10, 4-inch mesh..	.035 per sq. ft.	....
" " light.......	.02 " "	....
Nails and spikes		.03 per lb.
Tar paper............		.005 per sq. ft.
Toch Bros. waterproof paint, No. 10...		1.25 " gal.
Bay State coating (for concrete surfaces) two coats		.02 " sq. ft.

If the girder or column is larger than 12 inches square, the cost of the concrete will then increase in proportion to its area.

In making up a tender on a prospective contract, it is necessary that all items of expense be included and provided for. Some of the extra expense items, that are not included in the regular estimate, are as follows:—

Superintendent.
Foreman.
Timekeeper.
Traveling Expenses.
Bond. Cost is 1 per cent. on amount of bond, which is usually 25 per cent. of contract.
Telephones.
Watchmen.
Fire Insurance.
Liability. Cost is 2½ to 3½ per cent of amount of pay roll.
Permit and License.
Water.
Setting out survey.
Rent of, or depreciation on plant.
Office and Storage sheds.
Material tests.
Models.
Signal lights.
Pumping and Baling.
Refilling and Leveling.
Shoring.
Removing Rubbish.
Incidentals.
Surfacing.

These items must be provided for and the amount of profit desired added to the total.

The approximate estimating prices given above, should be changed to suit local conditions and the varying state of the market. Prices of material and labor change according to location and time, and prices that are suitable in the East may not hold for work in the West or South. The greatest care is necessary in estimating the foundations, for the part that is unseen is uncertain. It is well to make unit prices for a greater or less amount of foundations than is shown on the plans, for frequently more is required than is anticipated.

Table of Approximate Quantities.

The following table gives the approximate quantities in Reinforced Concrete Arch Highway Bridges for clear spans varying from 20 to 150 feet, and a clear width of roadway of 16 feet.

They have solid earth filled spandrels with reinforced concrete side retaining walls and the rise of arch is one-tenth the span.

They are proportioned for a live load of 200 pounds per square foot on the roadway. The quantities of material in the abutments are only approximate.

TABLE OF APPROXIMATE QUANTITIES.

Clear Span in Feet.	Crown Thickness in Inches	Steel, Bars 12 in. c. c.		Concrete in Arch, Cu. Yds.	Concrete in Abutments, Cu. Yds.
		Size.	Area in Sq. Ins.		
20	9	2 x 1/4	.50	30	89
30	11	2 x 5/16	.62	45	118
40	13	2½ x 5/16	.78	63	110
50	15	2½ x 3/8	.93	89	162
60	16.5	3 x 5/16	.94	118	205
70	18	3 x 3/8	1.13	150	240
80	19	3½ x 3/8	1.31	186	280
90	21	3½ x 7/16	1.53	220	320
100	22	4 x 3/8	1.50	265	360
110	24	4 x 7/16	1.75	312	410
120	26	4½ x 3/8	1.69	360	460
130	28	4½ x 7/16	1.97	415	515
140	30	5 x 3/8	1.88	475	570
150	32	5 x 7/16	2.19	540	630

Potomac Memorial Bridge Design.

This is one of several designs submitted to the United States Government in the year 1900 for a proposed memorial bridge across the Potomac River at Washington. It has a clear width of 60 feet, consisting of a 40-foot roadway and two 10-foot sidewalks. The total length of open bridge is 3,400 feet. It has one deck and no provision for car tracks. There are six segmental reinforced concrete arch spans of 192 feet clear length and 29 feet rise, with 53 feet clearance underneath. A double leaf trunnion bascule draw span is centrally located between the arch spans, having a clear opening of 159 feet and a distance between centers of trunnions of 170 feet. The Washington approach consists of twelve semicircular reinforced concrete arch spans of 60 feet clear length, and 550 feet of embankment, while the Arlington approach has fifteen similar spans and 1,350 feet of embankment. The entire exterior surface is shown faced with granite. The face rings for main spans are 5 feet 6 inches deep at the crown and 9 feet 6 inches at the springs. Each main span has five concrete-steel arch ribs 30 inches deep at the crown and 7 feet 3 inches at the springs, supporting a system of interior steel columns carrying the floor beams. Spandrel curtain walls with expansion joints rest upon the arch rings and are faced with granite. The design shows asphalt road and granolithic walks laid on concrete floor arches between the steel floor beams. The estimated cost is $3,680,000. William H. Burr, engineer; E. P. Casey, architect.

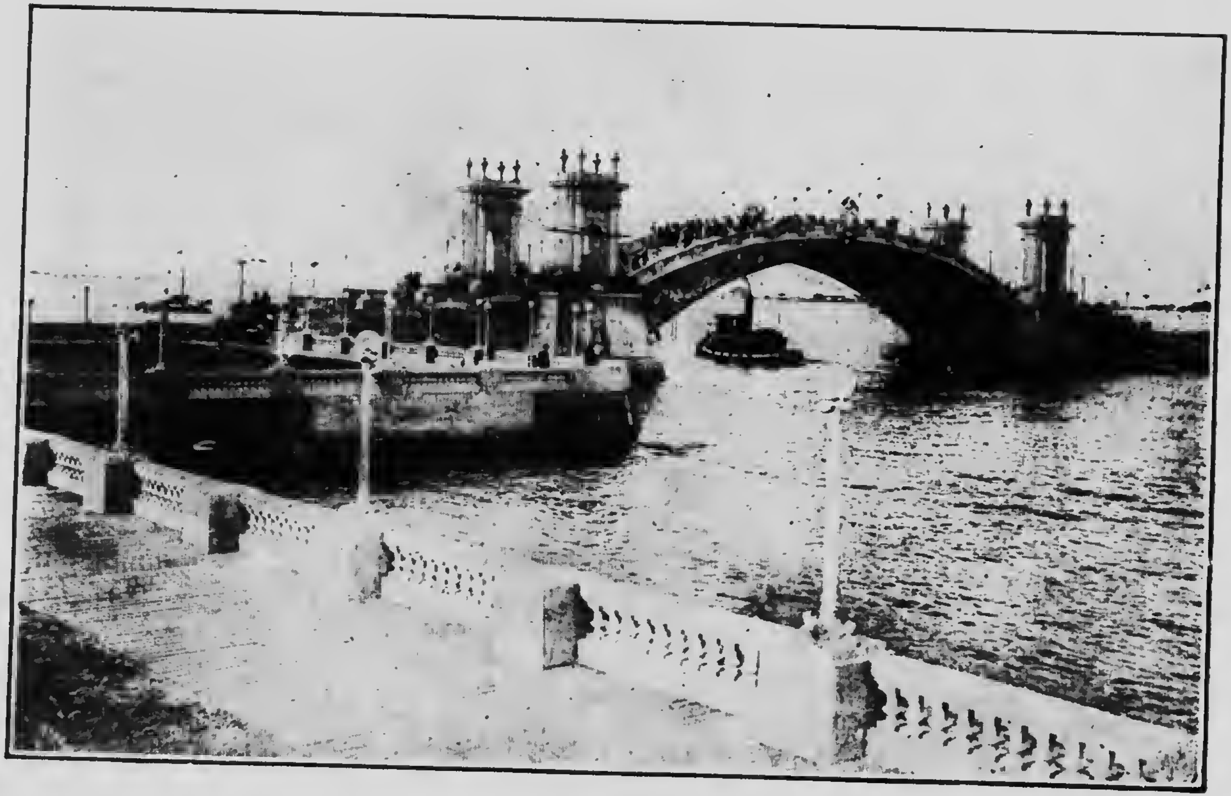


Fig. 28
JAMESTOWN EXPOSITION BRIDGE

Jamestown Exposition Bridge.

This bridge was built in 1907 by the United States Government to connect the outer ends of two piers. It is of reinforced concrete and has a clear span of 151 feet, with a 20-foot rise. It is 36 feet wide and is for pedestrians only. The ascent of the roadway is made by means of a series of steps and landings. It has two reinforced concrete arch ribs carrying the roadway on four longitudinal walls. The abutments are cored out and rest on piles. There are 26 plumb piles and 126 batter piles under each abutment. It was designed and built by the Scofield Company of Philadelphia.

Franklin Bridge, Forest Park, St. Louis.

Forest Park has a very interesting concrete bridge of the Melan type, known as Franklin Bridge. It has a span of 60 feet, a total width of 33 feet, and a rise of 15 feet. It has a 24-foot roadway and one 6-foot sidewalk, with a total length of 92 feet. The arch ring is three-centered and varies in thickness from 11 inches at the crown to 30 inches at the springs. At the four corners there are ornamental iron lampposts not shown in the illustration. Its total cost was $5,600. The Geisel Construction Company were the contractors and John Dean, Engineer for the Park Department.

Jefferson Street Bridge, South Bend, Ind.

The bridge across the St. Joseph River with four elliptical arches of 110-foot span each. The piers are quite elaborate in design, being carried

Fig. 29
FRANKLIN BRIDGE. FOREST PARK. ST. LOUIS, MO.

up to support retreats at the sidewalk, and there is a heavy moulded cornice surmounted with an artistic railing. At the ends are steps leading down from the roadway to the river. The lines of the structure are true to a design in concrete, and there has been no effort made to imitate stone. The Concrete Steel Engineering Company of New York, were engineers, and James O. Heyworth of Chicago, contractor. A. J. Hammond, City Engineer of South Bend.

Gary, Indiana, Bridge.

Gary is the home of the new steel companies where an entirely new town is being built. The bridge shown is quite ornamental, and illustrates some possibilities for single spans. The face of arch and spandrels are paneled, and the wings are curved to facilitate approach. At either end of the arch are pilasters extending up to the cornice and forming in the balustrade, pedestals for future lamp standards. The bridge spans the Calumet River and was built in 1908 by Rudolph S. Blome & Co., of Chicago.

Como Park Foot Bridge, St. Paul.

The Como Park Bridge was built in the year 1903 for the Twin City Rapid Transit Company to carry traffic entering Como Park, over the tracks of the street railway company. The bridge has a clear span of 50 feet, a roadway of 15 feet and is built on the Melan system. As a large number of passengers leave the cars at the bridge, it was

Fig. 29
FRANKLIN BRIDGE. FOREST PARK. ST. LOUIS, MO.

Fig 30
JEFFERSON STREET BRIDGE, SOUTH BEND, INDIANA

Fig. 31
BRIDGE AT GARY, INDIANA

desirable that the structure should have a neat appearance. In order to avoid form marks on the exposed surfaces the forms were covered with metal lath and neatly plastered before placing the concrete. The length between centers of abutment piers is 83 feet, and the total width of arch is 17 feet 2 inches. It has a rise of 12 feet 6 inches, and is 10 inches thick at the crown. The length of span openings over spandrels and abutments is 12 feet, and the thickness of the skewback piers is 2 feet. There are five latticed steel Melan arch ribs in the concrete. It was built by William S. Hewitt & Co., of Minneapolis. George L. Wilson was consulting engineer.

Boulder-Faced Bridge, Washington.

In a park at Washington, D. C., there is a boulder-faced arch of rustic design made to conform with the surroundings. It has a span of 80 feet, a rise of 15 feet, and a clear width of roadway between parapets of 23 feet. The entire arch ring is built of concrete, but the soffit is darkened with lampblack to harmonize with the boulder facing. The boulders of the arch ring extend down below the soffit several inches, and partly obscure the concrete arch soffit. It was built in 1901 at a cost of $17,500. W. J. Douglas, Engineer.

Grand Rapids Arch Bridge.

This is a good example of the best American practice in reinforced concrete arch bridge design. It has five spans, the center one being 87 feet, the

Fig. 32
COMO PARK BRIDGE, ST. PAUL, MINN.

Fig. 33
BOULDER FACED BRIDGE, WASHINGTON, D. C.

two adjoining ones 83 feet, and the two end spans 79 feet. It has a clear width between railings of 64 feet. The piers have moulded concrete cornices at the springs, and there is a continuous cornice supported on brackets at the floor level. There are retreats in the sidewalk above the piers, and a heavy open balustrade with seven heavy railing posts in each span. It was designed by William F. Tubesing under the direction of L. W. Anderson, City Engineer, and was built in 1904 by J. P. Rusche, contractor, of Grand Rapids, Mich.

Bridge at Venice, California.

At the little town of Venice in lower California, laid out with numerous canals in imitation of Italian Venice, are a number of bridges mostly built of concrete with features of unusual design. The town being on the sea coast, in a region where flowers and foliage abound, has probably suggested the ornamentation. The faces of the arch are elaborately decorated with festoons, and on the ends of the balustrade are grotesque figures of sea animals in concrete, the size of which may be estimated by comparison with the people on the bridge.

Garfield Park Bridge, Chicago.

The illustration shows an attractive park bridge built in the year 1893 in Garfield Park. The open balustrade with the heavy circular piers together with the combination of rough and smooth finish unite to produce a pleasing appearance. Medallions on the piers have monograms with the park

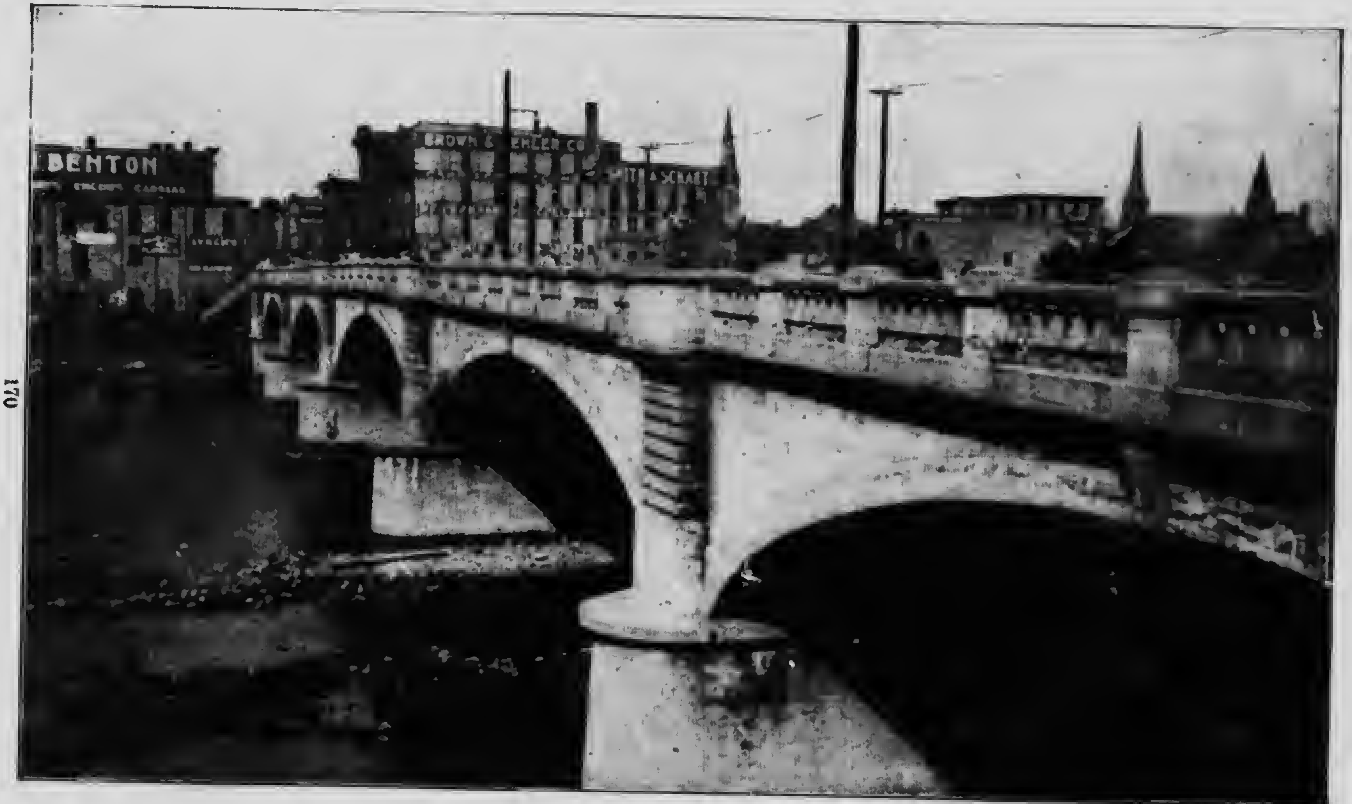


Fig. 34
BRIDGE OVER GRAND RIVER, GRAND RAPIDS, MICHIGAN

Fig. 34
BRIDGE OVER GRAND RIVER, GRAND RAPIDS, MICHIGAN

Fig. 35
BRIDGE AT VENICE, CALIFORNIA

MICROCOPY RESOLUTION TEST CHART

(ANSI and ISO TEST CHART No. 2)

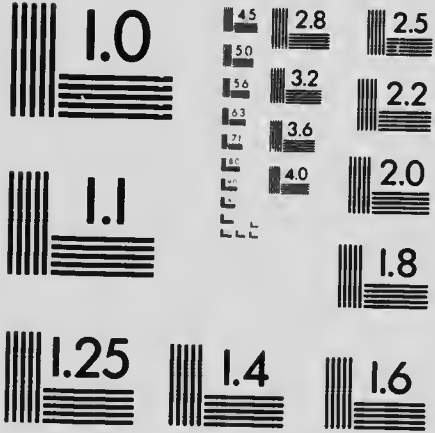


Fig. 36
GARFIELD PARK BRIDGE, CHICAGO, ILLINOIS

initials, and the spandrels are paneled. The balustrade posts are mounted with ornamental urns. The design is one which can well be reproduced in concrete with either cut stone or moulded concrete facing.

Stein-Teufen Bridge, Switzerland.

The longest concrete arch span completed is at Stein, Switzerland. Its total length is 550 feet, and the roadway, 32 feet wide, is 216 feet above the Sitter River. The central span is 259 feet, with two approach spans $33\frac{1}{2}$ feet long at the Teufen end, and four at the other end. The central piers are heavily reinforced to resist unbalanced thrusts from the adjoining arches. The main arch rings are $21\frac{1}{2}$ feet wide and 4 feet thick at the crown, increasing to the springs, and reinforced with $1\frac{1}{8}$-inch round bars from 10 to 18 inches apart. It has a Telford pavement and 2-foot walks on concrete slabs supported on stringers and spandrel columns. The concrete balustrade has openings 3 feet wide, guarded with embedded bars. It was designed by Professor Morsch, and cost $80,000.

