A TEXT-BOOK

ON

ROOFS AND BRIDGES.

PART III.

BRIDGE DESIGN.

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PREFACE.

THE present edition has been entirely rewritten in order to bring the subject fully up to date, for the changes in bridge design during the past eight years have been remarkable. The rapid increase in live loads on the principal railroads in this country has necessitated an unusually large amount of new construction to replace the old bridges, which were designed for much lighter traffic. The extensive scale on which this work had to be done led to a general revision of specifications and to careful attention to the design of details so as to secure greater stiffness as well as strength in the new structures. These changes include the introduction of some new forms of details; the elimination, as far as practicable, of adjustable members; the entire superseding of wrought iron by steel; the substitution of riveted trusses for pin-connected trusses in the shorter spans; and increased care in designing the joints so as to reduce the secondary stresses to a minimum.

In the descriptions of the details of plate-girder and truss bridges, introduced in Chapters VI, VIII, and XI, only those are given which may be properly claimed as standard in the best recent practice. Special attention is called to the new feature of carefully selected references to engineering periodicals where more extended descriptions and applications of various details may be found.

The designs in Chapters VII, IX, and X are new, being made in accordance with the latest specifications and most approved practice. As stated in the preface to the first edition, the subject is presented "both rationally, as an application of the principles of mechanics, and practically, as an illustration of modern economic construction. Since probably more than ninety percent of all bridges are those resting on two supports, this volume is confined to that class. For a beginner the study of bridge design should be largely that of proportioning details according to given specifications, and simple bridges furnish these in endless variety."

Grateful acknowledgments are due to many railroad and bridge engineers for kind assistance: to RALPH MODJESKI and E. H. MCHENRY for permission to reproduce three sheets of standard plans; to J. A. L. WADDELL for permission to reprint the larger portion of his specifications for steel railroad bridges for simple spans; to C. C. SCHNEIDER, J. E. GREINER, W. J. WILGUS, W. A. PRATT, F. W. SKINNER, and A. F. ROBINSON for photographs and drawings; to ENGINEERING NEWS and ENGINEERING RECORD for permission to reprint those illustrations which are marked with their respective names; to THADDEUS MERRIMAN for the chapter on Bridge Shops and Shop Practice; and to F. O. DUFOUR for the chapter on the Design and Detailing of a Highway Bridge.

A comparison with the third edition shows that the number of pages has been increased from 316 to 374, and the number of cuts from 57 to 149, of which 20 are full-page illustrations; the number of folding plates is the same, but all of these are new. In rewriting the volume, it has been the constant aim of the authors not only to give the latest details of modern bridge practice, but also to set forth the reasons for such practice in a manner especially adapted to the needs of students and young engineers.

MARCH, 1902.

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be made entire in the shop and swung into place by a derrick, the only field-riveting required being that necessary to connect the girders by lateral bracing.

A tubular bridge is a girder structure with its sides formed of plates and stiffeners, and its top of channels, angles, and plates, all being riveted together so as to form a closed tube. This type originated in England about 1840, and in 1850 STEPHENSON built the great Britannia bridge in Wales on this plan, the tube being 25$\frac{1}{2}$ feet high and 13$\frac{3}{4}$ feet wide inside, and there being four spans, two of 230 feet and two of 460 feet. The Victoria bridge over the St. Lawrence River at Montreal, completed in 1859, was of this type, but it was replaced in 1898 by a truss structure. These tubular bridges, though stiff, were unnecessarily heavy, and accordingly very expensive, and the passage through them was like that through a tunnel. All experience indicates that the girder system of construction cannot be economically applied to bridges of long span.

A lattice truss, or lattice girder, as it is sometimes called, consists of flanges formed like that of the plate girder, but with the solid web replaced by flat, diagonal bars. The Warren truss, with a double system of web bracing (Part I, Art. 64), originated in England about 1840, and it may be regarded as being an attempt to economize material by removing unnecessary parts of the web. This was a step in the right direction, as the web stresses were thereby more closely determinate than before. But, as will be seen in the following articles, greater precision regarding stresses and greater economy in material have been attained by discarding the double set of diagonals, and using only a single system of bracing to connect the chords.

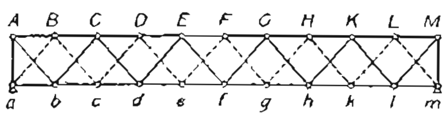


Fig. 2.

ART. 2. TRUSS DESIGN PRIOR TO 1800.

Bridge design prior to the year 1800, and indeed for many years after, was almost wholly empirical. Arch bridges of stone had been successfully built since the time of the Romans, and structures of timber were used for roofs and often for bridges, but the true idea of a bridge truss and of the functions of its members was not fully understood until near the middle of the nineteenth century. About 1830, owing to the introduction and development of railroads in both Europe and America, bridge construction assumed an importance never before known. In Europe the main line of evolution was based upon the metal girder, as described in the last Article. In America, however, the evolution was along the line of the truss, starting with timber and gradually developing into structures of iron and steel. A truss is a framework whose members are so arranged that they are subject only to longitudinal stresses of tension and compression. These members should be arranged in triangular figures so that no distortion of the structure can occur without bringing the proper stresses into action, and the applied loads should preferably be concentrated at the joints (Part I, Art. 23). The simple truss, supported at its two ends, is the one whose history is now to be considered.

The king-post truss shown at *a* in Fig. 3 may be supposed to be the origin of all modern bridge trusses. Prior to 1800,

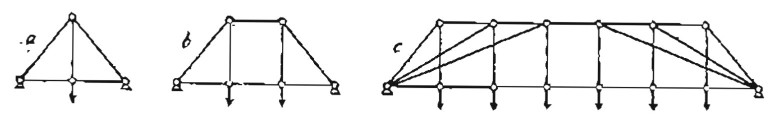


Fig. 3.

however, the principal line of development was that indicated by the diagrams *b* and *c*. On this plan many wooden bridges were erected during the seventeenth and eighteenth centuries. There were two chords, usually with a high camber, connected by vertical timbers acting as ties to support the floor which was placed along the lower chord. From the top of each vertical an inclined brace was carried to the nearest abutment and the tops of the corresponding pairs connected by a straining beam. True truss action as we now understand it scarcely existed, the main idea being to carry each load to the abutment by the shortest route. This was a simple plan, but it proved uneconomical on account of the long braces whose stresses increase both with their length and the angle of inclination to the vertical. On this plan was built, in 1760, by GRUBENMANN, a timber bridge near Baden, which had the great span of 366 feet, and which exhibited much skill in carpentry.