Fig. 36
GARFIELD PARK BRIDGE, CHICAGO, ILLINOIS

TABLE III

LIST OF REINFORCED CONCRETE BRIDGES

Number.	PLACE.	Over.	No. Span.	Length of Span.	Rise.	Total Length.	Width.	Height.	Curve.
1	Stein, Switzerland	Sitter River	1	259	87	550	22	216	43
2	" "	" "	6	33.5	15	"	"		14
3	Fogaras-Kronstadt, Hungary			197					
4	Decize, France.	Loire River.	2	184	15		34		
5	Pyrimont, France.	Rhone River	2	177	25	612	12	100	C.
6	" "	" "	1½	175	"	"	"	"	
7	Bormida, Italy.		1	167	16.7				
8	Chatellerault, France.	Vienne River	1	164	15.7	443	26	22	
9	" "	" "	2	131	13.2	"	"	"	
10	Painesville, Ohio.	Grand River	1	160	71	401	68	90	Seg.
11	" "	" "	2	70		"	"	"	
12	Jamestown, Virginia.		1	151	26		36		
13	Playa del Rey, Cal.		1	146	18	205	19		
14	Wakeman, Ohio.	Vermillion River	1	145	33.5	219	21	36	
15	Route Waidhofen, Austria.		1	144					
16	Steyr, Austria.		1	139	8.5	165	19.7	19	Seg.
17	Branch Brook Park, Newark		1	132	16.2	244	74	24	
18	Topeka, Kansas	Kansas River.	1	125	18.9	693	40	32	
19	" "	" "	2	110	16.3	"	"	"	
20	" "	" "	2	97.5	14.6	"	"	"	
21	Route Wildegg, Switzerland		1	122	11.4		12.8		
22	Yellowstone Park.	Yellowstone River	1	120	15	160	17	43	
23	Porto Rico	Jacaquas River.	1	120	12	404	20	39	3 C.
24	" "	" "	2	100	11.4	"	"	"	"
25	Lansing, Michigan	Grand River	1	120	23		64		
26	Lake Park, Milwaukee.	Ravine.	1	118	18	214	54	25	
27	Portugal.	Lena River.	5	114	14.4		11.8		
28	Third St., Dayton, Ohio.	Miami River	1	110	9.6	710	12	30	
29	" " " "	" "	2	100	11.3	"	"	"	
30	" " " "	" "	2	90	13.3	"	"	"	
31	" " " "	" "	2	80	14.3	"	"	"	
32	Avranche, France.		1	110	23.5		10.5		
33	Boulevard, St. Paul.		1	110	40	222	40	50	
34	Green Island.	Niagara River	1	110	11.5	371	41	16	
35	" "	" "	2	100	10	"	"	"	
36	Jefferson St., South Bend.	St. Joseph River	4	110					
37	Emerichsville, Ind.	White River.	3	110					
38	Morris St., Indianapolis.	" "	5	90-110					
39	Laibach, Austria.		1	108	14.6	170	50	25	3 C.
40	Huntington, Indiana.		2	104	14	240	16	21	
41	Buda Pesth, Austria.	Danube River.	1	108	14.4		45		
42	" " "	" "	2	96	9.5	"	"	"	
43	Canal Dover, Ohio.	Tuscarawas River.	3	107	11.7	522	55	30	

TABLE III—Continued

LIST OF REINFORCED CONCRETE BRIDGES

Crown Thickness in Inches.	Size Bars.	Distance Apart.	Date.	Kind H. R. or El.	Cost	Engineer.	References. E.C., Eng.-Con. N., " News. R., " Record	Rib.	Hinge.	Cost Per Sq. Ft.	Number.
1 1/8	10		1909	H.	$80,000	Morsch	N., Aug. 5, '09				1
9/16	8										2
											3
18				H.							4
			1907	H.	$42,400	De Mollius	N., Apr. 2, '08	Rib.		$5.50	5
			"	"							6
21			1902	H.							7
22			1899	H.	35,000		N., Apr. 10, '02	Rib.		3.05	8
18			"	"							9
87			1908	R. R.		Leffler	R., Apr. 24, '09				10
54			"	"							11
			1907			Scofield Eng. Co.		Rib.			12
24			1906			De Palo	N., July 26, '06	Rib.			13
			1908	H.	16,870	Watson	E.C., Feb. 24, '09	Rib.	3 H.	3.66	14
				H.				Rib.	3 H		15
24		36	1897	H.							16
		36	1895	H.	84,000	Reynolds	R., Aug. 12, '05			4.65	17
20	Frame	36	1897	H.	150,000	Keepers & Thacher	R., Apr. 16, '98			5.40	18
"		"					N., Apr. 2, '96				19
19		"									20
7			1890	H.							21
24		24	1904	H.		Crittenden	N., Jan. 14, '04				22
28	4 x 3/4	30	1901	H.	59,440	Judson	R., Aug. 3, '01			7.40	23
21	"	"	"	"			N., Aug. 1, '01				24
			1902	H.	31,000		E.C., Mar. 17, '09				25
			1904	H.		Newton Eng. Co.	R., Nov. 25, '05	Rib.			26
			1901	E. R.							27
25		32	1906	H.	184,000	Turner	R., Mar. 4, '06			4.18	28
"		"	"	"							29
"		"	"	"							30
"		"	"	"							31
								Rib.			32
			1909	H.	18,800	C. A. P. Turner	R., Apr. 3, '09			2.12	33
40	6 x 3/4	36	1900	H.	102,070		N., Dec. 6, '00			6.60	34
38	"	"	"	"			R., Feb. 16, '01				35
				E. R.		Hammond					36
				H.							37
				H.							38
20		42	1901	H.	32,000	Melan	N., July 16, '03			3.77	39
21	3/4	12	1907	H.		Luten					40
20			1900	H.							41
24											42
24	1 1/4	12	1905	H.	105,000	Thacher	R., Feb. 9, '07				43

TABLE III—Continued

LIST OF REINFORCED CONCRETE BRIDGES

Number.	PLACE.	Over.	No. Span.	Length of Span.	Rise.	Total Length.	Width.	Height.	Curve.
44	Canal Dover, Ohio	Tuscarawas River	1	70	10	522	55	30	.....
45	Pelham	Chester Bay	6	105	16.5	807	52	30	El.
46	" Draw Span	" "	1	62	....	"	"	"	.....
47	Paterson, New Jersey	Passaic River	1	108	12	360	40	20	3 C.
48	Wayne St., Peru, Indiana	Wabash River	1	100	15	686	30	28	.....
49	" " " "	" "	2	95	"	"	"	"	.....
50	" " " "	" "	2	80	"	"	"	"	.....
51	" " " "	" "	2	75	13	"	"	24	.....
52	Sixth Ave., Des Moines, Iowa	Des Moines River	3	100	27.7 23.9 20.	360	42.7	32	3 C.
53	Stockbridge, Mass.	Hoosatonic River	1	100	10	124	7.5	15	.....
54	Decatur, Illinois	Sangamon River	2	100	30	640	28	60	.....
55	" "	" "	2	93	"	"	"	"	.....
56	Yorkton, Indiana	.....	1	95	11	....	18	.....	.....
57	Cartersburg, Indiana	.....	2	90	15.7	212	22	26	.....
58	Washington St., Dayton	Miami River	1	90	11.5	620	54	30	.....
59	" " "	" "	2	86	10	"	"	"	.....
60	" " "	" "	2	80	9.3	"	"	"	.....
61	" " "	" "	2	74	8	"	"	"	.....
62	Waterville, Ohio	Maumee River	12	75-90	22-25	1200	16	45	.....
63	Main Street, Dayton	Miami River	1	88	....	588	56	30	3 C.
64	" " "	" "	2	83	....	"	"	"	"
65	" " "	" "	2	76	....	"	"	"	"
66	" " "	" "	2	69	....	"	"	"	"
67	Paterson, New Jersey	Passaic River	3	88	9.5	317	50	18	.....
68	Grand Rapids, Mich.	Grand River	1	87	....	493	64	30	.....
69	" " "	" "	2	83	8	"	"	"	.....
70	" " "	" "	2	79	11	"	"	"	.....
71	Seeley St., Brooklyn	Prospect Avenue	1	85	8.5	144	53	18	.....
72	New Goshen, Ohio	.....	5	83.5	8.2	494	16	20	5 C.
73	Sarajero, Bosnia	Miljacka	1	81	8	107	36	16	Seg.
74	Decorah, Iowa	.....	2	81	9.7	187	26	18	.....
75	Washington, D. C.	Rock Creek	1	80	15	130	27	18	Seg.
76	Soissons, France	Aisne River	3	80	8	305	45	30	Seg.
77	Colfax Ave., South Bend	.....	1	77	7	....	....	....	.....
78	Cedar Rapids, Iowa	.....	8	75	7	....	42	.....	3 C.
79	Pollasky, California	San Joaquin River	10	75	11	780	19.5	18	.....
80	Kresno, Galicia	.....	1	75	11	257	....	21	.....
81	" "	.....	2	73	"	"	....	"	.....
82	Hyde Park-on-Hudson	Crum Elbow Creek	1	75	14.7	...	20	....	5 C.

TABLE III—Continued

LIST OF REINFORCED CONCRETE BRIDGES

Crown Thickness in Inches.	Size Bars.	Distance Apart.	Date.	Kind H. R. or El.	Cost.	Engineer.	References. C., Cement N., Eng. News R., " Record	Rib.	Hinge.	Cost Per Sq. Ft.	Number.
15	1¼	18	1905	...	......	Thacher........	R., Feb. 9, '07..	......	....	.....	44
24	Frame	36	1908	H.	......	Lindenthal......	R., Oct. 31, '08..	......	....	.....	45
...	......	...			......	....	....	......	....	.....	46
28	1	24	1907	H.	37,200	Wise...........	R., Mar. 7, '08..	......	....	.....	47
25	¾	6	1905	H.	36,900	Luten..........	N., Mar. 29, '06..	......	....	1.80	48
"	"	"	"	"	......	....	....	......	....	.....	49
"	"	"	"	"	......	....	....	......	....	.....	50
21	"	12	"	"	......	....	....	......	....	.....	51
21	......	...	1901	H.	......	....	C., July, '02..	......	....	.....	52
9	7 I	28	1895	F. B.	1,475	Von Emperger....	N., Nov. 7, '95..	......	....	1.58	53
45	1	12	1907	R. R.	117,000	Cunningham......	N., Mar. 21, '07..	......	....	6.50	54
"	"	"	"	"	......	....	....	......	....	.....	55
...	......	...	1905	...	......	Luten..........	N., May 11, '05..	......	....	.....	56
21	¾	6	1907	E. R.	......	Luten..........	....	......	....	.....	57
20	Frames	36	1905	H.	122,000	Turner..........	R., Mar. 2, '07..	......	....	3.60	58
11		"	"	"	......	....	....	......	....	.....	59
7		"	"	"	......	....	....	......	....	.....	60
17		"	"	"	......	....	....	......	....	.....	61
24	1	6	1908	E. R.	77,000	Walker..........	....	......	....	4.00	62
20	Frames	.	1903	H.	140,000	Turner..........	R., Aug. 8, '03..	......	....	4.30	63
"			"	"	......	....	N., May 19, '04..	......	....	.....	64
"		36	"	"	......	....	....	......	....	.....	65
"			"	"	......	....	....	......	....	.....	66
15	10 I	36	1897	"	......	Thacher........	N., Mar. 16, '99	......	....	.....	67
1[illegible]	1¼	36	1904	"	......	Tubesing........	N., Dec. 1, '04	......	....	.....	68
"	"	"	"	"	......	....	....	......	....	.....	69
19	"	"	"	"	......	....	....	......	....	.....	70
27	1¼	8	1903	"	21,800	Fort...........	N., Dec. 31, '03..	......	....	2.90	71
18	1¼	18	1906	"	36,900	Murray..........	R., Mar. 30, '07..	......	....	4.65	72
12	......	24	1897	"	16,500	Wunsch.........	....	......	....	4.30	73
18	⅝	12	1906	"	......	Luten..........	....	......	....	.....	74
17	......	33	1901	"	17,500	Douglas.........	R., Aug. 16, '02.. N., Aug. 14, '02	......	....	4.95	75
12	......	...	1903	R. R.	......	Riboud..........	....	Rib.	....	.....	76
...	......	...	1901	H.	......	....	....	Rib.	....	.....	77
16	......	36	1906	"	......	Marsh Bridge Co.	....	......	....	.....	78
18	¾	...	1905	"	48,000	Leonard.........	R., Feb. 24, '06..	......	....	3.15	79
...	......	...		...	......	....	....	......	....	.....	80
...	......	...		...	......	....	....	......	....	.....	81
16	....		1897	H.	.....	Concrete Steel Co.	....	......	....	....	82

TABLE III—Continued

LIST OF REINFORCED CONCRETE BRIDGES

Number.	PLACE.	Over.	No. Span.	Length of Span.	Rise.	Total Length.	Width	Height.	Curve.
83	"Big Four" bridge near Terre Haute	....	3	75					
84	Wabash, Indiana	Charley Creek	2	75	18	240	32	24	Par
85	Mission Ave., Spokane	....	4	70	....	....	....	....	El.
86	Olive Ave., Spokane	Spokane River	5	95	12	565	56	26	El.
87	Meridian St., Indianapolis	Fall Creek	3	74	9.5	284	70	18	3 C.
88	Illinois St. "	" "	3	74	9.5	284	60	18	3 C.
89	Northwestern Ave., Indianapolis	....	3	74	....	....	....	....	....
90	Derby, Conn.	....	3	72	....	....	54	....	....
91	Waterloo, Iowa	....	7	72	7.2	586	46	20	C.
92	Eden Park, Cincinnati	Park Drive	1	70	10	....	33	....	C.
93	Logansport, Indiana	Pine Creek	2	70	14	163	16	19	....
94	Austell, Georgia	....	4	70	20	300	26	45	5 C.
95	Trinidad, Colorado	....	2	70	7	201	64	15	3 C.
96	Wabash, Indiana	....	2	70	18	240	32	24	Par.
97	Seventeenth St., Boulder, Col.	....	1	70	10.5	89	24	13	....
98	Iola, Kansas	Miners Ford	3	70	9	242	14	14	....
99	Porto Rico	Guaya River	3	70	7	270	20	20	3 C.
100	Boulevard Bridge, Philadelphia	....	1	69	....	....	....	....	....
101	Jacksonville, Florida	R. R. Tracks	11	66	7	845	52	30	....
102	Herkimer, N. Y.	W. Canada Creek	3	66	14	755	....	32-40	Seg.
103	" "	" "	7	62	12	"	....	"	"
104	Sandy Hill, N. Y.	Hudson River	15	60	8.5	1025	35	24	....
105	Franklin Bridge, St. Louis	Park Stream	1	60	15	92	33	....	3 C.
106	Lima, Ohio	....	2	59	8.5	161	16	18	....
107	Plainwell, Michigan	Kalamazoo River	7	54	8	446	23	22	Seg.
108	Maryborough, Queensland	Mary River	11	50	4	613	23	22	Seg.
109	Como Park, St. Paul	Tracks	1	50	12.5	83	17	....	....
110	Atlantic Highlands, N. J.	Grand Ave.	1	50	11	....	25	....	C.
111	Glendoin, Cal.	San Gabriel River	18	50	....	1019	26	....	....
112	Forest Park, St. Louis	River Des Peres	1	45	12	65	45	22	....
113	London, Ohio	....	1	45	6.2	150	16	12	....
114	" "	....	2	40	5.7	"	"	"	....
115	Cruft St., Indianapolis	....	1	43	8.5	53	20	11	....
116	Oconomowoc, Wisconsin	....	1	42	6.8	....	15	....	....
117	Columbia Park, Lafayette	....	1	40	4	56	6	8	....
118	Interlaken Bridge, Minneapolis	....	1	38	....	82	63	....	....
119	Plainfield, Indiana	....	1	42	7.6	216	16	17	....
120	" "	....	2	39	7.2	"	"	"	....
121	" "	....	2	35	6.5	"	"	"	....
122	Chicago, C. & E. I. Ry	Trim Creek	1	38	7	....	48	....	....

TABLE III—Continued

LIST OF REINFORCED CONCRETE BRIDGES

Crown Thickness in Inches.	Size Bars.	Distance Apart.	Date.	Kind H. R. or El.	Cost.	Engineer.	References. E.C., Eng.-Con. N., " News R., " Record	Rib.	Hinge.	Cost Per Sq. Ft.	Number.
				R. R.		Duane					83
18	3 x 1	24		H.		Hilty					84
					55,000	McIntyre	N., Dec. 5, '07				85
27				H.		Ralston					86
16	10" I	36	1900	H.	54,400	Jeup	N., Apr. 11, '01			2.65	87
16	10" I	36	1900	"	50,900	Jeup	N., Apr. 11, '01			2.90	88
				"		Concrete Steel Co.					89
				"							90
14	2 1/2 x [illegible]	8"	1902	"	54,000	Concrete Steel Co.	E.C., Mar. 17, '07 R., Feb. 13, '04			2.00	91
15	9" I	36	1895	"		Von Emperger	N., Oct. 3, '95				92
14	3/4	12	1905	"		Luten					93
40	1 1/4	12	1905	R. R.		Wells	R., Sept. 22, '06				94
14	1 1/8	[illegible]	1905	H.		Hibbard	R., Feb. 10, '06				95
18	3 x 1	24				Kahn					96
14	3/4	12	1906	H.		National Bridge Co.					97
13	3/4	[illegible]	[illegible]	"		Luten					98
	4 x [illegible]			"	25,680	Ju[illegible]son	N., Aug. 1, '01			4.75	99
							R., Apr. 20, '06				100
18				H.	14[illegible],900	Concrete Steel Co.	E.C., Sept. 2, '08			3.40	101
21	1 1/4		[illegible]	E. R.		Osborn					102
"	"		"	"							103
			1906	H.	77,000	Burr	N., May 9, '07			2.15	104
11	8" I	36	1897	"	5,640	Dean	R., Dec. 10, '98			1.84	105
20	3/4	8	1907	E. R.		Luten					106
18	4"	21	1903	H.	19,900	Courtright	N., May 12, '04			1.96	107
18	Frame	24	1896	"	75,000	Brady	R., Nov. 17, '00			5.[illegible]0	108
10			1904	F. B.			N., Apr. 6, '05				109
10	6" I	36	1896	H.		Melan Con. Co.					110
				E. R.		Mercereau					111
	3/4	6	1902		12,600					4.31	112
17	3/4	12	1907	E. R.		Luten					113
14	"	"	"	"							114
9	3/4	12	1905	H.		Luten					115
							N., Oct. 19, '99				116
10	3/4	12	1902	H.		Luten					117
	5" I		1900			Hewett					118
15	5/8	4		H.		Luten					119
14	"	6									120
13	"	8									121
26			1905	R. R.	12,000		E.C., Sept. 2, '08			6.55	122

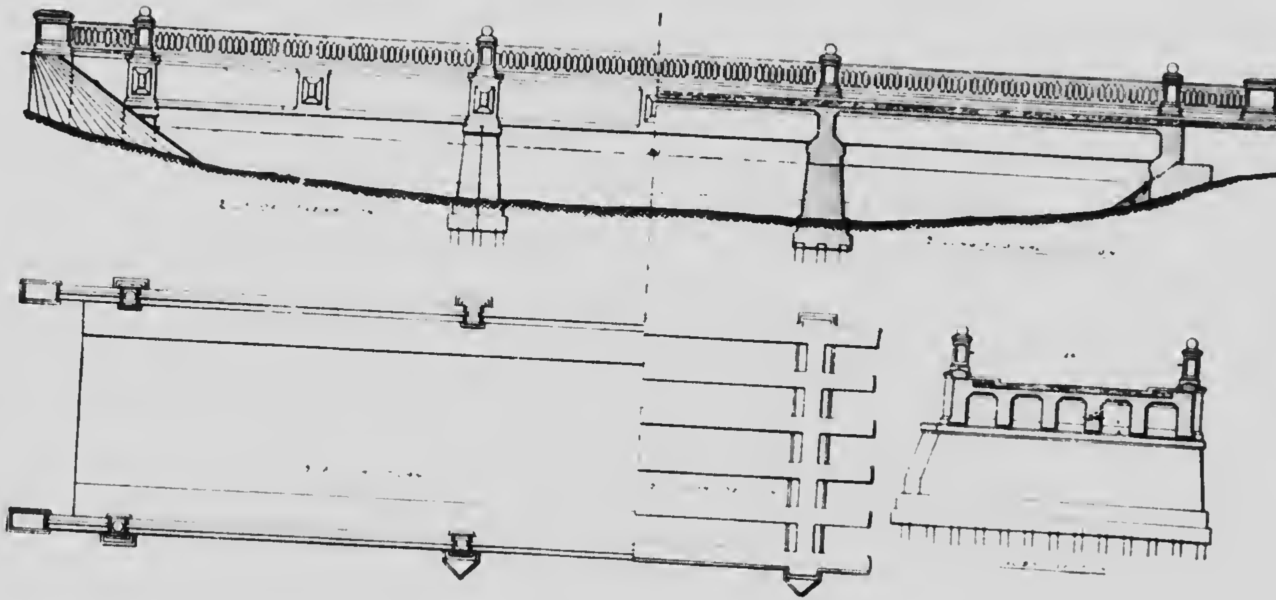


Fig. 37
THREE SPAN CONCRETE HIGHWAY BRIDGE

PART III.

Highway Beam Bridges.

Comparison of Arch and Beam. The advantages of arch bridges have already been described in Part I of this book, and original formulae have been given from which the approximate cost of concrete bridges may be determined. One of the chief merits of arch bridges is that when properly designed, they may be made beautiful in outline.

Some of the advantages of beam bridges are as follows:—(1) It is possible in a beam bridge to locate the grade of the bridge floor much lower and nearer to the high water level or other clearance line than can be done when an arch is used; (2) foundations for beam bridges may be built on soil that is more or less yielding, which cannot be done with arch bridges, unless hinges are used at the center and spring. The lateral thrust of arches on soft foundations is liable to cause serious injury to the structure, while the corresponding amount of settlement under the abutments of beam bridges produces no injurious effect.

A frequent objection to the use of beam bridges is that they are not susceptible to artistic treatment. It will be seen, however, by referring to Figures 37, 38 and 39, that beam bridges may be designed that are equally pleasing in appearance to arch bridges, and for many locations are more suitable.

In making a selection between an arch and a beam design, the chief consideration will generally

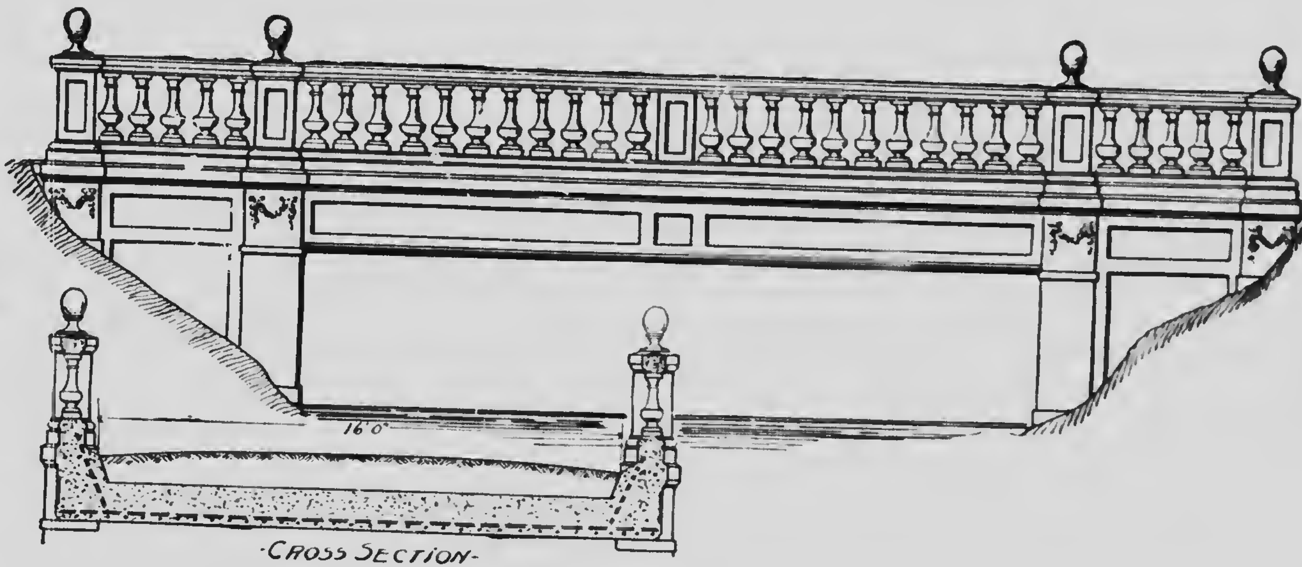


Fig. 38
CONCRETE HIGHWAY BRIDGE. SLAB CONSTRUCTION. BOTH SPAN AND ABUTMENTS REINFORCED

be their relative cost. The cost of concrete arch bridges has already been given by the formulae referred to above, and for the purpose of comparison, the costs of concrete beam bridges, in spans ranging from 4 to 40 feet in leng[illegible], are given in the tables on Figures 38 and 39. The estimated costs of these beam bridges include the filling, pavement and two lines of railing, but do not include lamps or other purely ornamental features. On Figure 38 is giv[illegible] also a table of approximate costs for concrete abutments of various heights, which estimates also include railing and pavement together with earth excavation and back filling in the abutments. These estimates will enable the designer to compare the relative cost of arch and beam bridges, and to select the form which he finds most economical.