The secret of economical and efficient truss arrangement lies in the panel system, which may be regarded as having been

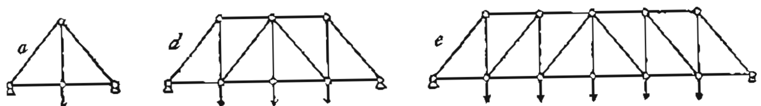


Fig. 4.

developed from the king-post truss in the manner shown in Fig. 4, where *d* is derived from *a* by the addition of a panel on each side, and *e* from *d* in like manner. This system was first used by PALLADIO, an Italian architect, about 1570, but it produced little or no influence on methods of construction, until it was rediscovered and used in the United States near the close of the eighteenth century by THEODORE BURR. The Burr truss may indeed be called the parent of nearly all the forms of bridge trusses now used in this country. Although so defective from the lack of counterbraces that it generally required the assistance of an arch to stiffen it under rolling loads, yet as it contained the sound principle of economy in a constant angle for the inclined members its panel system was transmitted to the Long truss, the Howe truss, and later to many other forms (Part I, Art. 25).

Concerning early timber bridges, as also for other valuable historical and descriptive matter, the student should consult COOPER's American Railroad Bridges, 1890, the article Bridge in the Encyclopædia Britannica, and the article Bridges in JOHNSON's Universal Cyclopædia, 1897.

ART. 3. PROGRESS FROM 1800 TO 1850.

Near the beginning of the nineteenth century many wooden bridges were erected in the eastern and middle states by THEODORE BURR and by TIMOTHY PALMER, both of whom used the panel system. PALMER's bridges generally combined the action of the truss and the arch in one structure, the lower chord being highly cambered, while BURR used the arch merely as auxiliary to the truss. The oldest truss bridge now standing in the United States is that built by BURR at Waterford, N. Y., in 1804, which is of hewn yellow pine, having four spans of 154, 161, 176, and 180 feet in the clear. Illustrations of this bridge and of one built by PALMER at Easton, Pa., in 1805 are given in COOPER's American Railroad Bridges. WERNWAG's great bridge of 340 feet span, built at Philadelphia in 1812, also deserves notice as a splendid example of early engineering work; the double diagonal bracing in its panels showing that probably its builder had considered the distorting action of rolling loads.

The lattice truss introduced by TOWN about 1820 consisted of planks pinned together, and was important only on account of ease of construction. In 1830, however, a radical advance was made in the true principles of truss arrangement through the introduction of panel counterbraces by S. H. LONG. In a pamphlet published by him in 1836 the function of counterbraces in preventing the distortion of the panels under rolling loads, and also their use in stiffening the truss when keyed up so as to be under initial stress, is clearly recognized. Wooden Long trusses were erected on the Baltimore and Ohio Railroad as well as many for highway service.

In 1840 WILLIAM HOWE patented a combination truss having wooden chords and web diagonals and wrought-iron vertical ties, which has since been extensively used. Each panel had counter as well as main struts, both butting against cast-iron angle blocks. Many important bridges were built on this plan prior to 1850, the most notable being that over the Susquehanna

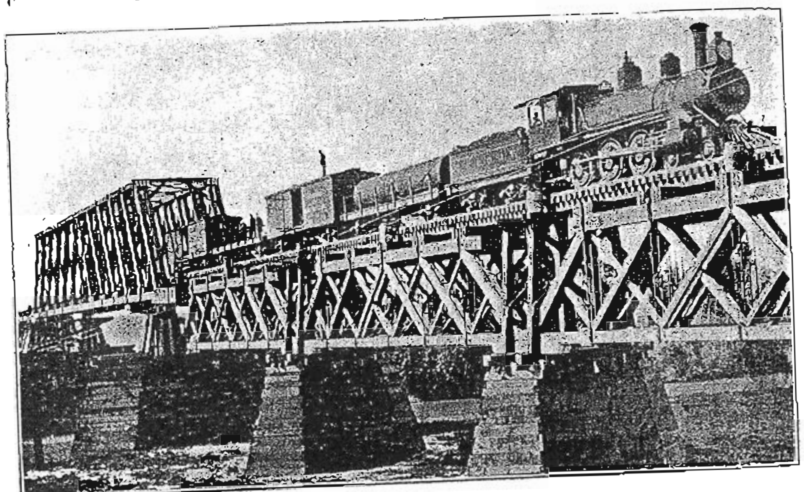


Fig. 5.

at Havre de Grace, Md., which had thirteen spans of 250 feet each and a draw span of shorter length. The Howe truss is still in common use in localities where timber is cheap, and for short spans and light traffic it often makes an efficient and economical bridge. Fig. 5 shows a Howe truss bridge of several spans over the Stanislaus River, near Riverbank, Cal., and on the Atchison, Topeka, and Santa Fé Railway.

In 1844 the Pratt truss was introduced. In this a radical departure was made in the arrangement of the web members,