TABLE IV

Span						Abutments	
Length	Slab Thick	Rods	Estimate: Steel Lbs.	Estimate: Conc. Cu. Yds.	Estimate: Cost	Height	Cost
Ft.	In.	In. Sq. In. cc.					
4	6	½ - 10	112	1 5	$ [illegible]	4	$ 280
6	6	$\frac{5}{8}$ - 10	232	[illegible].2	11[illegible]	5	340
8	7.5	$\frac{5}{8}$ - 8	365	[illegible].7	150	6	410
10	9	$\frac{5}{8}$ - 7	510	5.5	200	7	510
12	11	$\frac{5}{8}$ - 6½	612	8 1	250	8	650
14	12 5	¾ - 7	970	10 8	310	9	770
16	14	¾ - 6½	1160	13.9	370	10	880
18	16	¾ - 5½	1540	17.7	440	11	1030
20	18	¾ - 5	1880	22.2	510	12	1190

Beam Bridges. Concrete beam bridges have been built in spans up to 70 feet in length, but they are not generally economical for lengths exceeding 35 feet, for above this length arch bridges will cost the least.

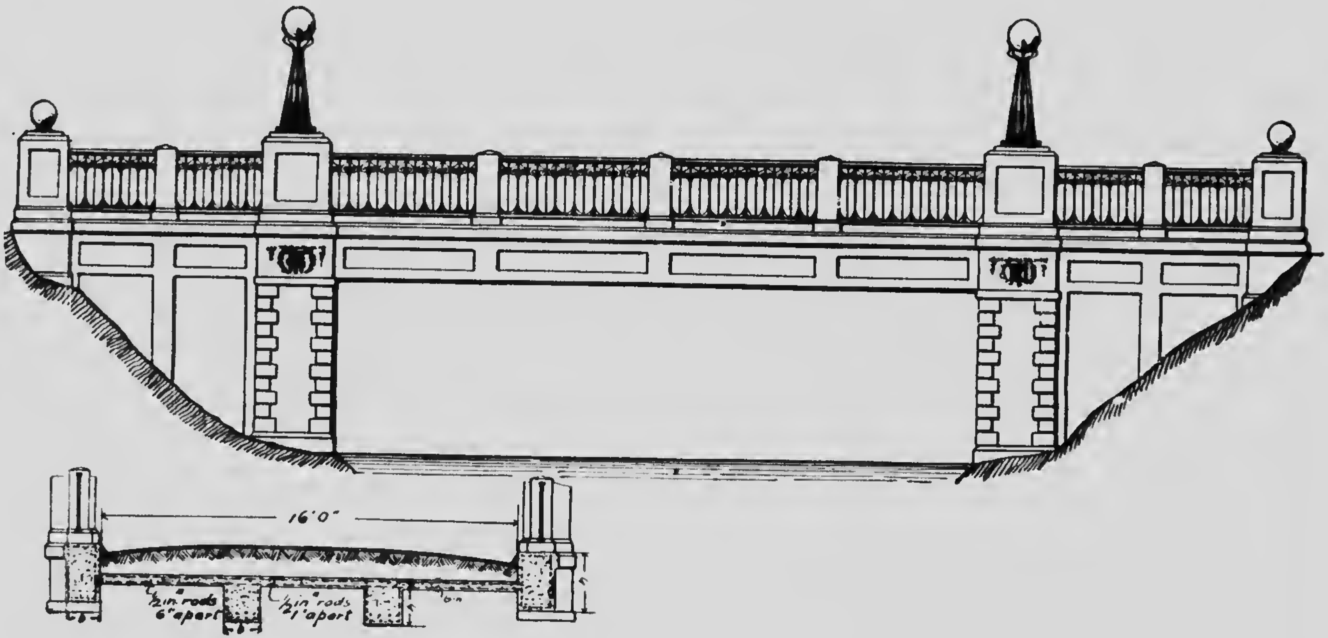


Fig. 39
CONCRETE HIGHWAY BRIDGE. BEAM CONSTRUCTION.

The economical lengths and forms for concrete beam bridges are as follows: Simple slabs are economical for spans up to 12 feet. Beam bridges similar to Figures 37 and 39, supported on parallel longitudinal beams, are economical for spans from 12 to 25 feet in length, while above 25 feet it is economy to use two lines of heavy side beams carrying light cross beams supporting the floor slab.

To determine the economic span length to use in a long bridge containing several intermediate piers,

TABLE V

Span	Side Beam		Center Beam		Estimate		
	Conc.	Rods	Conc.	Rods	Conc.	Steel	Cost
Ft.		Ft. In. Sq.		Ft. In. Sq.	Cu. Yds.	Lbs.	
8	12x20	2 - $\frac{3}{4}$	12x16	3 - $\frac{3}{4}$	3.8	656	$ 164
10	12x20	2 - $\frac{7}{8}$	12x18	3 - $\frac{3}{4}$	4.9	850	207
12	12x20	3 - $\frac{7}{8}$	12x20	4 - $\frac{3}{4}$	6.1	1160	256
14	12x20	3 - $\frac{7}{8}$	12x23	4 - $\frac{3}{4}$	7.3	1360	304
16	12x21	3 - 1	12x27	4 - $\frac{7}{8}$	8.9	1780	360
18	14x22	3 - 1	14x28	4 - $\frac{7}{8}$	11.2	2000	420
20	14x25	3 - $1\frac{1}{8}$	14x32	4 - 1	13.3	2550	490
22	14x28	3 - $1\frac{1}{8}$	14x35	4 - 1	15.5	2800	545
24	14x31	4 - 1	14x39	4 - $1\frac{1}{8}$	16.2	3350	603
26	14x34	4 - 1	14x42	4 - $1\frac{1}{8}$	20.7	3620	682
28	14x37	5 - 1	14x46	6 - 1	23.8	4460	775
30	16x38	5 - 1	16x46	6 - 1	28.2	4770	855
32	16x41	6 - 1	16x50	6 - $1\frac{1}{8}$	32.0	5800	960
34	16x44	6 - 1	16x54	6 - $1\frac{1}{8}$	35.7	6200	1090
36	16x48	7 - 1	16x57	8 - 1	39.8	7000	1140
38	16x52	7 - 1	18x58	8 - 1	45.5	7450	1244
40	16x56	7 - $1\frac{1}{8}$	18x62	8 - $1\frac{1}{8}$	51.2	9400	1400

the rule is to select such a span length that the cost of one span will be approximately equal to the cost of a pier.

Methods of Design. Single span concrete bridges of either slab or beam design must be considered non-continuous, but for a series of spans the effect of continuity in the beams may be considered. To provide for this continuity, it is customary to proportion the beams for only 80% of the maximum bending moment. The floor slabs must be protected from injury by a sufficient depth of earth filling, which is shown 12 inches on Figures 38 and 39. This provides depth enough for bedding ties of street railway tracks. A suitable pavement or wearing surface may be laid on this earth filling which may be renewed as required.

It is permissible and good practice in designing small concrete beams which are united by slabs, to consider the effect of a portion of the floor slab and to proportion the beams as T beams. Large longitudinal beams carrying floor loads directly to the piers, should be proportioned as simple beams without considering the effect of the adjoining slab. They will then have additional strength due to the presence of such slab.

The bridges shown in Figures 38 and 39 are designed for total loads of from 400 to 500 pounds per square foot of floor surface. It is customary to provide for impact either by adding a percentage to the live load or by using a factor of 2 for dead load stresses, and a corresponding factor of 4 for live load stresses.

It has been proven by numerous experiments that the adhesion of concrete to metal is sufficiently great so no additional bond is required, but as voids in the concrete are liable to occur and it is difficult to always secure the highest grade of workmanship, it is desirable to use rough bars with mechanical bond. As provision must also be made for shear by the use of inclined or bent rods and stirrup irons, it is desirable in all large beams, to use reinforcing bars which have the inclined stirrups or shear members rigidly connected to the main tension metal.

In all bridges where appearance is any consideration, the railing should be designed with care so the design may properly harmonize with the rest of the structure. Generally speaking, the balustrade that presents the best appearance on a concrete bridge is one composed of either natural or artificial stone, but it is also evident (Figure 39) that an equally artistic effect may be secured with an ornamental metal railing and stone or concrete posts and pedestals. Open balustrades are usually preferable to solid ones, not only because they are susceptible to more artistic treatment, but also because their light and open design emphasize by contrast the solidity and strength of the supporting structure beneath them. Solid balustrades are permissible chiefly for through bridges, where the concrete side girders standing above the roadway form a sufficient protection. The exposed girder surface may then be paneled or otherwise ornamented.

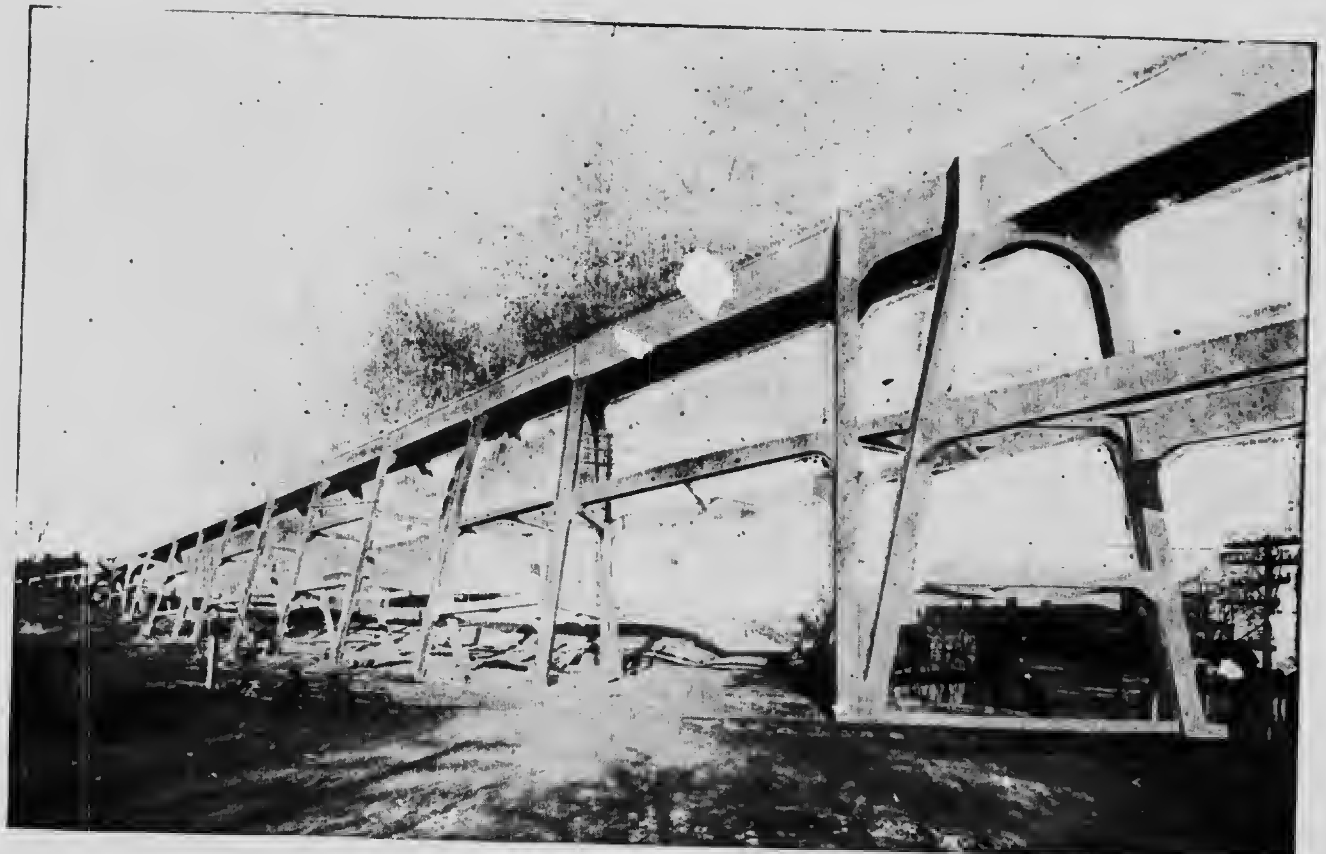


Fig. 40
CONCRETE TRESTLE, RICHMOND, VIRGINIA

PART IV.

Concrete Culverts and Trestles.

Since the introduction of reinforced concrete as a building material, many railroad companies are rebuilding their permanent bridges and culverts in concrete, either plain or reinforced. The use of reinforced concrete for culvert construction has become almost general with the railroad companies, while the building of trestles in this material is gradually coming into favor. Many old wooden structures, both of the open and the gravel deck types, are being replaced by better ones of concrete masonry. Among the railroad companies that are using reinforced concrete extensively for the construction of trestles may be mentioned the Illinois Central, the Cleveland, Cincinnati, Chicago & St. Louis (Big Four), and other branches of the New York Central Railroad system. A notable concrete trestle or viaduct that has attracted much attention is the one recently built at Richmond, Virginia, for the Richmond & Chesapeake Bay Railroad Company. This viaduct is 2,800 feet in length, and varies in height from 18 feet at the ends to 70 feet near the middle, and is shown in Figure 40. At Atlanta, Georgia, there is a reinforced concrete viaduct carrying Nelson street over the tracks of the Southern Railroad. It contains 10 spans of various lengths from 20 to 75 feet, has a total length of 480 feet, and is shown in Figure 41. The main line of the Big Four Railroad is carried for a distance of

1,200 feet across the Lawrenceville Bottoms on a reinforced concrete trestle 20 feet in height. This entire region is periodically flooded with backwater from the Ohio and Miami rivers, making it neces-

Fig. 41.
NELSON STREET VIADUCT, ATLANTA, GEORGIA.

sary to build, not only this road, but all others in the vicinity at an elevation of 30 feet above low-water level of the Ohio River.

On the following pages are designs and estimates for about 1,000 railroad culverts and trestles, and the estimated costs are given on charts shown in Figures 46, 47 and 66.

It will be seen that the trestle designs are equally suitable for culverts, and may be adapted for that purpose by increasing their width to correspond with the depth of structure below the base of rail, or to conform to the depth of the embankment. When used as culverts, abutment wing walls must be added and the nature of the foundation soil may be such as to require culvert pavement. These modifications in the trestle estimates may easily be made either for one or more openings, and adapted for either single or double box culverts.

The culvert designs are shown with a minimum depth of filling of not less than 3 feet above the concrete top. This depth is desirable not only for the purpose of distributing the live load from the engine and train wheels, but also for the purpose of forming a cushion to absorb and distribute the shock and impact from rapidly moving trains. Trestle designs G and H, Figures 64-65, are shown with a 3-foot depth of filling. It frequently occurs, however, that thin floors are necessary and only sufficient depth can be secured for the usual 15 inches of ballast. This arrangement has been shown in trestle designs A to F inclusive. (Figures 58 to 63.)

Required Size of Culvert Opening.

The most important consideration effecting the final cost of a culvert is the selection of its form and size. It frequently occurs that structures of too large a size and excessive cost are specified, when smaller ones would be ample to carry off the greatest rainfall.

The selection of the proper size of culvert is of much greater importance than any consideration of design. If a culvert costing $10,000 be specified, where a smaller one costing only $5,000 would be sufficient, the loss by such an error would evidently be $5,000. On the other hand, if the size of structure as specified be used, the engineer may by careful estimating, select a form with the required waterway, and with a cost of only $8,000. The saving in this case is only $2,000, whereas, if greater care had been given to the selection of the proper size, there might have been a saving, not only of this $2,000, but of $5,000 additional. It will be seen, therefore, that the one consideration outweighing all others in effecting the final cost is the selection of a structure with the necessary waterway.

In the State of Wyoming there are four bridges within a short distance of each other, carrying a road over the same stream. The last of these bridges to be built has two spans 65 feet in length, or 130 feet extreme. The second bridge has two 40-foot spans, and is 80 feet in length. The third has a single 60-foot span, while the fourth is an old 30-foot wooden truss, which has for fifty years proved itself sufficient to meet even flood conditions. There are, therefore, in close proximity to each other four bridges over the same stream, the longest of which is four times greater than the shortest, and the longest one was the last one built. After selecting a length of structure four times greater than required, it is possible that the engineer may have spent considerable time and thought in his endeavor to build this bridge at the least possible cost, and may have succeeded in saving a few hundred dollars on his original estimate.

A bridge 130 feet in length would cost approximately $7,000, while a 30-foot bridge would not exceed $1,500. This saving is, therefore, only a fraction of the saving that might have been effected, had a 30-foot bridge been used, which length had proved sufficient for half a century.

The most reliable data on which to base the size of a prospective structure is the high-water level of previous years. It is frequently possible to obtain such data from local records, or to determine the size from that of other bridges passing the same flow of water in the near vicinity. In the case referred to above, if the engineer, before building the 130-foot bridge, had made sufficient inquiry, he could easily have learned that a 30-foot span had carried the entire stream discharge for fifty years, and was therefore large enough for the rainfall of the future.

It is not economy to provide openings of sufficient size to carry the rainfall of freshets or cloudbursts that may not occur oftener than once in a century. For such unusual occurrences it is better to make occasional repairs than to invest additional money in larger structures than may ever be required, when such money might be drawing interest to cover the cost of an occasional repair.

Where reliable data in reference to the maximum rainfall cannot be obtained, it is customary for the railroads to build temporary wooden trestles at the proposed bridge or culvert site, and to make these trestles unnecessarily long, so there will be no doubt whatever of the openings being large enough. These temporary bridges will last from six to ten years, and during this period careful observations of the water flow may be made, and other data secured from which to determine the necessary culvert area. As the cost of these temporary trestles will not exceed $10 per lineal foot their entire cost may easily be saved by selecting the minimum required size for the permanent structure.

Where no reliable data in reference to the volume of water is obtainable, the culvert area may be computed approximately by a empirical rule known as Meyer's Formula, which is as follows:—The Required Culvert Area $= \sqrt{\text{Drainage area in acres} \times F}$, where F is a coefficient varying from unity for flat country, to 4 for rolling or mountainous country, from which rainfall is discharged at a greater velocity. The proper value for this coefficient for any particular location must be selected entirely by the judgment of the engineer.

Reinforced Concrete Box Culverts.

The following series of designs for single and double box, reinforced concrete railroad culverts, includes between 800 and 900 separate estimates, and is therefore very comprehensive and complete. The charts of comparative costs, Figures 46 and 47, show these to be more economical than any other form of culvert, excepting perhaps reinforced concrete oval culverts of the form shown in Figure 57. While arch culverts of this latter form may contain less material than box culverts of equal area, they are more difficult to build because of their curvature, even though collapsible centers be used. Several large railroad systems in America are now using arch culverts of this general form, in place of the old segmental or semicircular types, which contain more masonry in the abutments than in the arch wing.

Loads. There is much uncertainty in reference to the amount of load carried by the cover of a railroad culvert. The amount of this load depends to a great extent on the depth of the culvert top below the base of rail. The greatest load occurs when the depth of filling above it is a minimum, for then the culvert top is subjected to the entire load from the locomotive wheels and their impact. On the contrary, when the culvert is buried beneath a deep embankment, the live load and impact is so distributed and dispersed that only a part of this load goes directly to the culvert. Various writers have endeavored to show that these loads are distributed crosswise of the embankment, and slope outward from the railroad ties at the rate of one foot horizontal for every two feet vertical. The pressure on the base of these triangles varies from zero at the outer point to a maximum under the end of tie. This assumption is only an approximation, though a reasonable one. Unfortunately, however, the author of this hypothesis assumes that the earth pressures slope outward at each side, but makes no provision for similar distribution lengthwise of the embankment. It is quite evident that whatever distribution of loads does occur, must occur equally in all directions. and the assumption referred to above is therefore incorrect.

Where a culvert has a small depth of filling above it, the entire weight of such filling is then supported by the culvert, but if located at the bottom of a high embankment, the culvert then carries only a portion of the live load above it, supporting also a portion only of the earth embankment. The amount of this portion depends upon the nature of the embankment material. If this material is cemented well together, it will then tend to support itself by acting either as an arch or beam, and thereby relieving the culvert of much superimposed load. The most reasonable assumption is to consider that the culvert carries the weight of a triangular section of the embankment, the sides of which slope outward from the vertical in the ratio of one foot horizontal to two feet vertical. If the embankment material is composed of clean sand, a larger proportion of the imposed material will then be borne by the structure. In view of the uncertainty of various conditions effecting the amount of load on culvert tops, it has been determined that these loads can never exceed the values occurring under a minimum depth of earth filling.

An assumed live load on each track equivalent to Cooper's engine load E. 50, spread out by the ties, rails and ballast, produces a distributed load on the culvert top of 1,100 pounds per square foot. To this has been added impact, amounting to 50% of the live load, or 550 pounds per square foot. Adding to these the weight of ties, rails, ballast, earth filling and concrete in the culvert top, produces a total load of from 2,100 pounds per square foot for small culverts with thin slabs, to 2,400 pounds per square foot for larger spans with a greater thickness of concrete. The following box culvert tops are therefore proportioned for total loads of from 2,100 to 2,400 pounds per square foot.

From the theory of horizontal earth pressure, it is known that the thrust per square foot on an embedded vertical surface is equal to one-third of the corresponding horizontal pressure on a unit of area at the same level. This condition exists when the embankment is composed of clean, dry sand with an angle of repose of about 30 degrees. The proper amount of pressure to assume on the culvert side is therefore from 700 to 800 pounds per square foot, or one-third of the corresponding roof loads. As the sides are, however, subjected to vertical loading and impact from moving trains, the assumed side pressure has been taken at one-half of the vertical, or from 1,050 to 1,200 pounds per square foot.

On account of the liberal provision for impact, amounting to 50% of the live load, high working values have been used for concrete and metal reinforcement. A reasonably rich concrete mixture, such as 1-3-5, has an ultimate crushing value of 2,800 pounds per square inch. One-fourth this amount, or 700 pounds per square inch, is therefore assumed as a working unit for concrete, and 12,000 pounds per square inch as a working unit for reinforcing steel.

Economic Length for Slabs and Beams. There is evidently a limit where economy ceases in the use of flat slabs for supporting loads in bending, and above that limit the economical construction is a combination of beams and slabs. For the purpose of determining these economic lengths, a slab table (Table No. VI) has been prepared, giving the amount of concrete and steel and the estimated cost per square foot for spans varying in length from 4 to 24 feet, and total imposed loads of from 2,100 to 2,400 pounds per square foot.

TABLE VI

REINFORCED CONCRETE SLABS—SIMPLE SPANS

TOTAL LOADS 2100 TO 2400 LBS. PER SQUARE FOOT.

Span.	Effective Depth.	Total Depth.	Sq. Bars.	Cost per square ft. Cents.
4	6	7.5	3/4 in. 7 1/2 in apart.	30.6
6	9	10.5	7/8 " 7 " "	43.6
8	12	13.5	7/8 " 5 1/2 " "	56.0
10	15	17.0	1 " 5 1/2 " "	71.6
12	18	20.0	1 " 4 1/2 " "	85.7
14	21	23.0	1 " 4 " "	97.7
16	24	26.0	1 1/4 " 5 1/2 " "	110.2
18	28	30.5	1 1/4 " 4 1/2 " "	131.5
20	31	33.5	1 1/4 " 4 " "	146.5
22	34	37.0	1 1/4 " 3 3/4 " "	159.0
24	37	40.0	1 1/4 " 3 1/2 " "	170.0

A corresponding set of ten tables was made giving the amount of material and the estimated costs per square foot for a combination of beam and slab construction, with spans varying from 6 to 30 feet in length, and beams spaced from 6 to 18 feet apart, on centers. The cost results from these ten tables are given on the chart, Figure 42. The thickness of slabs and beams are proportioned so the stress at the outer edge will not exceed 700 pounds per square inch from dead, live and impact loads. The thicknesses were determined from the writer's original formula

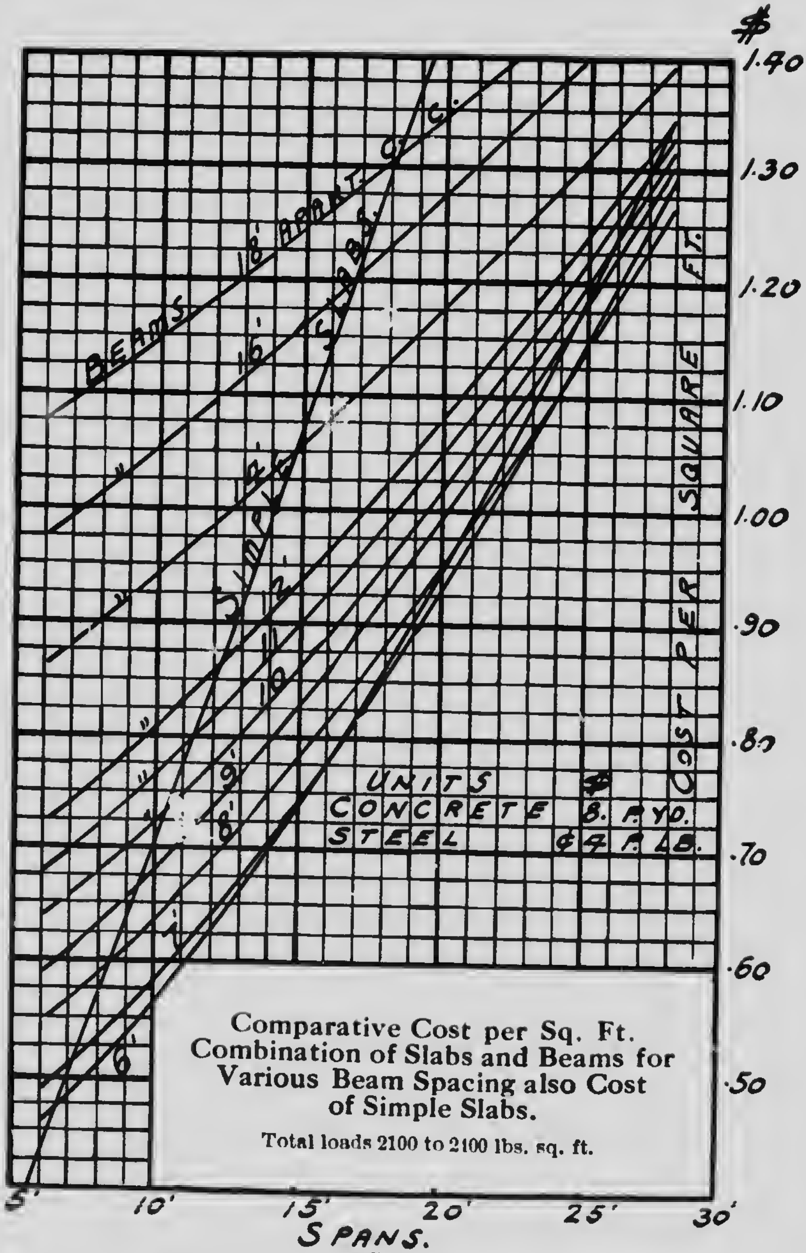
$$d^2 = \frac{M}{K}$$

where M. is the bending moment in inch pounds,
d the distance from slab top to center of tension bar, and
K a variable factor.

It is advisable to neglect the effect of continuity in proportioning slabs, even though a considerable amount doubtless exists, which would reduce the slab thickness by about 20%. Slab thicknesses are, therefore, given, as required for non-continuous beams. From the comparative cost chart, Figure 42, the following conclusions are obtained. For loads of from 2,100 to 2,400 pounds per square foot:—

Simple slabs are economical for clear spans up to 7 feet in length.

Slabs with beams 6 feet apart are economical for spans from 7 to 14 feet in length.



Comparative Cost per Sq. Ft. Combination of Slabs and Beams for Various Beam Spacing also Cost of Simple Slabs.

Total loads 2100 to 2100 lbs. sq. ft.

Fig. 42.

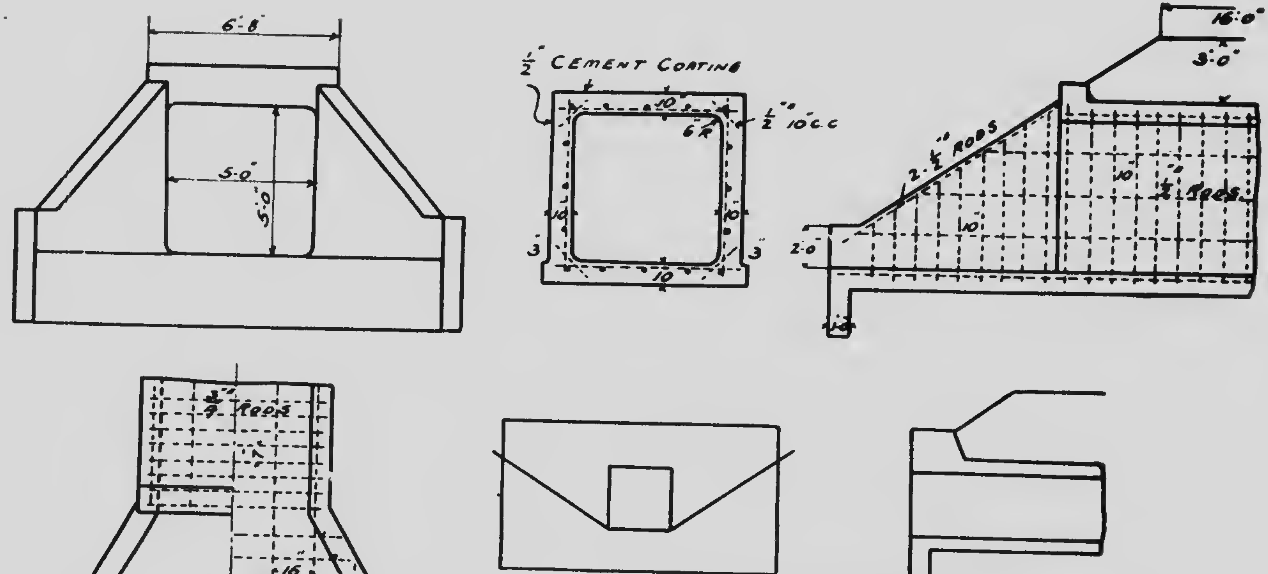
Slabs with beams 7 feet apart are economical for spans from 14 to 20 feet in length.

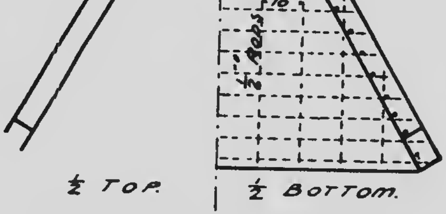
Slabs with beams 8 feet apart are economical for spans from 20 to 30 feet in length.

The comparative cost chart, Figure 42, was obtained from 130 separate estimates, and the conclusion from it is that slabs for the above loads are not economical for greater lengths than 8 feet or greater thicknesses than 12 inches.

Figures 43, 44 and 45 are typical drawings for single and double box railroad culverts for both slab, and a combination of beam and slab construction, and Tables VII, VIII, IX and X give the corresponding sizes, quantities and costs for culverts varying in area from 4 to 480 square feet. These tables give separately the quantities and cost for the two portals and for the culvert barrel per foot of length, and also the lengths and total costs of culverts for six different heights of embankment, varying from 10 to 50 feet.

The single and double slab culvert tables contain 34 different sizes each, varying from 2 feet by 2 feet to 12 feet by 12 feet for each opening, while the combined beam and slab culverts contain 30 corresponding sizes each, varying from 8 to 20 feet in width, and from 4 to 12 feet in height. The estimated costs of these culverts for banks 20, 30, 40 and 50 feet in height are shown in Figure 46. These curves represent the cost of the economic forms, which generally have openings of a greater height than width, such as 4 feet wide by 6 feet high, either



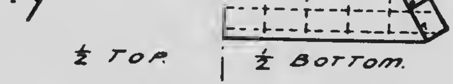


This Form of Portal for 1, 2, 3, 4, 7, 8.

This form of culvert is economical for openings up to 50 sq. ft.
Bars either medium or high tension steel.
Mechanical Bond.
Assumed loads on cover 2400 lbs. per sq. ft.
Assumed loads on sides 1200 lbs. per sq. ft.

Fig. 43.

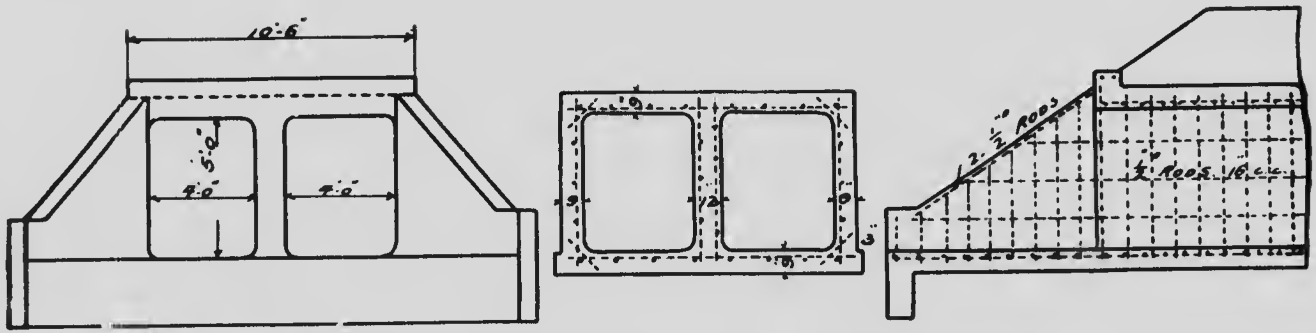
SINGLE BOX CULVERTS. SLAB CONSTRUCTION.

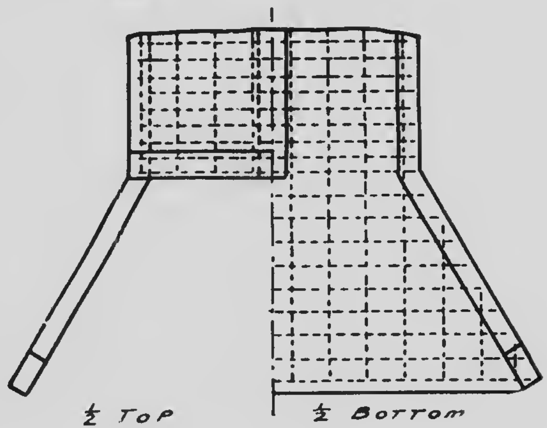


Assumed loads on sides 1200 lbs. per sq. ft.

Fig. 43.

SINGLE BOX CULVERTS. SLAB CONSTRUCTION.





This type is economical for openings from 50 to 75 square feet in cross area.

Bars either medium or high tension steel.

50,000 lbs. E. L. with mechanical bond.

Portal paving for openings up to 5 feet has solid slab.

For culverts of a greater width, portal pavements are a combination of beam and slab.

Fig. 44.

DOUBLE BOX CULVERTS. SLAB CONSTRUCTION.

TABLE VII

REINFORCED CONCRETE, SINGLE BOX, RAILROAD CULVERTS—SLAB CONSTRUCTION

TO ACCOMPANY FIGURE 43

				Top and Bottom		Sides.		Quantities, per lin. ft.			2 Portals.		
	Width, Fee.	Height, Fee.	Area, Fee.	Concrete, In.	Square Rods. C. C.	Concrete, In.	Square Rods C. C.	Concrete, C.Y.	Steel, Lbs.	Cost, $	Concrete, C.Y.	Steel, Lbs	Cost, $
1	2	2	4	6	½"—6"	6	½"—12"	.19	22	2.43	1.78	....	14
2	“	3	6	“	“ “	“	½ —10	.23	26	2.91	3.40	....	27
3	3	2	6	8	¾ —9	“	¾ —18	.28	35	3.69	3.55	....	28
4	“	3	9	“	“ “	7	“ 15	.34	43	4.50	4.00	....	32
5	“	4	12	“	“ “	8	“ 12	.42	51	5.39	4.50	200	44
6	“	5	15	“	“ “	9	“ 10	.51	62	6.55	6.00	250	58
7	4	2	8	9	¾ —7½	6	“ 18	.36	58	5.25	3.85	....	40
8	“	3	12	“	“ “	7	“ 15	.42	55	5.60	4.30	....	34
9	“	4	16	“	“ “	8	“ 12	.50	65	6.66	5.50	250	54
10	“	5	20	“	“ “	9	“ 10	.59	75	7.75	6.80	300	66
11	“	6	24	“	“ “	10	“ 8	.69	92	9.26	8.70	500	89
12	5	3	15	10	¾ —7	8	“ 15	.55	66	7.06	5.00	500	72
13	“	4	20	“	“ “	9	“ 12	.64	75	8.10	6.70	600	65
14	“	5	25	“	“ “	10	“ 10	.74	86	9.42	8.50	800	84
15	“	6	30	“	“ “	10	“ 8	.79	101	10.37	10.40	1000	107
16	“	8	40	“	“ “	12	“ 6	1.04	132	13.60	13.00	1600	136
17	6	4	24	12	⅞ —7	10	⅞ —12	.83	113	11.22	10.00	900	116
18	“	6	36	“	“ “	10	“ 10	.96	134	12.97	12.40	1200	147
19	“	8	48	“	“ “	12	“ 8	1.20	168	16.30	13.70	1800	173
20	“	10	60	“	“ “	14	“ 6	1.4[illegible]	216	20.60	15.[illegible]0	2200	208
21	8	4	32	14	⅞ —5½	12	“ 12	1.18	139	15.00	12.00	900	132
22	“	6	48	“	“ “	12	“ 10	1.33	185	18.00	22.00	1800	248
23	“	8	64	“	“ “	12	“ 8	1.47	215	20.40	30.00	2200	328
24	“	10	80	“	“ “	14	“ 6	1.76	265	24.70	43.00	3200	472
25	10	4	40	17	1 —5½	15	1 —15	1.71	241	23.30	18.00	1400	200
26	“	6	60	“	“ “	15	“ 12	1.84	266	25.40	28.00	2200	312
27	“	8	80	“	“ “	15	“ 12	2.07	282	27.90	37.00	3200	424
28	“	10	100	“	“ “	18	“ 10	2.50	324	32.90	46.00	4000	528
29	“	12	120	“	“ “	18	“ 8	2.88	381	38.30	57.00	5000	656
30	12	4	48	20	1 —4½	18	“ 15	2.33	327	31.80	23.00	1800	256
31	“	6	72	“	“ “	18	“ 12	2.56	362	34.90	37.00	2800	378
32	“	8	96	“	“ “	18	“ 12	2.78	378	37.30	52.00	4000	576
33	“	10	120	“	“ “	20	“ 10	3.00	412	40.40	68.00	5000	744
34	“	12	144	“	“ “	20	“ 8	3.40	466	45.80	82.00	6500	916

TABLE VII—Continued

REINFORCED CONCRETE, SINGLE BOX, RAILROAD CULVERTS—SLAB CONSTRUCTION

TO ACCOMPANY FIGURE 43

10 ft. Bank.		15 ft. Bank.		20 ft. Bank.		30 ft. Bank.		40 ft. Bank.		50 ft. Bank.		
Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	
39	106	54	145	69	182	99	254	129	327	159	400	1
36	122	51	175	66	219	96	305	126	392	156	479	2
39	172	54	228	69	283	99	393	129	501	159	611	3
35	189	50	257	65	324	95	458	125	593	155	727	4
32	216	47	296	62	376	92	538	122	696	152	859	5
29	248	44	346	59	444	89	643	119	838	149	1033	6
38	230	52	318	68	386	98	546	128	700	158	861	7
35	230	50	314	65	399	95	564	125	734	155	1004	8
32	267	47	366	62	466	92	664	122	864	152	1064	9
29	291	44	407	59	522	89	756	119	986	149	1216	10
26	330	41	467	56	609	86	884	116	1157	146	1439	11
34	312	49	417	64	524	94	737	124	947	154	1162	12
31	315	46	427	61	547	91	772	121	1050	151	1295	13
28	348	43	489	58	630	88	914	118	1194	148	1484	14
25	365	40	532	55	675	85	987	115	1297	145	1607	15
..	...	34	598	49	801	79	1206	109	1616	139	2016	16
31	366	46	631	61	806	91	1136	121	1486	151	1816	17
25	470	40	662	55	857	85	1247	115	1637	145	2017	18
..	...	34	725	49	762	79	1453	109	1943	139	2433	19
..	...	28	783	43	1090	73	1708	103	2328	133	2928	20
30	582	45	807	60	1032	90	1482	120	1932	150	2382	21
..	...	40	968	55	1238	85	1778	115	2318	145	2848	22
..	...	34	1020	49	1328	79	1938	109	2548	139	3148	23
..	...	28	1157	43	1522	73	2262	103	3072	133	3732	24
30	900	45	1250	60	1600	90	2300	120	3000	150	3700	25
..	...	39	1297	54	1682	84	2442	114	3192	144	3962	26
..	...	33	1339	48	1754	78	2594	108	3424	138	4244	27
..	...	27	1413	42	1908	72	2888	102	3878	132	4849	28
..	...	..	....	36	2036	66	3176	96	4316	126	5456	29
29	1166	44	1656	59	2126	89	3076	119	4026	149	4978	30
..	....	38	1698	53	2218	83	3278	113	4298	143	5348	31
..	....	32	1766	47	2326	77	3436	107	4536	137	5676	32
..	....	26	1794	41	2394	71	3604	101	4814	131	6024	33
..	....	..	....	35	2516	65	3886	95	5266	125	6636	34