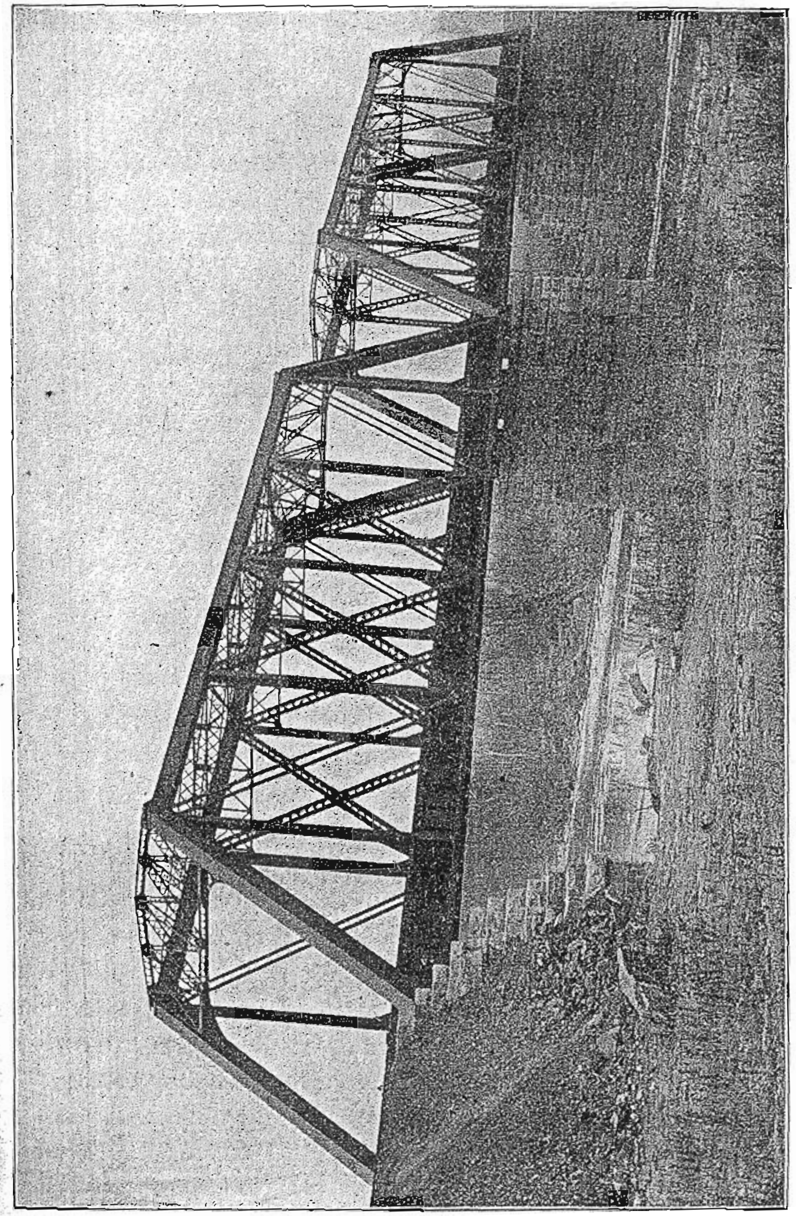


Fig 6 Baltimore and Ohio Railroad Bridge crossing South Branch of Potomac River in West Virginia

the timber verticals being made to take compression, and the iron diagonals to take tension. This was a move in the direction of economy, since the length of the struts was decreased and thus the necessary cross-section somewhat diminished. Although at first built as a combination bridge, it never attained great popularity in this form, but soon after 1850 it began to be constructed entirely in iron, and in this form it has probably been more extensively used than any other form of truss. Fig. 6 shows a Pratt truss bridge of two spans, each $172\frac{1}{2}$ feet long, erected in 1901.

Few iron structures were built in the United States prior to 1850, the first one being a span of 77 feet erected in 1840 over the Erie Canal, which was formed of cast-iron girders strengthened by wrought-iron rods. About the same time WHIPPLE built a truss with a curved upper chord of cast iron and a straight lower chord of wrought iron, forming the bowstring truss. A Howe truss in iron was introduced in 1844, and the Rider iron truss with a multiple web system was first built about 1847, but neither came into general use, and some that were built failed.

The first rational discussion of the determination of stresses and proportioning of cross-sections of truss members was published in 1847 at Utica, N. Y., by SQUIRE WHIPPLE under the title A Work on Bridge Building, in which are given methods of computing stresses due to dead and live loads, investigations as to the angle of economy for web bracing, with plans and details of the bowstring truss and of the double system Pratt truss, since known as the Whipple truss. WHIPPLE was far in advance of his time in rational views of economic truss design, but the circulation of his book was small, so that its influence was not fully exerted until several years after publication. He also built over twenty bridges on his plans which gave good service for many years. SQUIRE WHIPPLE is justly regarded

as the father of American rational bridge design. Drawings of bridges built between 1840 and 1850 may be seen in DUGGAN'S Stone, Iron, and Wood Bridges of United States Railroads, 1850; and also in HAUPT'S General Theory of Bridge Construction, 1851.

ART. 4. TRUSS EVOLUTION SINCE 1850.

The modern bridge truss is the result of an evolution or development in the sense that it exhibits those features which experience has found to be most economical and stable. Forms costly or unsafe have disappeared, while those cheap and strong have remained in use. Thus, the panel system has survived, while the method of transferring loads directly to the abutments by long braces, as seen in Fig. 3, has gone out of use. The Bollman truss, introduced soon after 1850, was an instance of the application of that erroneous principle, but it could not be built for spans greater than 160 feet, and even for shorter spans it was unable to compete in economy and stability with trusses of the panel system. The Fink truss (Part I, Art. 53) is another example of the use of that principle, and it too has disappeared.

The Whipple truss (Fig. 7) is an instructive instance of a form which was extensively used from 1850 to 1885, even for the longest spans, but which now is no longer built. This has

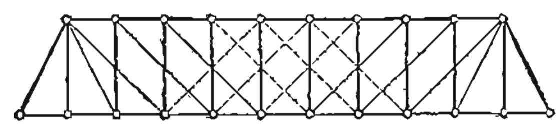


Fig. 7.

all the advantages of the Pratt type as regards the use of vertical compression members in the web, and also by the double system of webbing the panel points are brought nearer together, thus decreasing the length of the stringers, which for long spans is a matter of importance. Stringers longer than 25 feet make an expensive floor; and this limits the economic depth of the Pratt truss to about 30 feet, and the span to about 300 feet, since it is not advisable to make the depth less than one tenth of the span. With the Whipple truss, however, using the same angle for the bracing, the depth of the truss can be doubled, and the span thus be economically increased. Many long bridges have been erected on this plan, among which may be mentioned the 515-feet span of the Ohio River bridge at Cincinnati, completed in 1877, and which at that date was the longest truss span ever erected. The Whipple truss began to go out of use merely because it was found to be more economical to support the floor beams by short sub-verticals attached to a single system of bracing than by the use of a double system, and because the single system is always more reliable and determinate in respect to stresses. The Post truss (Part I, Art. 55) is another example of a form once popular and now no longer in use.

The Warren or triangular truss was built with both single and double systems of webbing, but with a single system it afforded opportunity for the support of intermediate floor beams in a panel by the use of independent vertical members. In 1869 the channel span of 390 feet over the Ohio at Louisville was built on this plan, and in 1885 the 525-feet span at Henderson. This plan has been found advantageous because simplicity of truss action is secured, the only apparent disadvantage being the use of long inclined compression members in the webbing; in accordance with the law of evolution the former of these tends to be perpetuated and the latter to disappear.

At the present time the Pratt truss is most generally used for short spans. The Baltimore truss (Fig. 8) is used for both short and long spans; it possesses all the advantages of the Pratt, and in addition that of supporting intermediate floor beams by the use

of sub-verticals. The modified bowstring truss, shown in Fig. 9, uses the same idea, and here is gained the important advantage that the stresses in the chords are rendered closely uniform, as also those in the webbing. These elements combined

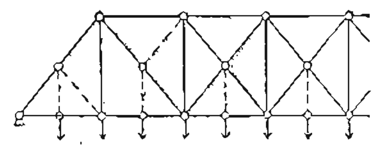


Fig. 8.

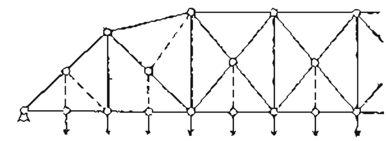


Fig. 9

have rendered this form applicable to the longest simple trusses, the longest of all being in the great spans of 542½ feet built over the Ohio at Cincinnati in 1888 and of 546½ feet at Louisville in 1893. Fig. 10 shows one span, 533 feet long, of the Delaware river bridge of the Pennsylvania Railroad, built in 1896.

To recapitulate, the principles which should control the arrangement of a simple truss are the following: first, the panel system whereby the inclined members in the webbing are kept

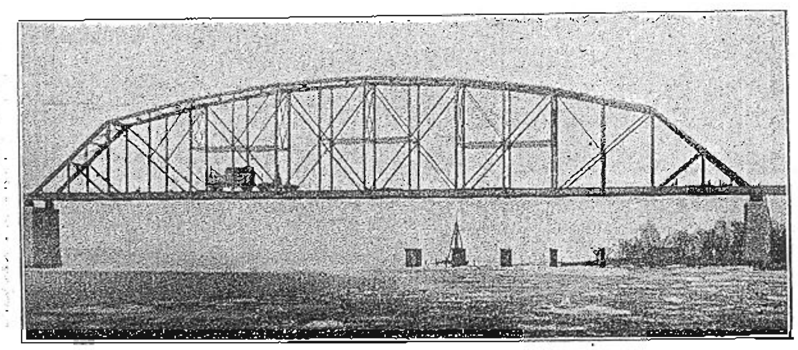


Fig. 10.

at approximately the same angle; second, the use of counterbraces to prevent distortion under a rolling load; third, that compression members should be made as short as possible; fourth, that a single system of webbing is always preferable, and that intermediate floor beams may be supported when necessary by the use of independent verticals; and fifth, that the form of the truss should be such that the stresses in members of the same kind may be approximately equal.

In addition to the references at the close of Arts. 2 and 3 the following may be noted as treating of the development of trusses: Bridge Superstructure, a committee report in Transactions of American Society of Civil Engineers, 1878, Vol. 7, pp. 339–368; an Address by JOSEPH M. WILSON in Proceedings of the Engineers' Club of Philadelphia for 1889, Vol. 7, pp. 65–104; The Evolution of the Modern Bridge by CHARLES D. JAMESON in Popular Science Monthly, Feb. 1890, pp. 461–481; and the Evolution of Bridge Trusses by MANSFIELD MERRIMAN in Railway Age for May 19, 1893, Vol. 18, pp. 381–393.

ART. 5. MATERIALS USED IN BRIDGES.

Prior to 1840 wood was the material used in this country for bridge construction. Great skill in carpentry was developed to devise the joints, mortises, keys, and other connections, although little was known regarding the strength of the timber or the rational principles of designing the proportions of the parts. The Howe truss combined the use of wood and iron in a most simple and successful manner, wrought-iron adjustable tie rods being used for the vertical members of the web, while the wooden diagonals butted against cast-iron angle blocks. In the original Pratt truss, cast-iron joint connections were also employed, through which the wrought-iron diagonal ties passed. The first bridges wholly in iron had the compression members of cast iron and the tension members of wrought iron, this being, as WHIPPLE advocated, the best theoretic combination, since cast iron is high in compressive and low in tensile strength. Wrought iron, moreover, was high in price, and could then scarcely be obtained except in the form of simple rods.

Bridges of cast and wrought iron were built extensively until about 1870, and many of short span since that year; but the numerous failures of the cast-iron parts led to the gradual substitution of wrought iron. Probably the first bridge in which both compression and tension members were made of wrought iron was that erected on the Lehigh Valley Railroad at Mauch Chunk in 1863, but in this cast-iron joint connections were used. Gradually but surely wrought iron displaced cast iron, both for truss members and for joint details, so that by 1875 cast iron was regarded as a material wholly inappropriate for use in bridge structures for railroad purposes, and the period of wrought-iron bridge development was at its height. But about this time steel began to be introduced.

The first extensive application of steel was in 1873 in the arches of the great St. Louis bridge, and later it was used in the trusses of the Brooklyn suspension bridge. In ordinary trusses it was at first employed in the form of eye-bars for tension members, and then for the webs of floor beams. But improvements in the methods of manufacture soon followed, so that by 1890 angles, beams, channels, and other shapes of medium or mild steel were easily obtainable in the market at the same price as those of wrought iron. This structural steel closely resembles wrought iron, but its strength is about ten or fifteen percent higher, and hence in 1900 it had entirely replaced wrought iron in bridge construction.

The average life of iron or steel railroad bridges is probably not far from twenty years, although under heavy traffic many are replaced after fewer years of service. They are liable to corrode from atmospheric influences and from the gases from the locomotives, while rivets and other connections are worn and loosened under the frequently repeated stresses and shocks. Bridges built twenty years ago are now generally unable to carry the heavier rolling stock with a proper margin of security. Hence a metallic structure cannot compete with stone with respect to durability, and accordingly many roads are replacing short spans by arches of stone. The cheapness of iron and steel, however, generally renders metallic structures more economical in spite of their shorter life, and of course for long spans no other materials are available.

Some interesting notes by SQUIRE WHIPPLE on early iron bridges will be found in Railroad Gazette, April 19, 1891. A historical paper on steel manufacture in America by W. F. DURFEE is given in Popular Science Monthly, Oct. 1891, pp. 729–749. See also COOPER's American Railroad Bridges, originally published in Transactions American Society of Civil Engineers for 1889, Vol. 21, pp. 1–28.

ART. 6. JOINT CONNECTIONS.

The members of the early wooden bridges, such as the Burr truss and the Long truss, were connected together by means of joints devised especially for timber structures. The fish and scarf joints employed in the chords are still used in the Howe truss and in other wooden constructions, but most of the special devices of shoulders, mortises, and keys now exist only in a few isolated examples.

The combination trusses which next followed, like the Howe and Pratt, employed the method of screw connections to join the webbing to the chords. In the Howe truss the several pieces of the chords were bolted together laterally, and connected longitudinally by fish joints so as to form one continuous member, but the web struts butted against angle blocks and were held in place by screwing up the vertical iron tie rods.

The Pratt truss in its early forms had wooden chords upon which was placed at each panel point a cast-iron joint block, and through this passed the diagonal iron ties which terminated in screws and nuts by which the whole was held in place. This method was also extensively used in the Pratt trusses built of cast and wrought iron, and many special forms of screw connections were devised and employed. In general, however, most of these screw joints have gone out of use, on account of the greater cheapness and reliability of the methods of riveted and pin connections.

The riveted system of connections is the prevailing method of construction in Europe, but in this country it is mostly limited to plate girders and to lattice trusses less than 200 feet in span. In this system the chords are formed of angles, or channels, and plates, riveted together, with splice joints so as to make them practically continuous from end to end; and the web members are connected to the chords by rivets, either directly or by means of special plates riveted to both. The first riveted bridges in this country were erected on the New York Central Railroad about 1860, and the system has proved very serviceable there and elsewhere.

The pin system of connections is the one which has been most used and which has generally been regarded with the most favor by American engineers. At each panel point a pin, or round bar, passes through holes in the chord or web members and serves to transfer the longitudinal stresses from one member to another by means of the shearing and bending stresses generated in it. Some of the early bridges built by WHIPPLE had pins which passed through looped eyes in the tension members, but the first bridge which was pin-connected throughout was erected by J. W. MURPHY in 1859 on the Lehigh Valley Railroad at Phillipsburg, N. J. Wide forged eye-bars in connection with pins were first used in 1861 by

J. H. LINVILLE on the Pennsylvania Railroad. The system then rapidly spread on account of ease of erection, and thousands of pin-connected bridges are now in service.