TABLE VIII

REINFORCED CONCRETE, DOUBLE BOX, RAILROAD CULVERTS—SLAB CONSTRUCTION

TO ACCOMPANY FIGURE 44

					Top and Bottom		Sides.		Quantities per ft.			2 Portals.		
	Width, Feet.	Height, Feet.	Area, Feet.	Partition Thickness, In.	Concrete, In.	Square Rods C. C.	Concrete, In.	Square Rod C. C.	Concrete, C.Y.	Steel, Lbs.	Cost, $	Concrete, C.Y.	Steel, Lbs.	Cost, $
1	2	2	8	6	6	½"—6"	6	½"—12"	.32	51	4.60	2.6	0	20
2	"	3	12	"	"	" "	6	" 10	.38	43	4.72	3.9	0	31
3	3	3	18	9	8	¾ —9	7	¾ —15	.61	76	7.94	4.9	0	39
4	"	4	24	"	"	" "	8	" 12	.72	89	9.30	5.3	200	50
5	"	5	30	"	"	" "	9	" 10	.84	104	10.85	7.0	250	66
6	4	3	24	12	9	¾ —7½	7	" 15	.82	101	10.52	5.5	0	44
7	"	4	32	"	"	" "	8	" 12	.94	114	12.05	6.6	250	62
8	"	5	40	"	"	" "	9	" 10	1.06	129	13.65	8.0	300	76
9	"	6	48	"	"	" "	10	" 8	1.19	152	15.60	10.0	500	100
10	5	3	30	12	10	¾ —7	8	" 15	1.06	123	13.50	6.7	600	77
11	"	4	40	"	"	" "	9	" 12	1.21	135	15.10	8.2	700	89
12	"	5	50	"	"	" "	10	" 10	1.35	151	16.81	9.7	800	109
13	"	6	60	"	"	" "	11	" 8	1.51	174	19.02	12.6	1000	140
14	"	8	80	"	"	" "	12	" 6	1.79	232	23.65	15.0	1600	184
15	6	4	48	12	12	⅞ —7	10	⅞ —12	1.62	206	21.10	11.0	900	124
16	"	6	72	"	"	" "	11	" 10	1.89	239	24.50	18.0	1200	192
17	"	8	96	"	"	" "	12	" 8	2.20	285	29.00	24.0	1800	264
18	"	10	120	"	"	" "	14	" 6	2.62	353	35.50	30.0	2400	336
19	8	4	64	15	14	⅞ —5½	12	" 12	2.35	306	30.90	14.0	900	148
20	"	6	96	"	"	" "	12	" 10	2.65	338	33.70	20.0	2000	246
21	"	8	128	"	"	" "	12	" 8	2.95	384	38.90	27.0	2700	304
22	"	10	160	"	"	" "	14	" 6	3.36	457	45.30	37.0	3200	424
23	10	4	80	15	17	1 —5½	15	1 —15	3.36	462	45.50	15.0	1400	176
24	"	6	120	"	"	" "	15	" 12	3.75	498	49.60	25.0	2200	288
25	"	8	160	"	"	" "	15	" 12	4.11	522	53.50	37.0	3200	424
26	"	10	200	"	"	" "	18	" 10	4.70	577	60.50	47.0	4000	536
27	"	12	240	"	"	" "	18	" 8	5.10	660	66.80	62.0	5000	696
28	12	4	96	18	20	1 —4½	18	" 15	4.62	647	62.80	20.0	1800	232
29	"	6	144	"	"	" "	18	" 12	5.08	685	67.60	35.0	2800	392
30	"	8	192	"	"	" "	18	" 12	5.50	708	71.70	45.0	4000	520
31	"	10	240	"	"	" "	20	" 10	6.02	759	78.00	62.0	5000	696
32	"	12	288	"	"	" "	20	" 8	6.55	838	85.00	75.0	6500	860

TABLE VIII—Continued

REINFORCED CONCRETE, DOUBLE BOX, RAILROAD CULVERTS—SLAB CONSTRUCTION

TO ACCOMPANY FIGURE 44

10 ft. B'k		15 ft. Bank.		20 ft. Bank.		30 ft. Bank.		40 ft. Bank.		50 ft. Bank.		
Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	
39	200	54	269	66	335	99	475	139	610	159	750	1
36	201	51	272	66	341	96	482	126	623	156	763	2
35	315	50	434	65	551	95	789	125	1029	155	1269	3
32	347	47	486	62	625	92	905	122	1180	152	1470	4
29	381	44	542	59	706	89	1031	119	1356	149	1686	5
35	414	50	569	65	729	95	1064	125	1384	155	1674	6
32	447	47	627	62	807	92	1172	122	1522	152	1882	7
29	471	44	676	59	881	89	1286	119	1696	149	2106	8
26	505	41	740	56	970	86	1440	116	1900	146	2370	9
34	535	49	729	64	939	94	1337	124	1747	154	2147	10
31	555	46	780	61	1009	91	1459	121	1909	151	2359	11
28	579	44	849	58	1079	88	1589	118	2089	148	2579	12
25	622	40	906	55	1196	85	1766	115	2316	145	2906	13
...	...	34	984	49	1344	79	2044	109	2754	139	3465	14
31	779	46	1094	61	1414	91	2044	121	2684	151	3324	15
25	802	40	1172	55	1532	85	2262	115	2992	145	3712	16
...	...	34	1244	49	1684	79	2544	109	3414	139	4264	17
...	...	28	1326	43	1856	73	2916	103	4086	133	5036	18
30	1073	45	1528	60	1998	90	2918	120	3848	150	4768	19
...	...	40	1586	55	2096	85	3106	115	4116	145	5096	20
...	...	34	1624	49	2204	79	3364	109	4544	139	5654	21
...	...	28	1684	43	2364	73	3724	103	5074	133	6424	22
30	1536	45	2226	60	2896	90	4276	120	5596	150	6976	23
...	...	39	2218	54	2968	84	4458	114	5938	144	7408	24
...	...	33	2184	48	2984	78	4584	108	6174	138	7774	25
...	...	27	2166	42	3066	72	4886	102	6786	132	8536	26
...	...	..	....	36	3096	66	5096	96	7098	126	9096	27
29	2052	44	2982	59	3932	89	5812	119	7882	149	9532	28
...	...	38	2950	53	3960	83	5992	113	7990	143	10040	29
...	...	32	2800	47	3920	77	6020	107	8170	137	10320	30
...	...	26	2716	41	3896	71	6216	101	8596	131	10896	31
...	...	..	....	35	3830	65	6370	96	8960	125	11460	32

TABLE IX

REINFORCED CONCRETE, SINGLE BOX, RAILROAD CULVERTS—BEAM AND SLAB CONSTRUCTION

TO ACCOMPANY FIGURE 45

				Top and Bottom			Sides.			Per Lineal ft.			2 Portals.		
	Width, Feet.	Height, Feet.	Area, Feet.	b	h	Square Rods.	b′	h′	Square Rods.	Concrete, Yds	Steel, Lbs.	Cost, $	Concrete, Yds	Steel, Lbs.	Cost, $
1	8	4	32	12	31	4—1″	12	12	3—3/4″	.98	177	14.6	9.4	1250	125
2	“	6	48	“	“	“	“	17	4— “	1.10	206	16.9	15.7	2060	206
3	“	8	64	“	“	“	“	22	4— 7/8	1.27	240	19.7	23.2	3050	305
4	“	10	80	“	“	“	“	27	4—1	1.48	278	22.9	32.0	4220	432
5	10	4	40	14	36	4—1 1/8	14	13	3— 3/4	1.26	223	19.1	13.5	1800	180
6	“	6	60	“	“	“	“	16	4— 3/4	1.37	260	21.3	21.2	2800	280
7	“	8	80	“	“	“	“	21	4— 7/8	1.56	296	24.3	30.6	4050	405
8	“	10	100	“	“	“	“	26	4—1	1.78	336	27.6	41.5	5500	550
9	“	12	120	“	“	“	“	30	4—1 1/8	2.00	381	31.1	53.5	7100	710
10	12	4	48	16	40	4—1 1/4	16	14	3— 3/4	[illegible]	[illegible]	23.8	16.5	2200	220
11	“	6	72	“	“	“	“	15	4— 3/4	[illegible]	[illegible]	26.2	25.8	3400	340
12	“	8	96	“	“	“	“	19	4— 7/8	1.88	357	29.2	36.5	4800	485
13	“	10	120	“	“	“	“	24	4—1	2.10	403	32.9	52.5	7000	700
14	“	12	144	“	“	“	“	29	4—1 1/8	2.37	452	37.1	64.0	8500	850
15	14	6	84	18	50	5—1 3/8	18	19	4— 7/8	2.35	388	34.3	40.0	5300	530
16	“	8	112	“	“	“	“	21	4—1	2.55	456	38.5	55.0	7200	730
17	“	10	140	“	“	“	“	26	4—1 1/8	2.84	504	42.8	72.0	9600	960
18	“	12	168	“	“	“	“	31	5—1 1/8	3.08	559	46.9	91.0	12100	1210
19	16	6	96	20	54	6—1 3/8	20	20	4— 7/8	2.82	491	42.2	49.0	6500	650
20	“	8	128	“	“	“	“	20	4—1	2.98	540	44.4	64.0	8500	850
21	“	10	160	“	“	“	“	25	4—1 1/8	3.25	589	48.4	84.0	11100	1110
22	“	12	192	“	“	“	“	29	5—1 1/8	3.50	647	53.7	106.0	14000	1400
23	18	6	108	22	58	7—1 3/8	22	20	4— 7/8	3.32	572	49.4	60.0	8000	800
24	“	8	144	“	“	“	“	20	4—1	3.47	624	52.8	66.0	8800	880
25	“	10	180	“	“	“	“	24	4—1 1/8	3.74	677	56.0	99.0	13100	1310
26	“	12	216	“	“	“	“	28	5—1 1/8	4.02	745	61.8	122.0	16200	1620
27	20	6	120	24	62	8—1 3/8	24	20	4— 7/8	3.88	662	58.7	71.0	9400	940
28	“	8	160	“	“	“	“	20	4—1	4.05	724	61.2	93.0	12300	1230
29	“	10	200	“	“	“	“	23	4—1 1/8	4.30	776	65.4	117.0	15500	1550
30	“	12	240	“	“	“	“	27	5—1 1/8	4.60	842	70.2	144.0	19160	1910

TABLE IX—Continued

REINFORCED CONCRETE, SINGLE BOX, RAILROAD CULVERTS—BEAM AND SLAB CONSTRUCTION

TO ACCOMPANY FIGURE 45

10 ft. Bank		15 ft. Bank.		20 ft. Bank.		30 ft. Bank.		40 ft. Bank.		50 ft. Bank.		
Length.	Cost, $	Length.	Cost.	Length	Cost, $	Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	
27	535	42	735	57	973	87	1403	117	1843	147	2283	1
..	...	36	816	51	1066	81	1566	111	2076	141	2576	2
..	...	30	895	45	1190	75	1775	105	2365	135	2955	3
..	...	..	....	39	1317	69	2002	99	2692	129	3372	4
25	656	40	940	55	1230	85	1800	115	2360	145	2940	5
..	...	34	1005	49	1320	79	1960	109	2600	139	3230	6
..	...	28	1085	43	145[illegible]	73	2175	103	2905	133	3625	7
..	...	..	....	37	1570	67	2400	97	3220	127	4050	8
..	...	..	....	31	1675	61	2610	91	3530	121	4510	9
25	820	40	1170	55	1530	85	2240	115	2950	145	3670	10
..	...	34	1230	49	1620	79	2500	109	3190	139	3980	11
..	...	28	1300	43	1735	73	2605	103	3485	133	4355	12
..	...	..	....	37	1910	67	2900	97	3870	127	4850	13
..	...	..	....	31	2000	61	3110	91	4210	121	5350	14
..	...	31	1590	46	2110	76	3130	106	4150	136	5180	15
..	...	25	1690	40	2270	70	3430	100	4580	130	5730	16
..	...	..	....	34	2420	64	3710	94	5000	124	6260	17
..	...	..	....	28	2520	58	3920	88	5310	118	6710	18
..	...	30	1920	45	2550	75	3810	105	5070	135	6350	19
..	...	..	....	39	2570	69	3900	99	5250	129	6550	20
..	...	..	....	33	2700	63	4080	93	5610	123	7030	21
..	...	..	....	27	2850	57	4450	87	6050	117	7650	22
..	...	28	2180	43	2920	73	4400	103	5850	133	7350	23
..	...	..	....	37	2830	67	4410	97	5980	127	7530	24
..	...	..	....	31	3050	61	4720	91	6410	121	8060	25
..	...	..	....	25	3160	55	5020	85	6870	115	8720	26
..	...	28	2580	43	3460	73	5200	103	6950	133	8740	27
..	...	..	....	37	3500	67	5330	97	7180	127	8980	28
..	...	..	....	31	3570	61	5536	91	7450	121	9450	29
..	...	..	....	25	3660	55	5760	85	7860	115	9910	30

TABLE X

REINFORCED CONCRETE, DOUBLE BOX, RAILROAD CULVERTS—BEAM AND SLAB CONSTRUCTION

TO ACCOMPANY FIGURE 45

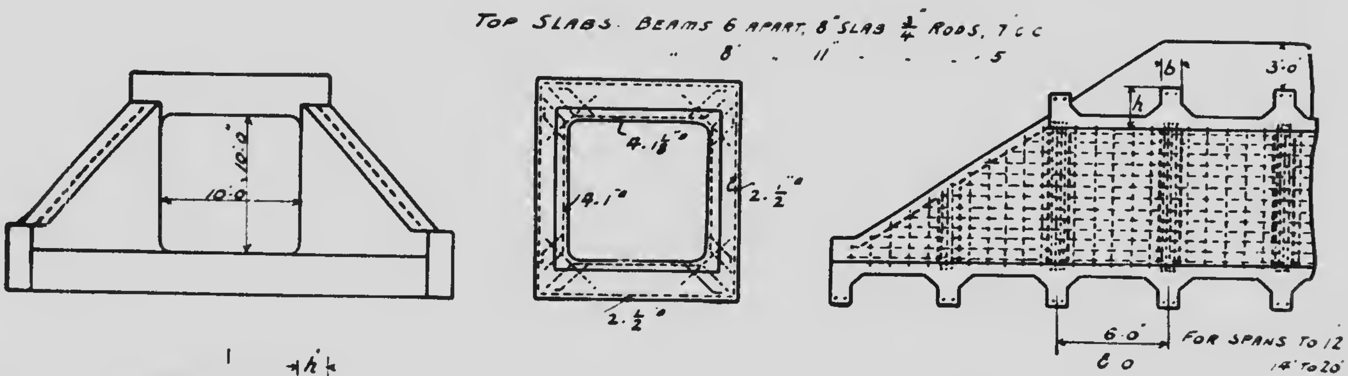
	Width, Feet.	Height, Feet.	Area, Feet	Partition Thickness, In.	Top and Bottom.			Sides.			Per Lin. ft.		
					b	h	Rods.	b'	h'	Rods.	Concrete, Yds.	Steel, Lbs.	Cost, $
1	8	4	64	15	12	29	4—1"	12	12	3—¾"	1.80	320	26.8
2	"	6	96	"	"	"	" "	"	17	4— "	2.05	348	30.3
3	"	8	128	"	"	"	" "	"	22	4—⅞	2.32	388	33.6
4	"	10	160	"	"	"	" "	"	27	4—1	2.61	433	37.9
5	10	4	80	15	14	33	4—1⅛	14	13	3—¾	2.08	405	33.0
6	"	6	120	"	"	"	" "	"	16	4—¾	2.33	446	36.3
7	"	8	160	"	"	"	" "	"	21	4—⅞	2.62	489	40.1
8	"	10	200	"	"	"	" "	"	26	4—1	2.91	537	44.4
9	"	12	240	"	"	"	" "	"	30	4—1⅛	3.42	587	50.8
10	12	4	96	18	16	37	4—1¼	16	14	3—¾	3.00	510	44.4
11	"	6	144	"	"	"	" "	"	15	4—¾	3.25	555	48.0
12	"	8	192	"	"	"	" "	"	19	4—⅞	3.52	601	52.2
13	"	10	240	"	"	"	" "	"	24	4—1	3.86	656	57.2
14	"	12	288	"	"	"	" "	"	29	4—1⅛	4.21	710	62.0
15	14	6	168	18	18	47	5—1⅜	18	19	4—⅞	4.38	767	65.5
16	"	8	224	"	"	"	" "	"	21	4—1	4.71	825	70.6
17	"	10	280	"	"	"	" "	"	26	4—1⅛	5.00	881	75.0
18	"	12	236	"	"	"	" "	"	31	5—1⅛	5.38	946	80.7
19	16	6	192	20	20	50	6—1⅜	20	20	4—⅞	5.28	911	78.2
20	"	8	25	"	"	"	" "	"	20	4—1	5.51	963	82.1
21	"	10	320	"	"	"	" "	"	25	4—1⅛	5.90	1024	88.0
22	"	12	384	"	"	"	" "	"	29	5—1⅛	6.30	1088	94.0
23	18	6	216	22	22	54	7—1⅜	22	20	4—⅞	6.25	1069	93.0
24	"	8	288	"	"	"	" "	"	20	4—1	6.55	1130	97.0
25	"	10	360	"	"	"	" "	"	24	4—1⅛	6.95	1195	102.5
26	"	12	432	"	"	"	" "	"	28	5—1⅛	7.38	1263	109.2
27	20	6	240	24	24	58	8—1⅜	24	20	4—⅞	7.40	1251	109.0
28	"	8	320	"	"	"	" "	"	20	4—1	7.70	1322	114.0
29	"	10	400	"	"	"	" "	"	23	4—1⅛	8.10	1395	119.5
30	"	12	480	"	"	"	" "	"	27	5—1⅛	8.50	1467	126.5

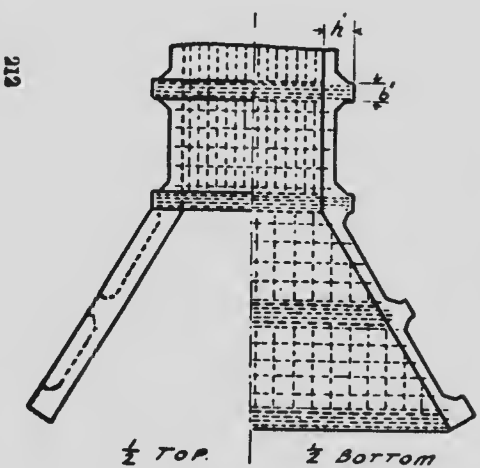
TABLE X—Continued

REINFORCED CONCRETE, DOUBLE BOX, RAILROAD CULVERTS—BEAM AND SLAB CONSTRUCTION

TO ACCOMPANY FIGURE 45

2 Portals.			10 ft. Bank.		15 ft. Bank.		20 ft. Bank.		30 ft. Bank.		40 ft. Bank.		50 ft. Bank.		
Concrete, Yds.	Steel, Lbs.	Cost, $	Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	Length.	Cost, $	
14.3	1910	191	27	915	42	1350	57	1720	87	2530	117	3330	147	4140	1
22.6	3000	300	..	...	36	1380	51	1870	81	2750	111	3650	141	4560	2
32.4	4300	430	..	...	30	1440	45	1940	75	2940	105	3940	135	4940	3
43.0	5700	570	..	...	..	....	39	2040	69	3180	99	4320	129	5490	4
20.4	2720	272	25	1091	40	1590	55	2082	85	3072	115	4040	145	5022	5
31.5	4200	420	..	...	34	1660	49	2200	79	3280	109	1370	139	5470	6
43.2	5720	572	..	...	28	1692	43	2292	73	3492	103	4692	133	5892	7
53.5	7100	710	..	...	..	....	37	2340	67	3660	97	5010	127	6310	8
72.0	9500	950	..	...	..	....	31	2520	61	4020	91	5550	121	7050	9
26.0	3450	345	25	1450	40	2075	55	2785	85	4105	115	5445	145	6745	10
38.6	5100	510	..	...	34	2140	49	2560	79	4310	109	5760	139	7160	11
53.5	7100	710	..	...	28	2170	43	2950	73	4510	103	6060	133	7660	12
73.0	9700	970	..	...	..	....	37	3090	67	4790	97	6480	127	8170	13
87.0	1160	116	..	...	..	....	31	3070	61	4920	91	6760	121	8660	14
54.0	7200	720	..	...	31	2750	46	3720	76	5680	106	7640	136	9620	15
71.0	9400	940	..	...	25	2700	40	3760	70	5890	100	8000	130	10140	16
91.0	12100	1210	..	...	..	....	34	3760	64	6010	94	8230	124	10510	17
111.0	14700	1470	..	...	..	....	28	3720	58	6120	88	8520	118	10920	18
72.0	9600	960	..	...	30	3310	45	4480	75	6860	105	9210	135	11560	19
92.0	12200	1220	..	...	..	....	39	4420	69	6870	99	9320	129	11820	20
117.0	15500	1550	..	...	..	....	33	4450	63	7050	93	9700	123	12350	21
143.0	19000	1900	..	...	..	....	27	4430	57	7250	87	10050	117	12900	22
91.0	12100	1210	..	...	28	3810	43	5210	73	8010	103	10810	133	13510	23
105.0	13900	1390	..	...	..	....	37	4960	67	7890	97	10790	127	13690	24
145.0	19200	1920	..	...	..	....	31	5100	61	8170	91	11320	121	14320	25
175.0	23200	2320	..	...	..	....	25	5050	55	8320	85	11620	115	14820	26
98.0	13000	1300	..	...	28	4350	43	6000	73	9250	103	12500	133	15800	27
142.0	18800	1880	..	...	..	....	37	6090	67	9480	97	12980	127	16380	28
176.0	23400	2340	..	...	..	....	31	6040	61	9590	91	13240	121	16740	29
211.0	28100	2810	..	...	..	....	25	5970	55	9760	85	13510	115	17310	30





Side Slabs

Beams 6 feet apart, 6 inch slabs, ⅜ inch rods, 9 inch c. c.
Beams 8 feet apart, 8 inch slabs, ¾ inch rods, 6 inch c. c.