Much might be said in comparison of the riveted and pin systems. Advocates of the former claim that it makes a stiffer structure and one less liable to accident from the failure of a single member. Advocates of the latter say that the stresses in the pin system are more determinate and that better workmanship is secured. But under present conditions the question of economy seems the controlling factor. A long span cannot be built as cheaply by the riveted system as by the other, and a short or medium span can sometimes be built more cheaply. Under proper specifications a good bridge can be designed and erected on either plan, and the item of cost will usually determine the decision. The riveted system generally requires a little more material than the pin system, and the latter requires more skilled workmanship. High prices for iron and labor were favorable to the development of the pin system, and as these become lower the riveted system comes more and more into use. The literature noted in the preceding articles contains much information regarding the various methods of joint connections. Further reference is made to the works named in the following pages, and also to a series of articles on Expired Bridge Patents by F. B. BROCK, in Engineering News during 1882 and 1883.

ART. 7. LITERATURE OF BRIDGE DESIGN.

The computation of stresses in the principal members of a bridge truss is the least part of the work of design, and hence books treating mainly on stresses are not noted in the following list. Bridge design includes of course the economic principles regarding the form of the truss, some of which have been mentioned in Art. 4, but more specifically it is the science of

details, that is, the proportioning of the members, the floor, the joint, and of all the splices, reënforcing plates, rivets, pins, and other parts which make up the structure. The list of books below includes such as treat wholly or in part of these topics, together with a few of historical and descriptive character. Although not complete, it is believed that it gives the works on Bridge Design most important for a college library and for the use of American students of bridge design. The list is arranged chronologically according to the date of the first editions.

POPE, T. A Treatise on Bridge Architecture. New York, 1811. This contains 196 pages of descriptions of early bridges, while the remainder is devoted to the author's "patent flying pendant lever bridge."

WHIPPLE, S. A Work on Bridge Building. Utica, N. Y., 1847, pp. 120 and 10 plates. The edition of 1869 contains also 128 pages of notes (printed by the author's own hands) explanatory of the original work. See Art. 2.

DUGGAN, G. Stone, Iron, and Wood Bridges of United States Railroads. New York, 1850. Consists mostly of drawings, with brief descriptive notes.

HAUPT, H. General Theory of Bridge Construction. New York, 1851, pp. 268 with 16 plates, giving examples of railroad bridges.

VOSE, G. L. Handbook of Railroad Construction. Boston, 1857, pp. 480. Contains 109 pages on wood, iron, and stone bridges.

HUMBER, W. Cast and Wrought Iron Bridge Construction. London, 1864, two volumes, with 80 plates, mostly descriptive of English bridges.

HEINZERLING, F. Die Brücken in Eisen. Leipzig, 1870, pp. 515. A historical and descriptive work on bridge development in all countries. Also Die Brücken der Gegenwart. Leipzig, 1884, pp. 754 with 60 plates.

MERRILL, W. E. Iron Truss Bridges for Railroads. New York, 1870, pp. 130. A comparison of seven kinds of trusses with respect to theoretic economy.

BOLLER, A. P. Construction of Iron Highway Bridges. New York, 1876, pp. 144. Although written for the use of town committees, this book has been of much value to young engineering students.

DU BOIS, A. J. Strains in Framed Structures. New York, 1883, pp. 390 with 27 plates. This devotes 124 pages to design, and gives the complete design of a pin-connected bridge. The edition of 1896 has 209 pages on design and erection.

WADDELL, J. A. L. Designing of Ordinary Iron Highway Bridges. New York, 1884, pp. 244 and 7 plates. A book which has done much to improve the design of highway structures.

BENDER, C. Principles of Economy in the Design of Metallic Bridges. New York, 1885, pp. 195 with 9 plates. This does not treat of details, but gives critical theoretic comparisons of different forms of trusses.

RICKER, N. C. Construction of Trussed Roofs. New York, 1885, pp. 158. Mainly deals with stresses, but has two chapters on dimensions and details.

BURR, W. H. Stresses in Bridge and Roof Trusses. New York, 1886, pp. 454 with 12 plates. Devotes 112 pages to details and to the design of a railway bridge.

SCHÄFFER, T., and SONNE, E. Der Brückenbau (Vol. II of Handbuch der Ingenieur Wissenschaften). Leipzig, 1886–90, pp. 1812 with 77 plates.

HIROI, I. Plate Girder Construction. New York, 1888, pp. 94. Gives the design and estimate for a span of 50 feet.

MORANDIÈRE, R. Traité de la Construction des Ponts et Viaducs. Paris, 1888, pp. 1891, with 332 large plates.

COOPER, T. American Railroad Bridges. New York, 1890, pp. 58 with 27 plates. A historical and descriptive work of special value.

FOSTER, W. C. Treatise on Wooden Trestle Bridges. New York, 1891, pp. 160 with 38 plates. Gives many standard plans, accompanied by their bills of material.

JOHNSON, BRYAN and TURNEAURE. Modern Framed Structures. New York, 1893, pp. 517 with 37 plates. This gives 238 pages on details, with designs of several bridge structures.

WARREN, W. H. Engineering Construction in Iron, Steel, and Timber. New York, 1894, pp. 372 with 13 plates. Devotes 92 pages to the details and designs of simple span bridges, besides the designs of several other classes of bridges.

WRIGHT and WING. A Manual of Bridge Drafting. Stanford University, 1896, pp. 214 with 51 plates and 5 blue prints. Gives tables of shears and moments for girders, and details for different types of bridges.

BERG, W. G. American Railway Bridges and Buildings. Chicago, 1898, pp. 705. Gives many illustrations of details of timber structures, and other information compiled from reports of railroad superintendents.

WADDELL, J. A. L. De Pontibus: A Pocket-Book for Bridge Engineers. New York, 1898, pp. 403. Gives general specifications, and many tables and diagrams for facilitating computations.

A number of monographs on large bridges have also been issued in book form, which are of special value to advanced students and engineers. Among these are The Quincy Bridge, by T. C. CLARKE, 1869; The Kansas City Bridge, by O. CHANUTE, 1870; The Omaha Bridge, The Cairo Bridge, The Bellefontaine Bridge, and others, by G. S. MORISON, 1889–93; and The Thames River Bridge, by A. P. BOLLER, 1891.