This form in single box is economical for openings from 75 to 120 square feet. Above 120 square feet, double box economical.

Fig. 45.
SINGLE BOX CULVERTS. BEAM AND SLAB CONSTRUCTION.

Cost of Reinforced Concrete R. R. Culverts.
Single Track Banks.
Form—Rectangular Box.

Base Prices.

Concrete in place $8.00 per yd.

Steel in place 4c per lb.

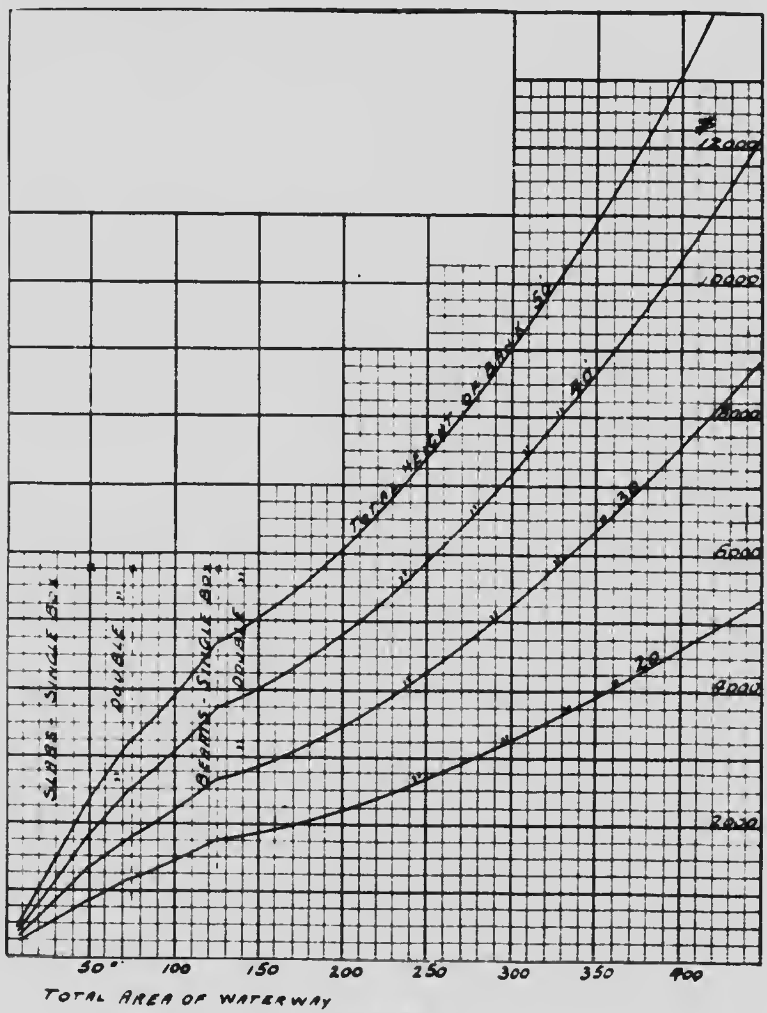


Fig. 46.

double or single. Culverts of these forms cost less for any given area than if made with wider and shallower openings. The reason for this is due to the fact that for wider openings the thickness of cover slabs increase more rapidly than the thickness of side walls.

From the comparative cost chart. Figure 46. the following conclusions are deduced:

Single box slab culverts are economical for areas up to 50 square feet.

Double box slab culverts are economical for areas up to 75 square feet.

Single box beam and slab culverts are economical for areas up to 125 square feet.

Double box beam and slab culverts are economical for areas above 125 square feet.

Wide, flat culverts cost somewhat more than narrow and higher ones of the same area, but they are more effective and offer less resistance to the free flow of water. For a bank of any given height, the low culvert will have a longer barrel than a higher one, though this will be offset to some extent by the shorter length of wing walls. There is little or no economy in reducing the length of culvert barrel by using high end parapet retaining walls, as material thus used might better be employed in increasing the length of culvert barrel, thereby causing shorter wing walls.

In proportioning the thickness of wing walls, when these wings are placed at a considerable angle to the culvert face, the stability of the wing wall is thereby greatly increased, and it is generally safe to make the base thickness of the wings near their connection to the tment from 20% to 25% of the wing wall height. Towards the ends where the wings receive no support from the culvert sides, the width or thickness of wing wall base should then be 40% of the unsupported height.

The size of beams and slabs given in Tables VII, VIII, IX and X are for culvert barrels subjected to the loads specified, which occurs at and near the center of the embankment. For long culverts, these sizes may be reduced towards the ends where the loads are somewhat less than at the middle.

Where the nature of the soil will permit, some economy may result by omitting the reinforced concrete pavement slab, and substituting offset footings under the side walls, as shown on the concrete trestle plans, Figures 58 to 65 inclusive, using cobble stone pavement, if required.

There is less probability of debris and drift collecting when the culvert bottom is curved or dished out at the center, than when built flat or horizontal between the two side walls. Box culvert corners should be braced with straight or curved corner fillets, reinforced with diagonal rods, as shown on the typical drawings.

It is unnecessary to increase the thickness of side walls from the top to the bottom, excepting perhaps for high culverts, and even then since the condition of earth pressure on the side walls is uncertain, any effort at ultra-refinement is unnecessary.

Waterproofing should be used on the exterior surfaces of the roof and sides to prevent drainage water from soaking into the concrete. The adhesion of concrete to steel is decreased about 10C when the concrete is continuously water soaked, and this decrease can be avoided by finishing the outer surface of the top and sides with a coating of neat cement or other waterproof material.

Comparative Costs of Culverts of Various Forms. Figure 47 shows the comparative costs of reinforced concrete box railroad culverts compared with corresponding costs of culverts of other forms. The chart gives the total cost of culverts for an embankment 20 feet in height, and for cross-sectional areas varying from 5 to 200 square feet.

The new reinforced concrete box culverts, the cost of which are shown by the heavy line number 10, are more economical than any other permanent culverts, and cost but little more than wooden box culverts. They range in cost from 30 to 50 cents per square foot of sectional area.

The various culverts, the costs of which are shown in Figure 47 by lines, are as follows:—

No. 1 gives the cost of standard cast iron pipe culverts, which are suitable only for small openings, and while they can be quickly placed, and sometimes inserted inside of worn-out temporary wooden box culverts, they are not economical.

No. 2 are reinforced concrete box culverts with bottoms, similar to those in use on the Union Pacific and Southern Pacific railroads.

Comparative Costs of Culverts.

SINGLE TRACK RAILROAD. HEIGHT OF BANK 20 FEET.

No. 1 **Cast iron pipe.**
" 2 **Reinforced concrete box, with bottoms.**
" 3 **Rail top concrete box.**
" 4 **Reinforced concrete arch.**
" 5 **Solid concrete arch.**
" 6 **Stone arch, Baker's standard.**
" 7 **Reinforced concrete box, no bottoms.**
" 8 **Rubble stone box.**
" 9 **Wood box.**
" 10 **Reinforced concrete box, new standard.**

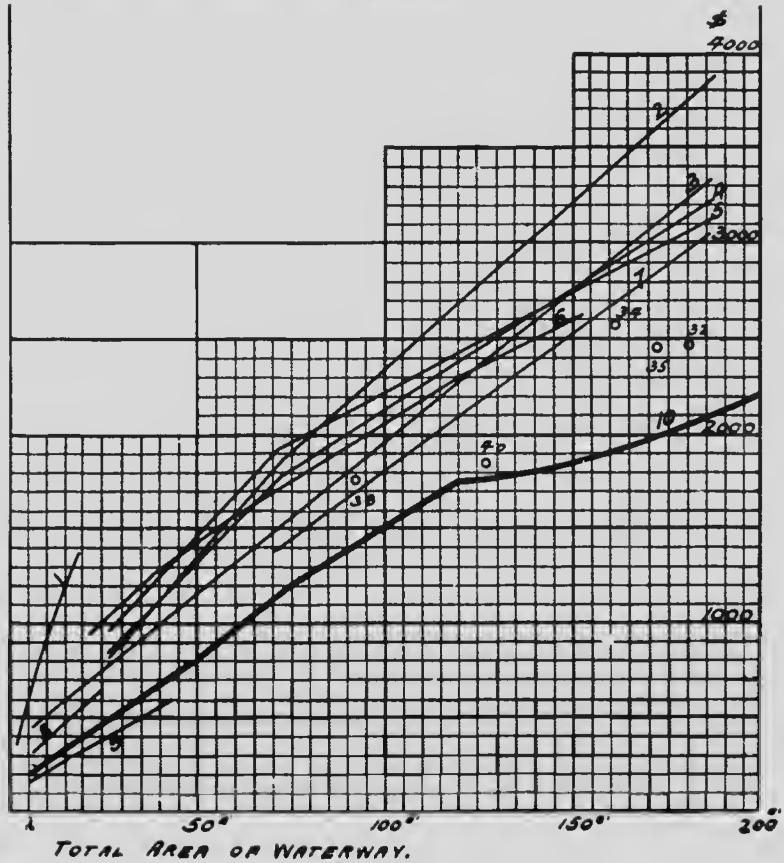


Fig. 47.

No. 3 are concrete rail top culverts having slabs 15 to 18 inches thick, and reinforced with rails spaced 18 inches apart for a 6-foot span, 10 inches apart for an 8-foot span, and 6 inches apart for a 10-foot span.

No. 4 are reinforced concrete arches, similar to those in use on the above named railroads.

No. 5 are concrete arches without reinforcement.

No. 6 are segmental stone arch culverts as proposed by Mr. Baker in his book on Masonry Construction.

No. 7 are reinforced concrete box culverts, similar to No. 2, excepting that they are without bottoms

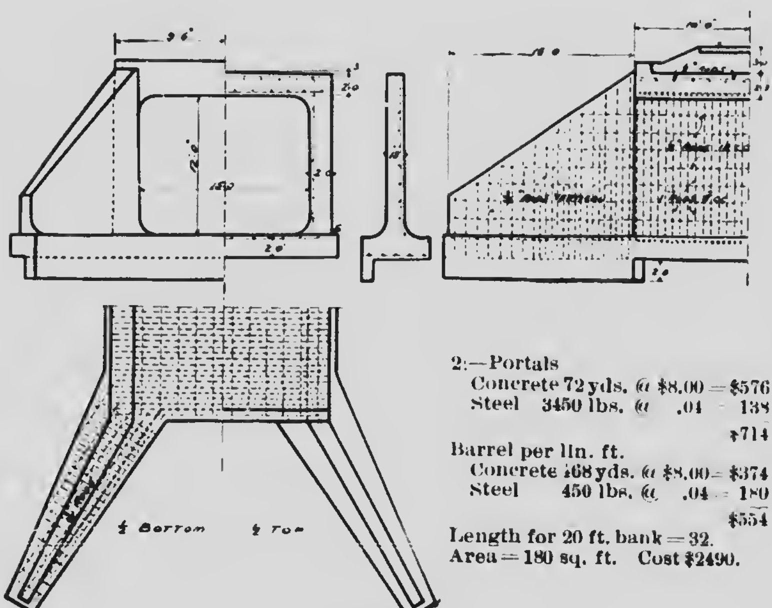


Fig. 48.

and cost proportionately less. They have offset footings under the side walls.

No. 8 are rubble stone box culverts, the kind most commonly used by the railroads until recently, for small openings.

No. 9 are wooden box culverts, and while they are not permanent, they have the merit of being the least expensive of all.

No. 10 are the new standard reinforced concrete box culverts, as shown in Figures 43, 44 and 45, the quantities and cost of which are given in Tables VII, VIII, IX and X.

An actual cost record for building a 4-foot concrete arch culvert under a railroad embankment in Idaho, during the thirty days from June 5th to July 11th, 1903, is as follows:—

Foundations contain 111 yards, and cost......$5.00 per yd.
Upper part contains 137 yards, and cost.. .....7.00 per yd.
Average cost about .... 6.00 per yd.
Cost of whole culvert per cu. yard of concrete.10.00 per yd.
Portland Cement used, 272 barrels. Cost...... 2.70 per bbl.
Foreman paid........$150.00 per month.
1 Finisher paid........... 3.00 per day
Laborers paid............ 2.00 per day
4 Carpenters paid......... 3.00 per day
Labor cost $1723.00
Material cost 830.00

Total $2553.00

Concrete made entirely from sand and gravel at railroad company's pit, without any broken stone.

Other Common Culvert Forms. Figures 48 to 57 inclusive, show other forms of culverts, and Table

XI contains their estimated quantities and costs. For the purpose of comparing these with others, the costs have been estimated for lengths required under a 20-foot embankment, and these costs are given in Figure 47, together with their corresponding numbers. They vary in cost from 26 to 36 cents per square foot of section area, for each lineal foot of culvert.

Figure 48 is a reinforced concrete box culvert 12 feet high and 15 feet wide, with rod reinforcement, similar to the new single box slab culvert. For so large a section area, the slab type is not economical.

Figure 49 is a reinforced concrete box culvert of combined beam and slab construction, 12 feet high

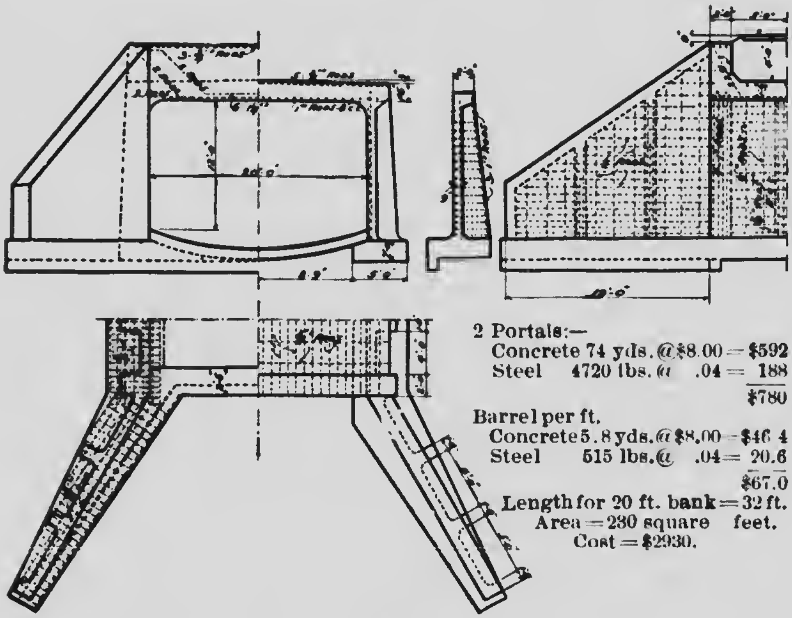
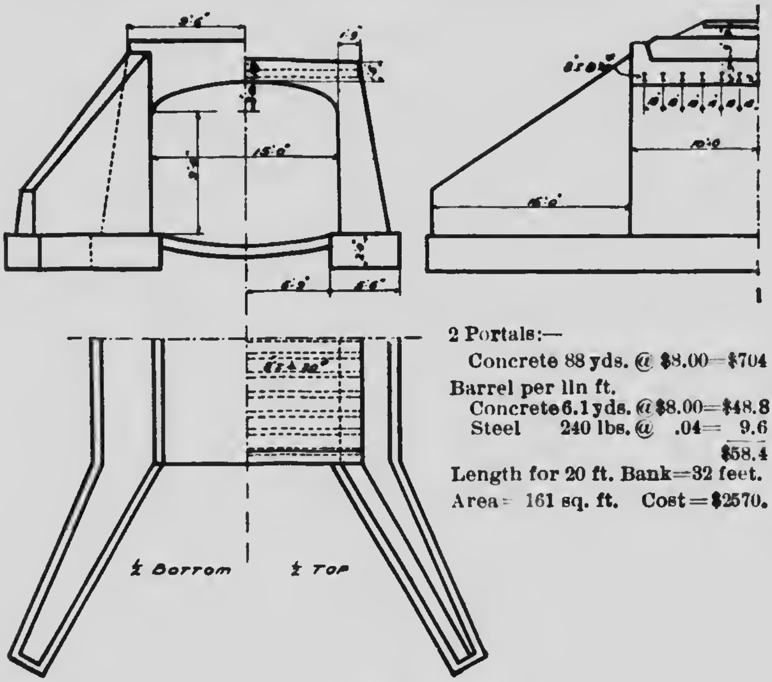


Fig. 49.

and 20 feet wide. For an area of this size a more economical form is secured by using a double box of the same general type.

Figure 50 is a beam top culvert, 12 feet high and 15 feet wide. The culvert top is arched 3 feet and the arch strength is considered when proportioning

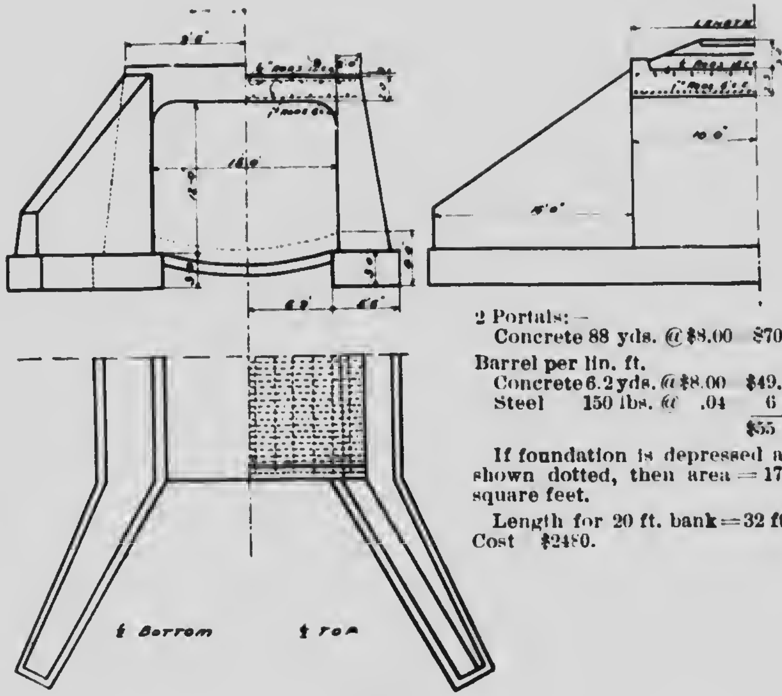


2 Portals:—
Concrete 88 yds. @ $8.00=$704
Barrel per lin ft.
Concrete 6.1 yds. @ $8.00=$48.8
Steel 240 lbs. @ .04= 9.6
$58.4
Length for 20 ft. Bank=32 feet.
Area= 161 sq. ft. Cost=$2570.

Fig. 50.

the thickness of the culvert top. Culverts similar to this have been used by the Illinois Central Railway Company.

Figure 51 is a concrete box culvert with rod reinforcement similar to Figure 58, excepting that in it offset footings and cobble stone pavement are used instead of a reinforced concrete pavement slab.



2 Portals: —
Concrete 88 yds. @ $8.00 $704
Barrel per lin. ft.
Concrete 6.2 yds. @ $8.00 $49.0
Steel 150 lbs. @ .04 6 6
$55 6

If foundation is depressed as shown dotted, then area = 172 square feet.

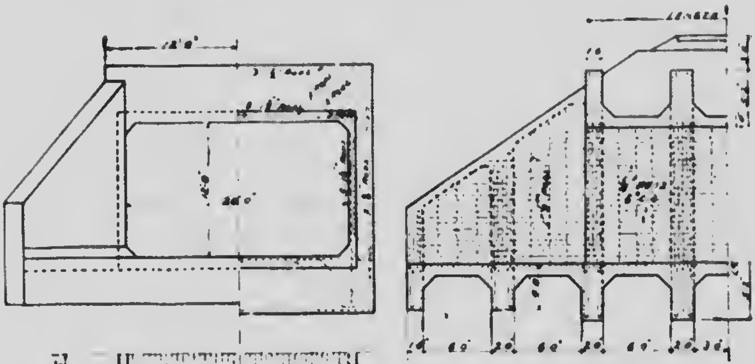
Length for 20 ft. bank = 32 ft. Cost $2480.

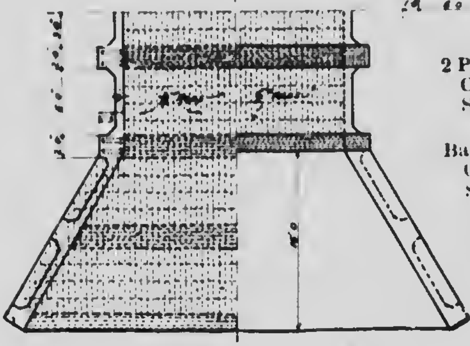
Fig. 51.