The Transactions of the American Society of Civil Engineers contain many papers both descriptive and critical. Of the latter class may be noted 'Specifications for the Strength of Iron Bridges,' by JOSEPH M. WILSON, in 1886, Vol. 15, pp. 410–490; 'Some Disputed Points in Railway Bridge Designing,' by J. A. L. WADDELL, in 1892, Vol. 26, pp. 77–282; and 'The Launhardt Formula and Railroad Bridge Specifications,' by H. B. SEAMAN, in 1899, Vol. 41, pp. 140–268. The volumes of Engineering News, Railroad Gazette, Engineering Record, and other technical periodicals, contain numerous articles, both theoretical and descriptive, on bridge design, and some of these will be mentioned in the following chapters. The Index of Engineering Literature, published by the Association of Engineering Societies, in 1892, and by the Engineering Magazine, in 1896 and 1902, gives many pages of titles of such articles, with brief notes of their contents; and this should be at the hand of every student who desires to become well informed on the progress of bridge development. But it cannot be too strongly urged upon the student to form the habit of making his own catalogue of articles, and of giving under each title his own synopsis of its contents and conclusions. By so doing he acquires a training in technical literary work which will be of the greatest value in promoting his professional advancement.

CHAPTER II.

PRINCIPLES OF ECONOMIC DESIGN.

ART. 8. DATA OF THE DESIGN.

In order that the most economic design may be made for a bridge it is necessary that complete data regarding its location should be known. An accurate map of the locality, showing the neighboring roads or streets, should be prepared, as also a profile of the crossing, giving the high and low water marks of the stream and the character of the earth or rock below its bed. This profile should be extended some distance from each bank of the stream in order to enable the approaches of the bridges to be properly arranged. The location of the bridge and of its abutments and piers are to be shown on the map, while the grade line of the bridge and its approaches are given on the profile. If there are more spans than one, the position of the piers is determined by making approximate estimates of their cost in different positions and then applying the principles of Art. 9.

In locating the abutments and piers it is always advisable to avoid a skew, as thereby the cost of the superstructure will be increased. When this cannot be done, as in the case of one street crossing another obliquely or in the case of a stream with rapid current, the angle of skew should be made as small as possible and the same in amount at each end of a span. In locating the grade line of the floor of the bridge the clear waterway desired is to be considered, as also the grades of the approaches; these will also determine whether the bridge is vary from about 8 to 12 feet, and four lines of stringers are used for all spans.

Class B is not employed for lengths exceeding 75 feet, as the saving in depth would not warrant it, class C or D being substituted for it under these conditions. Classes A, B, and D have lengths increasing by increments of 3 to 5 feet, and in classes C and D additional plans are made adapted to curves of 5 and 10 degrees. The weights of the bridges increase in the order of the class letters for any given span, the shipping weights for a span of 60 feet, for example, comparing as the percentages 100, 121, 156, and 176. No expansion rollers are used in any case, but rockers are employed at one end in spans exceeding 75 feet.

CHAPTER VIII.

DETAILS OF RAILROAD PIN BRIDGES.

ART. 70. FORMS OF TRUSSES.

A comparison of the leading bridge specifications and railroad standards indicates that the preferred lower limit of span for plate girders ranges from 15 to 26 feet, that for riveted trusses from 75 to 100 feet, and that for pin-connected trusses from 120 to 150 feet. The New York Central and Hudson River Railroad, however, does not use pin-connected trusses for spans less than about 200 feet.

The riveted trusses are most frequently made either of the Warren type or of the Warren with sub-verticals, the Pratt truss being employed to some extent for the longer spans. The New York Central and Hudson River Railroad introduced in 1899 riveted trusses of the Baltimore type for spans from 100 to 200 feet, which prior to that time had been applied only to pin-connected trusses and to spans exceeding the larger limit named. Some details of riveted trusses are given in Chapter XI.

The Pratt is the prevailing type for the shorter spans of steel pin-connected trusses. The Warren truss with sub-verticals has been used in a few cases like that on the terminal improvements at Providence, R. I. (see Railroad Gazette, vol. 27, page 457, July 12, 1899), and that on the terminal improvements at Richmond, Va. (see Engineering News, vol. 44, page 379, Nov. 29, 1900). Formerly Warren pin trusses were employed more frequently, but it appeared later as though they would go out of use entirely. Pegram trusses are used to a very limited extent on the Union Pacific and several other western railroads.

As the spans increase, the Pratt trusses are modified by curving the upper chord, and for still larger spans the panels are subdivided as in the Baltimore and the Pettit trusses. WADDELL'S specifications indicate the preference of Pettit trusses for all spans above 250 feet, but a few have been built of slightly shorter span. While most of the newer simple truss bridges exceeding 300 feet in span are of the Pettit type, the Baltimore has been used up to 440 feet, as in the case of the Bellefontaine bridge erected in 1893. (See Fig. 105, Art. 80.)

ART. 71. OPEN FLOOR AND STRINGERS.

In through bridges there are generally two stringers to a track spaced from $6\frac{1}{2}$ to 8 feet apart, which support the track ties. The details of the ties, guard rails, etc., are about the same as for deck plate-girder bridges, except that alternate ties are frequently extended the full width for a footwalk. A few railroads, like the Boston and Maine, use four lines of stringers under each track, the main stringers being placed directly under the track rails, while the safety stringers are about $2\frac{1}{2}$ feet outside of the others. The continuity of the spacing of the cross-ties is broken by the floor beams, which support the stringers; but as the top flange of the floor beams is seldom more than a few inches below the tops of the ties, a derailed wheel will pass over the wider space in safety.

In some deck bridges of short span the ties are extended over the full width of the bridge and rest upon the chords of the trusses, as in the case of deck plate girders. As the span increases and with it the spacing of the trusses, this type of floor increases in cost and deflection, and is replaced by one of the same kind as that used for through bridges. In this case

the upper chords of the trusses frequently act also as safety stringers. See the report on bridge floors, to which reference was made in Art. 45.

When the panels are very short, the stringers may consist of I-beams, but generally their construction is similar to that of plate girders of short span. The flanges either consist of two angles or of two angles with one cover plate. The practice of not allowing cover plates is becoming quite prevalent, since it affords a better bearing for the ties, and simplifies the work of track maintenance. In some cases the web is extended $\frac{1}{2}$ or $\frac{3}{4}$ inch above the flange angles, thus obviating the necessity of notching the ties for the full width of the flange.

The stringers of each track are united by a lateral system of the Warren type attached to the upper flanges and by an intermediate cross-frame. Both of these features are used in long panels, and only one of them in short panels, some engineers using the lateral system in this case, while others use the cross-frame only. A cross-frame is also inserted at the ends of end stringers when there is no floor beam at the end of the bridge. The elevation of an intermediate stringer and of part of an end stringer, together with that of a cross or sway frame, is shown on Plate III. It will be noticed that there are no intermediate stiffeners in this example.

While the lateral system of stringers is generally of the simple Warren type, sub-struts are occasionally employed at the other panel points, as well as where the cross-frames are placed. On the inset of Engineering News, Jan. 11, 1900, may be seen an example where a double intersection Warren bracing is used. This arrangement, however, is quite unusual.

In through bridges the ends of the stringers are usually riveted to the webs of the floor beams between their flange angles by means of pairs of connecting angles and of bracket angles, as indicated on Plates III and IV, Art. 82. Sometimes, however, the upper flange angles are extended over the floor beam, and the web cut out so as to clear the flange of the floor beam. (See Fig. 75.) Splice plates connect the tops of the adjacent stringer flanges, thus making them practically continuous and relieving the upper rivets in the connecting angles from tension when the adjoining panels are loaded. This arrangement also permits the ties to be spaced uniformly.

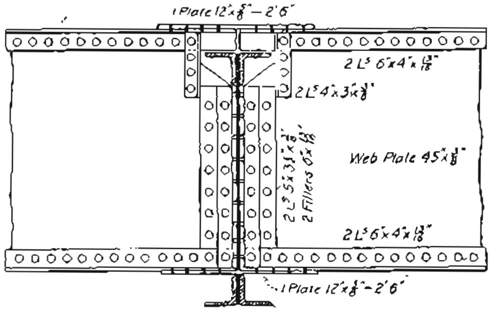


Fig. 75.

In deck bridges the stringers frequently rest on top of the floor beams, as illustrated on Plate V. The lateral system of the bridge may then be connected to the bottom of the stringers, the top of the floor beams, and the bottom of the chords of the trusses, and thereby avoid bending the floor beam horizontally by the tractive force developed on applying the brakes to the train.

When, however, the stringers are connected to the webs of the floor beams and the lateral system is connected to the top flanges of both floor beams and stringers, the web of the stringer may be extended far enough above the regular flange so as to attach secondary flange angles, on which to receive the ties. The projecting web and secondary flange are cut to allow the laterals to pass. This arrangement was adopted in the New Glasgow bridge, whose characteristic details are shown in Engineering Record, vol. 43, page 241, March 16, 1901.

The longest stringers in this country are those of the Delaware river bridge on the Pennsylvania Railroad, their span being 33 feet 3¾ inches.

ART. 72. SOLID FLOORS.

Several types of the trough floors described in Art. 46 are used in pin-connected truss bridges as well as in girder bridges. Some of the references given in Art. 47 contain descriptions and illustrations of their details when so applied. In an article on the Willamette bridge at Portland, Ore., in Railroad Gazette, vol. 21, page 260, April 19, 1889, the drawings show a splayed-channel trough floor riveted to the sides of the stiff lower chord of the trusses. In Engineering News, vol. 36, page 406, Dec. 17, 1896, may be seen the application of a trough system like Fig. 64, Art. 46, to the floor under the double-track railroad of the double-deck highway and railroad bridge at Rock Island, Ill. The floor is laid upon four lines of stringers, and continuous plates, $20'' \times \frac{3}{8}''$, are placed under the rails and riveted to the troughs so as to form an effective lateral bracing.

In the 348-foot span of the Victoria bridge at Montreal, the double tracks are laid on a continuous half-inch floor plate which is supported by transverse 24-inch I-beams spaced only about 14 inches apart. These I-beams are connected to the webs of longitudinal plate girders lying in the planes of the trusses and riveted to the posts below the lower chords. Longitudinal plates, $10'' \times \frac{1}{2}''$, are riveted on top of the floor plates under each rail.

The inset of Engineering News, Aug. 24, 1899, shows the plan of a solid floor built up of 12-inch channels and plates on the upper deck of the Wells Street bridge in Chicago. Two channels with their webs vertical, their flanges toward each other, and their backs 11¾ inches apart are connected by a top flange plate. Similar pairs of channels and cover plates are spaced 12 inches apart in the clear and connected by 12-inch channels with their webs horizontal and their backs at about the mid-height of the vertical channels. The ties for the tracks of the Northwestern Elevated Railroad were laid directly in the shallow troughs of this floor without any ballast.

ART. 73. FLOOR BEAMS.

The floor beams of truss bridges are similar to those of through plate-girder bridges. The objection to cover plates in the case of stringers does not hold for floor beams. In through trusses a part of the web has to be cut away in order to clear the lower chord, and in order to secure enough space for rivets in the end

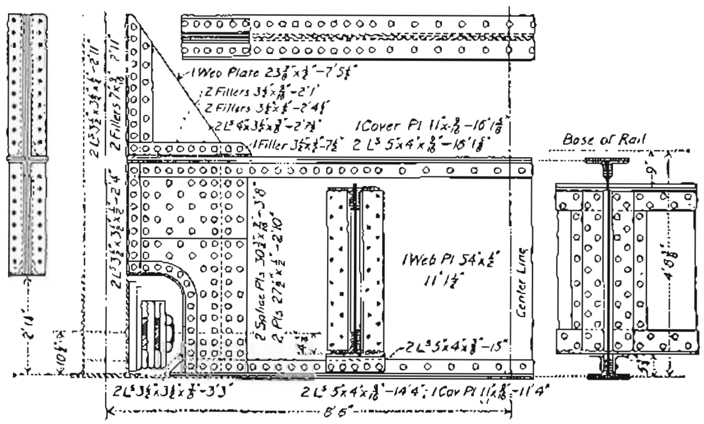


Fig. 76.

connecting angles the web must be extended beyond the upper flange. One form of construction is shown in Fig. 76. The web is spliced so that the end plate may extend up the required distance. The splice plates are continued to the end so as to act also as filler plates and to aid in strengthening the web around the cut. The connecting angles pass over the vertical legs of the upper flange angles, neither of the legs being cut away, and the curved angles pass over the lower end of the connecting angles. An additional pair of filler plates is there-

fore required under the curved angles, and they are extended beyond the angle to give increased strength and to simplify the construction. The lower flange and the bottom of the post are connected by a plate to which the diagonals of the lateral system are also attached.

In Fig. 77 is shown the end of a floor beam in the Pratt truss whose side elevation is given in Fig. 111. The inner edge of the extended web plate is stiffened by a pair of small angles. Several other special features will be noticed, especially the stiffeners between the stringer connections. Another form for a through bridge is shown on Plate III. Sometimes the lower flange angles are bent up outside of the stringers to take the place of the separate inclined angles, in which case another pair of short horizontal angles is riveted to the bottom of the web plate as shown on Plate IV. See also Engineering Record, vol. 43, page 244, March 16, 1901. When the floor beam is not extended down past the lower chord, the eccentric connections of the lateral system cause a bending moment in the bottom of the post which is avoided in the forms just described.