Figure 52 is a reinforced concrete box culvert of beam and slab construction, 12 feet high and 20 feet wide. For so large an area, a double box of the same type will be more economical.

Figure 53 is a culvert of the same dimensions as Figure 52, with solid concrete side walls, bottom cobblestone pavement, and roof reinforced with double lines of 60-pound track rails, united with ¾-inch.

Figure 54 is a reinforced concrete arch culvert with buttressed side walls and slab pavement. Structures similar to this are used by the Northern Pacific Railroad.





QUANTITIES

2 Portals:—

Concrete	113 yds. @	$8.00	$904
Steel	14500 lbs. @	.04	580
			$1484

Barrel per lin. ft.

Concrete	4.6 yds. @	$8.00	$36.8
Steel	700 lbs. @	.04 =	28.0
			$64.8

Length for 20 ft. bank = 25 ft.
Area 240 sq. ft. Cost $3100.

Fig. 52.

Figure 55 is a beam top culvert 12 feet high and 20 feet wide, similar to Figure 50. It will be seen that neither of these types are economical.

Figure 56 is a parabolic arch culvert.

Figure 57 is a reinforced concrete arch culvert possessing greater merit than any other form of arch culvert now in use. It contains the least amount of material, the saving being chiefly in the sides. Masonry arch culverts of the old type, whether built of stone or concrete, have the greater part of their material in the side walls or abutments. Figure 57 is designed similar to a tunnel center, or a sewer arch, and its form and light construction are possible only because of the presence of reinforcing metal in the arch ring. Culverts of this general form are being used by several of the railroad companies and are economical. They have a disadvantage, however, in requiring the use of curved forms, but this is overcome to some extent by using collapsible centers.

A modification of this form of culvert using a semicircular top, is also shown in Figure 57.

Mr. Luten's rules for proportioning such archcs under railroad banks, in spans of 50 feet or less, and with a depth of earth filling above of not less than 10 feet, are as follows:—

$$\text{Crown Thickness } D = \frac{\text{span}}{30} + \tfrac{1}{3}.$$

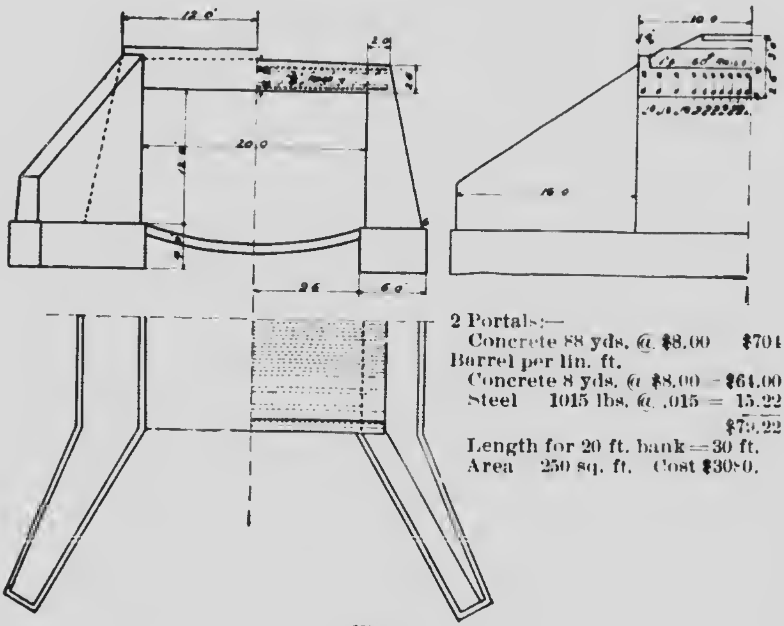


Fig. 53.

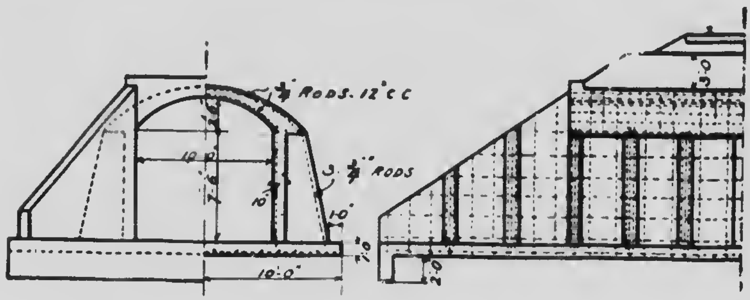
$$E = \frac{\text{span}}{10}.$$

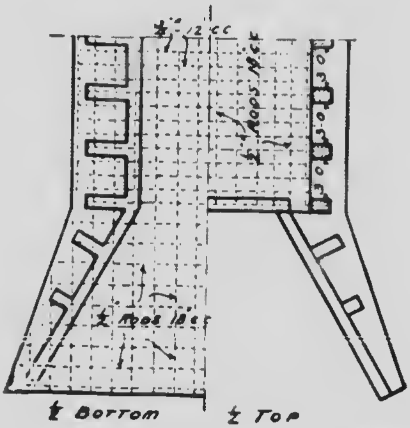
Back of abutments batter one in four.

The number of square inches of steel for one edge per lineal foot of arch is

$$\frac{R\ L}{400{,}000\ D}.$$

L is the live load in pounds that can be concentrated on the half arch for one track.



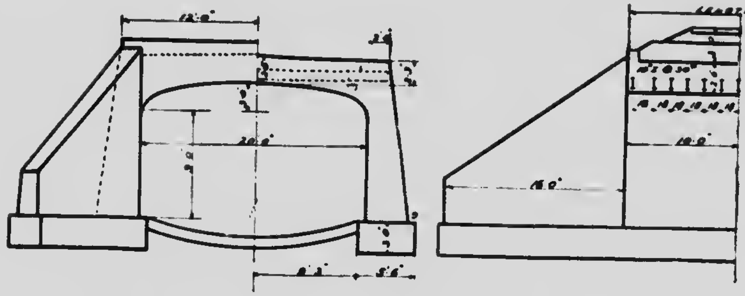


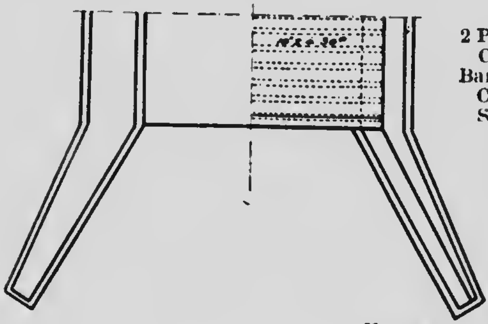
2 Portals:—
Concrete 47 yds. @ \$8.00 = \$376
Steel 1600 lbs. @ .04 = 64
\$440

Barrel per ft.
Concrete 3.1 yds. @ \$8.00 = \$24.8
Steel 170 lbs. @ .01 = 6.8
\$31.6

Area = 92 sq. ft. Length for 20 ft. bank = 42 ft. Cost for 20 ft. bank, \$1780.

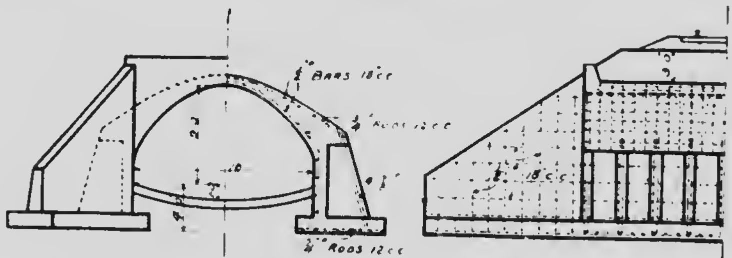
Fig. 54.

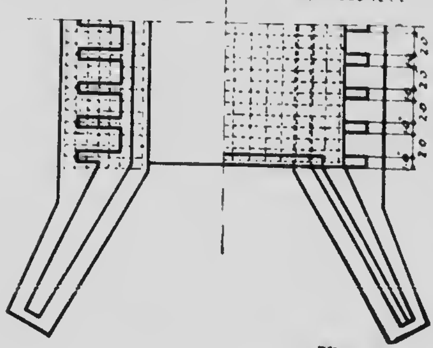




2 Portals:—
Concrete 88 yds. @ $8.00 = $704
Barrel per ft.
Concrete 7.25 yds. @ $8.00 = $58.0
Steel 460 lbs. @ .04 = 18.4
$76 4
Length for 20 ft. bank = 32 ft.
Area 215 sq. ft. Cost $3150.

Fig. 55.





2 Portals:—
Concrete 43.4 yds. @ $8.0) $347.2
Steel 2670 lbs. @ .04 106.8
$454.0
Barrel per ft.
Concrete 3.3 yds. @ $8.00 $26.4
Steel 230 lbs. @ .04 - 9.2
$35.6
Area 128 sq. ft. Length for 20 ft. bank 39 ft. Cost for 20 ft. bank, $1844.

Fig. 56.

R is the height in feet, and D the crown thickness in inches.

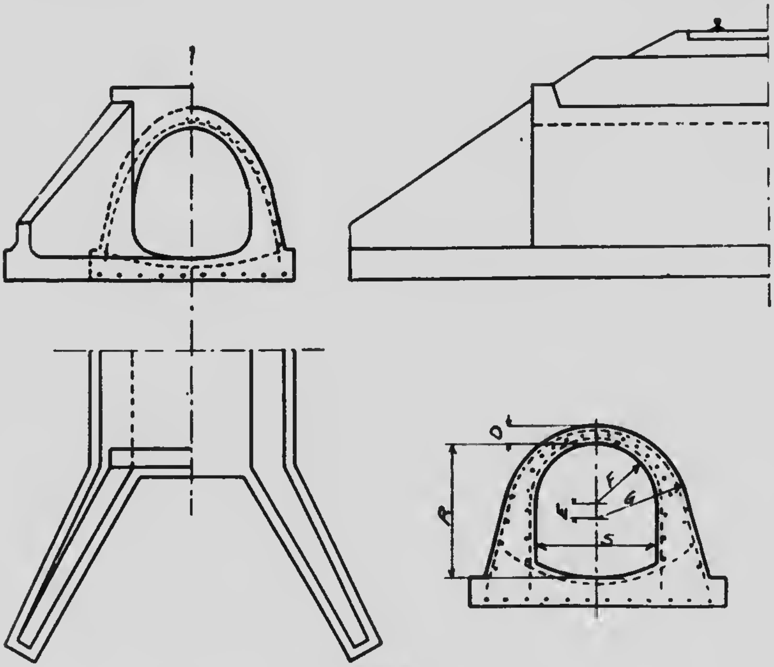


Fig. 57.

TABLE XI
CULVERT DATA, FIGURES 48 TO 56

				Barrel, per ft.			2 Portals.			20 ft. Bank		
Figure.	Width.	Height.	Area, sq. ft.	Concrete, Cubic Yards.	Steel, Lbs.	Cost, $	Concrete, Cubic Yards.	Steel, Lbs.	Cost, $	Length, Feet.	Cost, $	Cost per Square Foot, Cents.
48	15	12	180	4.7	450	55.4	72	3450	714	32	2486	30.8
49	20	11	230	5.8	515	67.0	74	4720	780	32	2924	29.1
50	15	11	161	6.1	240	58.4	88	.....	704	32	2572	36.4
51	15	11	172	6.2	150	55.6	88	.....	704	32	2483	32.4
52	20	12	210	4.6	700	64.8	113	14500	1484	25	3104	27.0
53	20	12	250	8.0	1015	79.1	88	.....	704	30	3077	31.8
54	10	10	92	3.1	170	31.6	47	1600	440	42	1784	31.4
55	20	11	215	7.2	460	76.4	88	......	701	32	3148	35.5
56	16	10	128	3.3	230	35.6	43	2670	454	39	1844	27.8

CONCRETE RAILROAD TRESTLES.

Figures 58, 59 and 61 to 65 inclusive show five different types of reinforced concrete railroad trestles. In connection with these and for the purpose of comparison, a diagram and table of dimensions is given in Figure 60, for double track steel beam bridges, a type generally in use by the railroad companies for short spans. The drawings for these different types of concrete trestles show double-track structures, 28 feet wide with 15 inches of filling, sufficient only for the usual depth of ballast. When headroom or other conditions will permit, additional space for earth filling beneath the ballast should be provided, making a minimum depth from base of rail to concrete of not less than 3 feet. In many bridges this depth has been exceeded. The arch viaduct over the Santa Ana River at Riverside, California, has a depth of 5 feet from the base of rail to the extrados at the crown.

These trestle designs marked A to H inclusive are of the following types:

Double Track Structures.

A. Railtops. Loads carried entirely by rails in bending.

B. Beamtops. Loads carried entirely by beams in bending.

C, Standard steel beam bridges. Open decks

D. Beamtops. Beams for reinforcing only.

E. Reinforced concrete. Slab type. Rod reinforcement.

F. Reinforced concrete. Beam and slab type. Rod reinforcement.

SINGLE TRACK STRUCTURES.

G. Reinforced concrete. Slab type. Rod reinforcement.

H. Reinforced concrete. Beam and slab type. Rod reinforcement.

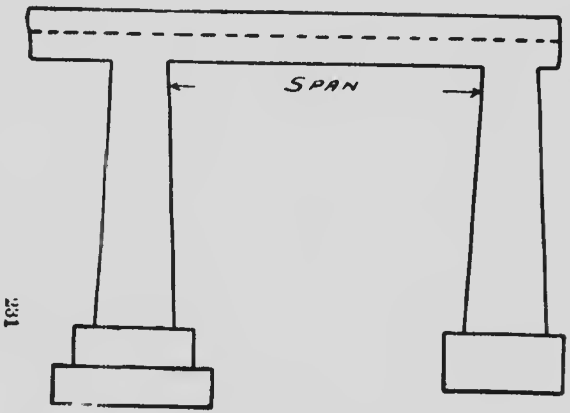
These standard trestles were designed by the author, without special reference to the standard culverts, and also under a somewhat different specification. Instead of making an impact allowance amounting to 50% of the live load and using a 700-pound concrete working unit, as in designing the concrete culverts, the standard trestles are designed with no impact addition and with a working unit of 500 pounds per square inch for concrete in compression. The assumed engine load is Cooper's E 50, which is equivalent when distributed by the ties, rails and ballast to a uniform live load of 1,100 pounds per square foot. To this is added the weight of track, filling and concrete, making the total loads from 1,500 to 1,700 pounds per square foot, as particularly noted on the various figures. The foundations are of sufficient width so the bearing pressure on the soil will not exceed three tons per square foot. For the purpose, however, of making the estimates liberal, the pier quantities in all cases include piles. It will be seen that on each plate is a table giving the length of span, thickness of concrete, size of metal, and the quantities of concrete, steel and ballast, together with the estimated costs for the various spans. In connection with designs B. D. E and G. there are also tables giving the sizes. quantities and costs for piers of various heights. The piers vary from 2 to 3 feet in thickness at the top, depending on their height, and they have side batters of 1 in 24. When piers have a less height than 15 feet, there is only a single footing course at the base, but for heights greater than 15 feet there are 2 footing courses. This is necessary to prevent the load on the soil exceeding 3 tons per square foot.

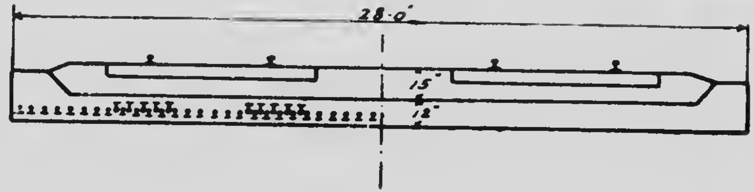
Economic Span Lengths. The designs are made for spans up to 24 feet in length and piers up to 30 feet in height, and are suitable for structures within these limits. The economic span length to use for any given height of trestle, is that one where the cost of the span is approximately equal to the cost of pier. The cost of pier for the given trestle height may be taken directly from the pier tables, and from the corresponding table giving the cost of span, a length may be selected, the cost of which is approximately equal to the cost of the pier. Having thus determined the economic span length, the various sizes may be taken directly from the tables.

Description of Various Trestle Designs.

The following are brief descriptions of the various trestle designs referred to above:—

Design A. Figure 58. This is a type that has been extensively used for small spans up to 12 feet





Span	Size of Rails	Weight of Rails	Concr. Cu. Yd.	Ballast Cu. Yd.	Cost
5	58@50 lbs.	6860 lbs.	7.8	8.4	$199
6	58 - 60	9280	8.7	9.6	256
7	84 - 50	12600	9.7	10.8	332
8	84 - 55	15400	10.4	12.0	394
9	78 - 60	17160	11.7	13.2	439
10	78 - 70	21840	12.6	14.4	540
11	78 - 80	27040	13.6	15.6	653
12	90 - 80	33600	14.1	16.8	788

Unit Prices:—

Old Rails in place 2c per lb.
Concrete " " $8.00 per yd.
Ballast " " .50 " "

Quantities and costs allow for a bearing of 1 ft. at each end.

F. S. 10000.

Total assumed load 1500 lbs. per sq. ft. Rails carry entire load in bending.

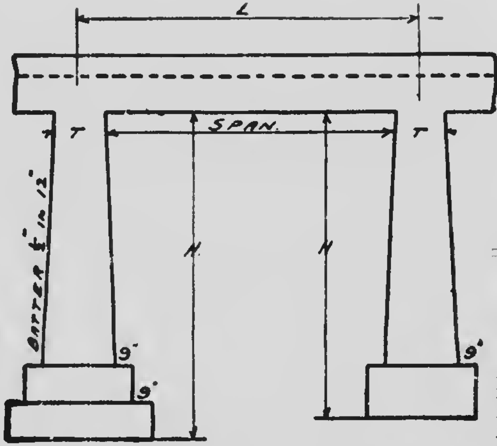
Fig. 58. Design A.

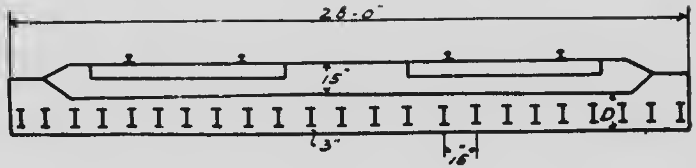
DOUBLE TRACK CONCRETE TRESTLES. RAIL TOPS.

in length, though usually restricted to a length of 8 feet. The loads are carried entirely by the bending resistance of the rails. Railroad companies usually have a large stock of old track rails on hand, which they are willing to sell to their construction department at a price of from $20 to $30 per ton. They are estimated in the table accompanying Figure 58, to cost $40 per ton, or 2 cents per pound, placed in position. Only a sufficient thickness of concrete is used, to completely embed the rails and hold them securely in position. The strength of the concrete is considered only by allowing a flange stress of 10,000 pounds per square inch on the metal, which is 20 per cent. greater than would be permitted, if the concrete filling were absent. This type of bridge is going out of favor, not only because it is not economical, but also because there is no provision for resisting shearing stresses. Bridge decks so constructed have excessive deflection, and the concrete frequently cracks and falls away from the rails, leaving the steel exposed.

If loads were carried by the bending resistance of the concrete and rails used only for the purpose of reinforcement, these rails would then be spaced from 2 to 3 feet apart. The best modern practice in the use of railtop trestles and culverts is to adopt a mean between these two extremes, and use slabs of concrete 18 inches in thickness, reinforced with old 60-pound rails spaced as follows:

For 6-foot span, place rails 18 inches apart on centers.





For heights less than 15′, piers have only one offset at bottom.

Following are assumed unit prices:—
Steel in place 4c per lb.
Concrete " $8.00 per yd.
Ballast " .50 " "
Piles " 5.00 each.

Quantities and cost are for length L.
F. S=12000 lbs. Load on soil=3 tons sq. ft.

Total assumed load, 1500 lbs. per sq. ft.

Beams carry entire load in bending.

Span

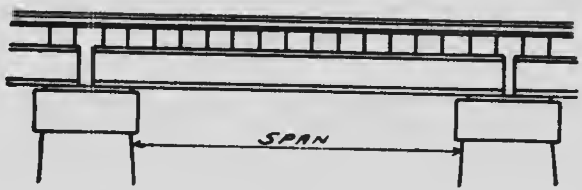
Span	Beams	D	Steel Lbs.	Conc. Yds.	Ballast Yds.	Cost
8′	9 @ 25 lbs.	15	5750	14.4	12.0	$ 358
9	10 - 25	16	6325	16.9	13.2	401
10	12 - 31.5	18	8875	21.1	14.7	537
11	12 - 31.5	18	9600	22.8	15.9	580
12	12 - 35	18	11672	24.9	17.4	680
13.5	15 - 42	21	15456	32.0	19.2	889
15	15 - 50	21	20412	35.3	21.4	1118
16.5	18 - 55	24	24351	43.8	23.1	1344
18	18 - 60	24	28980	47.6	25.1	1560
19.5	20 - 65	26	33637	55.2	27.1	1808
21	20 - 70	26	38640	58.6	28.8	2036
22.5	20 - 80	26	46920	61.8	30.7	2393
24	24 - 80	30	49680	76.3	32.5	2620

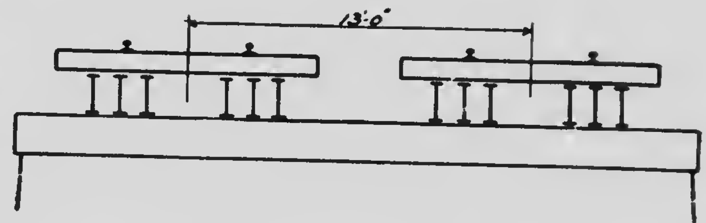
Pier

Ht. H	T	Conc. Yds.	Piles	Cost
6′	2-0	20	20	$ 260
8	2-1	24	20	297
10	2-2	31	20	348
12	2-3	41	20	428
14	2-4	49	20	492
16	2-5	60	30	630
18	2-6	81	30	798
20	2-7	91	30	878
22	2-8	104	30	982
24	2-9	117	40	1136
26	2-10	130	40	1240
28	2-11	145	40	1360
30	3-0	161	40	1488

Fig. 59. Design B.

DOUBLE TRACK CONCRETE TRESTLES. EMBEDDED STEEL BEAMS.





For extreme length add 3 ft. to span.
Cost of steel in place—4½c per lb.
Total assumed load—12000 lbs. per ft. track.
F. S.=8000.

Span	Size of Beams	Weight of Beams	Weight of Bracing	Total Wt. Steel	Cost
10'	8-I-15 @ 60 lbs.	6240	1440	7680 lbs.	$ 344
12	8 - 20 - 65	7800	3000	10800	486
14	8 - 20 - 80	10880	3640	14520	650
16	12 - 20 - 65	14640	4200	18840	850
18	12 - 20 - 80	20160	4830	24950	1100
20	12 - 24 - 80	22080	5520	27600	1240
22	16 - 24 - 80	32000	6240	38250	1720
24	16 - 24 - 90	38880	7020	45900	2060

Fig. 60. Design C.
DOUBLE TRACK BEAM BRIDGES.

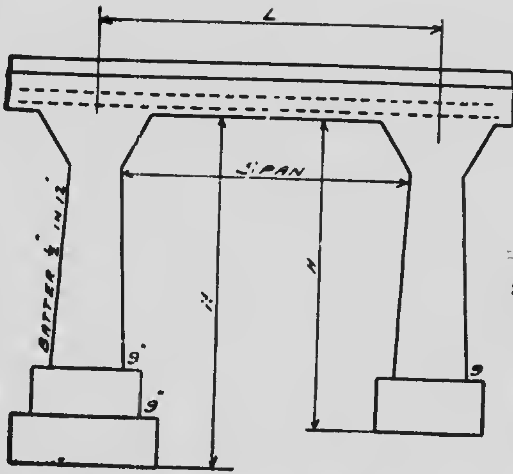
For 8-foot span place rails 10 inches apart on centers.

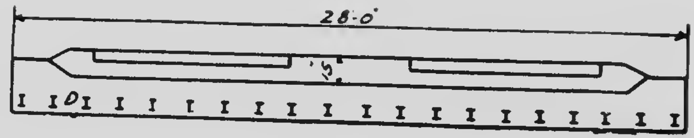
For 10-foot span place rails 6 inches apart on centers.

Design B. Figure 59. In this design beams are placed 15 inches apart on centers, and are sufficiently heavy to carry the entire load by the bending resistance of the beams. No reliance is placed upon the concrete excepting that a working fibre stress on the metal of 12,000 pounds per square inch is assumed, which is greater than would be used, if the concrete were absent. The beams are firmly embedded in concrete with a minimum thickness of 3 inches beneath the beams, and a similar depth of concrete above the beams at the gutter. The upper surface of the concrete slab is sloped from the gutter up to the center sufficiently to drain the water to the gutter and prevent it from soaking into and disintegrating the concrete.

Design C. Figure 60. There is no concrete whatever in connection with this design. It is one of the common forms of short-span railroad bridges, and the table of sizes, weights and estimated costs is given for comparison with the cost of reinforced concrete designs. The type of bridge is inferior to the concrete designs because of their open decks. An open-deck bridge is a weak place on a permanent roadway. If a train is derailed on a solid deck bridge, the chance of injury either to the train or structure is less than when derailment occurs on an open deck bridge.

Fig. 60. Design C.
DOUBLE TRACK BEAM BRIDGES.





For Heights less than 15 ft. piers have only one offset at bottom.

Quantities and costs are for length L.

Unit prices are as follows:—
Steel in place 4c per lb.
Concrete " $8.00 per yd.
Ballast " .50 " "

Total assumed load, 1500 to 1600 lbs. per sq. ft.

SPANS

Span	D	Size of Beams	Weight Steel	Conc. Yds.	Ballast Yds.	Cost
6	10	20 I 4 - 9¼ lb	1520	7.8	9.5	$ 124
8	12	20 - 5 - 12¼	2470	12.6	12.0	203
10	15	20 - 5 - 12¼	2981	17.8	14.7	265
12	18	20 - 6 - 14¾	4204	25.0	17.4	373
14	21	20 - 7 - 17½	5717	33.7	20.0	503
16	24	20 - 8 - 20	7365	43.2	22.5	646
18	27	20 - 9 - 25	10250	53.2	25.1	842
20	30	20 - 10 - 30	13550	65.5	27.5	1073
22	33	20 - 12 - 31½	15540	79.0	30.0	1260
24	36	20 - 12 - 35	18725	94.0	32.5	1510

PIERS

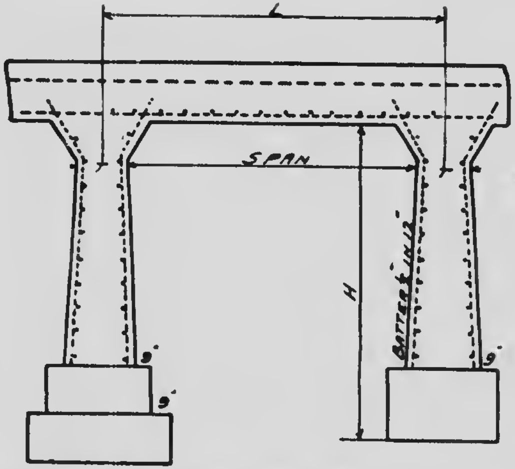
Ht. H	T	Conc. Yds.	Piles	Cost
6	2 - 0	20	20	$ 260
8	2 - 1	24	20	292
10	2 - 2	31	20	348
12	2 - 3	41	20	428
14	2 - 4	49	20	492
16	2 - 5	60	20	630
18	2 - 6	81	30	798
20	2 - 7	91	30	878
22	2 - 8	104	30	982
24	2 - 9	117	40	1136
26	2 - 10	130	40	1240
28	2 - 11	145	40	1360
30	3 - 0	161	40	1488

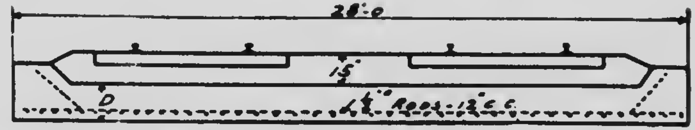
Fig. 61. Design D.

DOUBLE TRACK CONCRETE TRESTLES. BEAM REINFORCEMENT.

Fig. 61. Design D.

DOUBLE TRACK CONCRETE TRESTLES. BEAM REINFORCEMENT.





For a series of spans make every fourth or fifth pier double thickness.

Costs are based on following units:
Steel in place 4c per lb.
Concrete " $8.00 per yd.
Ballast " .50 " "

Quantities and costs are for length L.

Total assumed loads 1500 to 1600 lbs. per square foot.

Span								Pier					
Span	D	Rods		Steel	Conc.	Ballast	Cost	Ht. H	T	Steel	Conc	Piles	Cost
Ft.	In.	Sq. In.	In. cc	Wt.	Yd.	Yd.		Ft.	Ft. In.	Lbs	Yd.		
6	10	1 -	13	932	7.8	9.5	$ 102	6	2 - 0	400	20	20	$276
8	12	1 -	12	1238	12.6	12.0	153	8	2 - 1	500	24	20	312
10	15	1 -	11	1613	17.8	14.7	210	10	2 - 2	700	31	20	376
12	18	1 -	9	2197	25.0	17.4	292	12	2 - 3	900	41	20	464
14	21	1 -	8	2820	33.7	20.0	387	14	2 - 4	1100	49	20	536
16	24	1 -	7	3560	43.2	22.5	493	16	2 - 5	1200	60	30	678
18	27	1 -	6	4508	53.2	25.1	612	18	2 - 6	1400	81	30	854
20	30	1 -	5	5890	65.5	27.5	766	20	2 - 7	1600	91	30	942
22	33	1 -	4½	7034	79.0	30.0	920	22	2 - 8	1800	104	30	1054
24	36	1 -	4	8535	94.0	32.5	1102	24	2 - 9	2000	117	40	1216

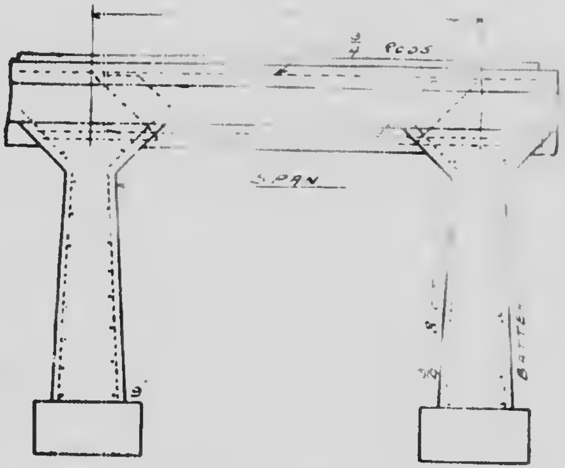
Fig. 62. Design E.

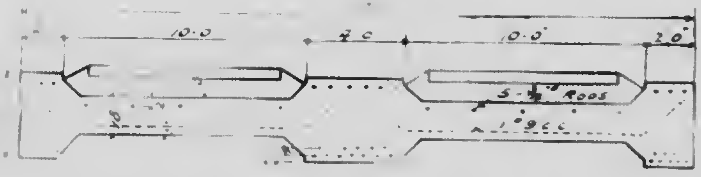
DOUBLE TRACK CONCRETE TRESTLES. SLAB CONSTRUCTION. ROD REINFORCEMENT.

Design D. Figure 61. This type is similar to Design B, but differs from it in having a sufficient thickness of concrete, reinforced with steel beams, to carry the earth loads by the bending resistance of the concrete slab. The steel beams are covered on the lower side with a 2-inch layer of concrete. The lower two inches only of the steel beams are considered effective as tension metal, for concrete reinforcement. Beams are spaced about 18 inches apart on centers. Piers have corbels and in proportioning the thickness of the slabs the effective span length is assumed one foot shorter than the actual, because of the presence of these corbels.

Design E. Figure 62. This is a reinforced concrete trestle design, both span and piers having rod reinforcement. In the two previous pier designs, reinforcing steel is omitted, but for Design E one-half inch square rods are placed 18 inches apart both horizontally and vertically. These rods serve not only to prevent cracks from change of temperature, but also resist any tensile stresses which might occur in thin piers, due to the sudden stopping of heavy trains on the bridge. The spans are slab construction, with a 10-inch slab for 6-foot span, increasing to 36 inches for a 24-foot span.

Design F. Figure 63. Like the previous one this design is reinforced entirely with rods, but is a combination of beam and slab construction. Longitudinal concrete beams are placed 10 feet apart in the clear, and to these loads are transmitted by means of 18-inch transverse slabs carrying the



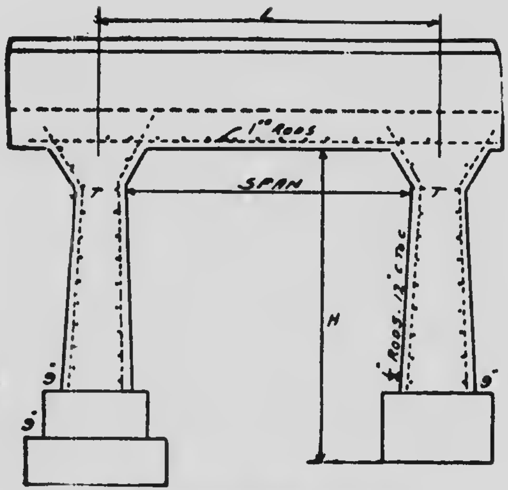


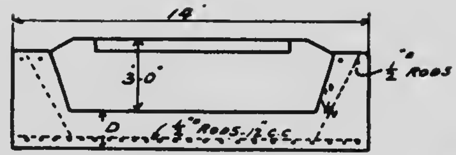
Span	Depth D	Rods in Beam Ft. Wide		Steel	Conc	Ballast	Cost
		D		Lbs.	Cu.Yd	Cu. Yd	
10			1	2776	[illegible]	14	[illegible]
12	[illegible]			[illegible]	32.5	[illegible]	490
14				[illegible]	37.5	15.0	470
16	49		[illegible]	4851	45.2	17.2	560
18	55			[illegible]	53.0	19.[illegible]	658
20	60		1¼	[illegible]	61.2	20.8	754
22	66	7	1¼	7587	[illegible]	23.0	873
24	[illegible]		1¼	8807	80.5	24.8	1002

Costs based on following units:

Steel in place, 4c per lb.
Concrete in place, $[illegible] per yd.
Ballast in place, 50c per yd.
Quantities and costs are for length L.
Total assumed load on side beams, 8500 to 9300 lbs. per ft.

Fig. 63. Design F.
DOUBLE TRACK CONCRETE TRESTLES, [illegible] AND SLAB CONSTRUCTION. ROD [illegible]INFORCEMENT





For a series of spans, make every fourth or fifth pier double thickness.

Costs are based on following units:

Steel in place, 4c per lb.
Concrete in place, $8.00 per yd.
Ballast in place, 50c per yd.

Quantities and costs are for length L.

Total assumed loads, 1670 to 1800 lbs. per square foot.

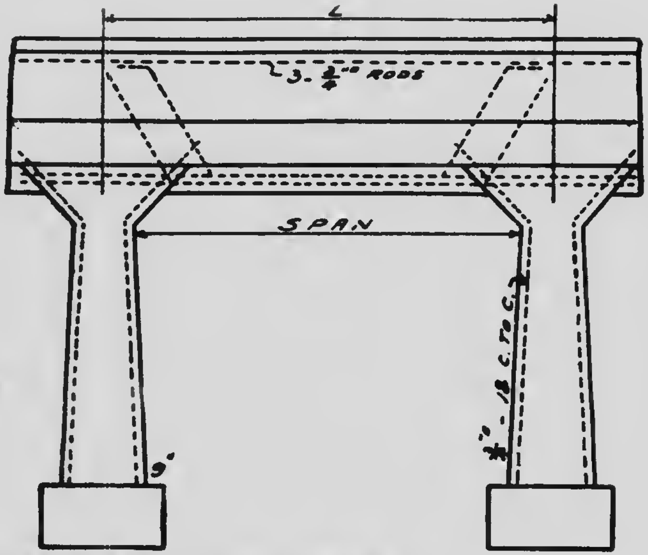
Span								Pier						
Span	D	Rods		Steel	Conc.	Bal.	Cost	Ht. H	T		Steel	Conc.	Piles	Cost
Ft.	In.	In.Sq.	In.cc	Lbs	Yd.	Yd.		Ft.	Ft.	In.	Lbs	Yd.		
6	12	1 -	13	472	5.0	11.5	$ 62	6	2 -	0	205	9.4	10	$133
8	15	1 -	12	674	7.9	14.5	94	8	2 -	1	295	11.8	10	156
10	18	1 -	10	892	11.5	17.6	132	10	2 -	2	385	15.6	10	190
12	21	1 -	8	1286	15.7	20.5	182	12	2 -	3	480	19.0	10	221
14	24	1 -	7	1643	20.5	23.5	236	14	2 -	4	570	23.5	10	260
16	28	1 -	6	2062	27.0	26.6	305	16	2 -	5	570	34.0	15	369
18	32	1 -	5	2749	34.4	29.5	396	18	2 -	6	665	38.2	15	407
20	35	1 -	4½	3335	41.5	32.6	473	20	2 -	7	750	45.0	15	465
22	38	1 -	4	4061	49.0	35.5	562	22	2 -	8	840	51.5	15	520
24	41	1 -	3½	4963	57.5	38.6	667	24	2 -	9	940	56.2	20	587

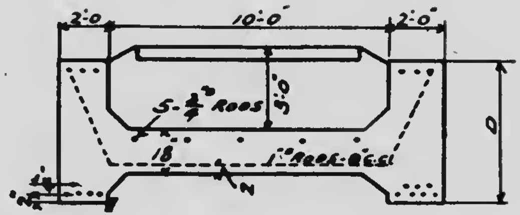
Fig. 64. Design G.

SINGLE TRACK CONCRETE TRESTLES. SLAB CONSTRUCTION. ROD REINFORCEMENT.

square foot.

Fig. 64. Design G.
SINGLE TRACK CONCRETE TRESTLES. SLAB CONSTRUCTION. ROD REINFORCEMENT.





Span	D	Beam Rods	Wt. of Steel	Conc.	Ballast	Cost
Ft.	In.	In. Sq.	Lbs.	Yds.	Yds.	
14	44	7 - 1	2076	19.0	18.0	$239
16	50	6 - $1\frac{1}{8}$	2425	23.0	20.6	286
18	56	7 - $1\frac{1}{8}$	2864	27.0	22.8	336
20	62	6 - $1\frac{1}{4}$	3240	31.0	26.0	384
22	68	7 - $1\frac{1}{4}$	3793	36.0	27.6	446
24	74	8 - $1\frac{1}{4}$	4404	41.0	30.0	511

Costs are based on the following units:—
Steel in place 4c per lb.
Concrete " $8.00 " yd.
Ballast " .50 " "

Quantities and costs are for length L.

Total load per ft. on side girder, 9200 to 10000 lbs.

Fig. 65. Design H.
SINGLE TRACK CONCRETE TRESTLES. BEAM AND SLAB CONSTRUCTION. ROD REINFORCEMENT.

track and ballast. The side beams are each 2 feet in width, while the center beam is 4 feet. The load per lineal foot on the side beams varies from 8,509 pounds for a 10-foot span to 9,300 pounds for a 24-foot span.

Designs G and H. Figures 64, 65. These are designs for single track trestles, similar to E and F already described. They differ, however, in that G and H have a 3-foot depth of earth and ballast filling.

Comparative Trestle Costs.

The comparative costs for the foregoing trestle spans for both single and double track structures is given on the chart, Figure 66. The horizontal ordinates represent clear spans in feet, while the vertical ordinates give the costs in dollars for a complete span, not including piers. This chart clearly shows that reinforced concrete trestles of the types marked E and F with rod reinforcement are more economical than any other form of permanent trestle, with solid roadway. The chart shows further that reinforced concrete railroad trestle spans of slab constructions are economical for single track in spans up to 14 feet, and for double track in spans up to 20 feet. Above these lengths the economic form of span is a combination of beam and slab.

Comparative Cost of Short Span Bridges.

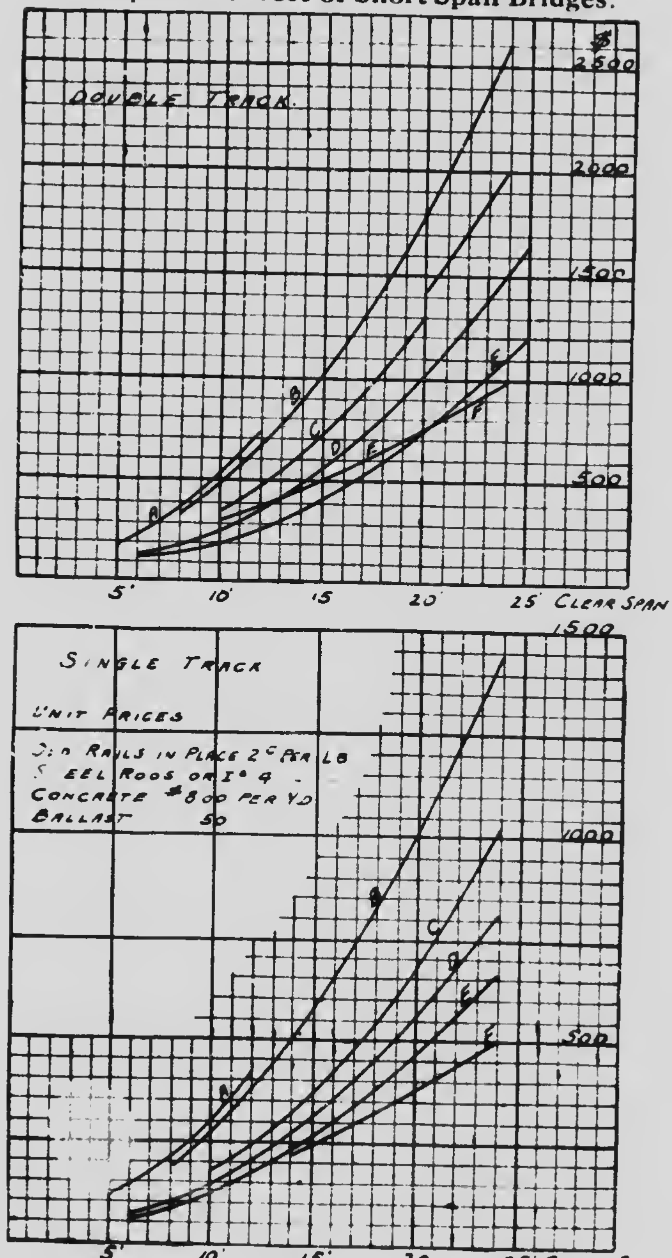


Fig. 66.

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