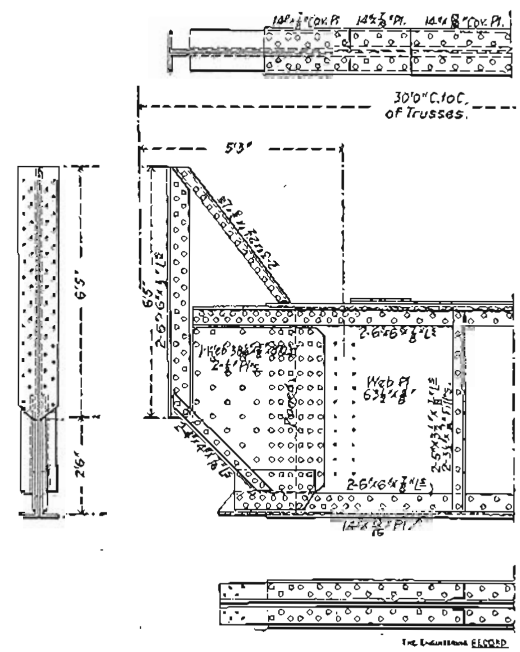


Fig. 77.

In the Port Perry bridge over the Monongahela river this result is secured in another way. A trapezoidal web plate stiffened with angles is riveted to the bottom of the floor beam just inside of the lower chord and also to the horizontal connecting plate of the lateral system which is attached to the bottom of the post. The effect of this construction is to cause a negative bending moment in the floor beam which neutralizes a part of the positive bending moments due to the dead and live loads. (See Fig. 78.)

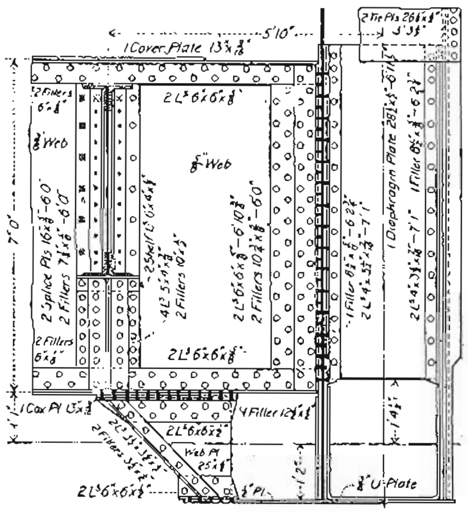


Fig. 78.

The floor beam of a deck bridge is shown on Plate V. The upper corner of the web plate is cut away to clear the diagonal eye-bars of the truss. The stiffeners below the stringers are required to distribute the concentrated loads to the web of the floor beam, fillers being put under the angles. An example in which the top of the floor beam is level with the top of the upper chord is given in Engineering Record, vol. 41, page 126, Feb. 10, 1900.

In double-track bridges the floor-beam flanges may be increased by means of side plates, as in plate girders. In the Bellefontaine, the Alton, and the Delaware river bridges this arrangement is adopted for the upper flanges only, while in the Rankin bridge it is adopted for both flanges. See Engineering Record, vol. 44, page 467, Nov. 16, 1901.

When a floor beam is not riveted to a post, but to some plates or to a short member which resembles a post in construction, but connects with a tension member, like the sub-vertical in a Baltimore or in a Pettit truss, or the suspender of a Pratt truss, as in Fig. 111, Art. 82, the floor beam is effectually stayed against rotation by rods extending to the adjacent panel points. The connection of a floor beam with the extension of a post below the lower chord is illustrated in Railroad Gazette, vol. 25, page 651, Sept. 1, 1893.

In all of these examples a diaphragm is required in order to carry its share of the load from the floor beam to the outer half of the post. It consists of a web plate united by a pair of angles to the two sides of the post.

Not many years ago end floor beams were employed in only a few cases, and those in trusses of large span. Now they are frequently used in short spans as well, and a number of railroads have adopted them as the standard construction.

ART. 74. INTERMEDIATE POSTS.

The simplest form of post consists of two channels, whose flanges are united by short plates at or near the ends, called tie plates, and by lattice bars between. When the flanges are

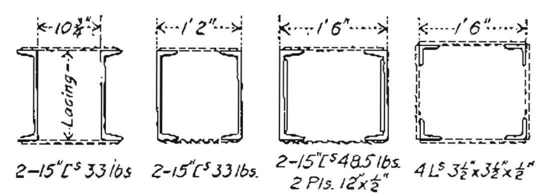
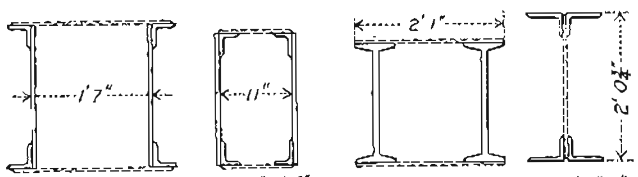


Fig. 79. Fig. 80. Fig. 81. Fig. 82.

turned out (Fig. 79) as in the older practice, it is necessary to cut the channel flanges near the joints, as indicated in Fig. 111, Art. 82. When the flanges are turned in, as in Fig. 80, this cutting may be avoided and a stronger column secured for the same out-to-out measurements. When the largest channels do not furnish sufficient area, the section is sometimes increased by adding two plates, preferably on the inside, as in Fig. 81. When still larger sections are required, the post is built up with plates and angles, as shown in Fig. 83. This form is sometimes said to consist of built channels. In Fig. 84 the angles are turned in, the advantage of so doing being the same as for rolled channels. The increasing area required for the posts toward the end of the span is obtained by increasing the thickness of the parts, or in case the thickness becomes excessive, by



4 Ls 4"x4"x 11/16" 2 Pls. 24"x 5/8" — Fig. 83.

4 Ls 4"x3"x 5/8" 2 Pls. 22"x 13/16" — Fig. 84.

2-20" Is 90 lbs. — Fig. 85.

4 Ls 6"x4"x 3/8" — Fig. 86.

adding an additional plate on each side, either of the full width of the side plates or to fill only the clear width between the angles.

The posts of the Victoria Jubilee bridge at Montreal have an unusual composition. Two I-beams are laced together (see Fig. 85) for each of the posts, 20-inch and 18-inch I-beams being employed in the posts near the ends and middle of a span respectively.

Fig. 84 shows how relatively narrow a post is sometimes made so as to be packed with the connecting diagonals in the upper chord. See Engineering Record, vol. 41, page 126, Feb. 10, 1900. On the other hand a post like Fig. 83, whose plates are only 22 inches wide, has the backs of the angles spaced $31\frac{1}{4}$ inches, in order to enter the outer spaces of the upper chord with its four webs. See Engineering Record, vol. 41,

page 516, June 2, 1900. Figs. 82 and 86 show additional post sections, which are mainly used for the sub-verticals of Baltimore and Pettit trusses, which support the upper chord midway between the long posts. The former section has also been used for collision struts.

Elevations of intermediate posts showing the tie plates and lattice bars which connect the two halves of the posts, as well as their diaphragms opposite the floor-beam connections, may be seen on Plates III, IV, and V, and in Fig. 111.

ART. 75. MAIN AND COUNTER DIAGONALS.

The simplest form used for a main tie consists of one or more pairs of eye-bars (Plate III). Tables of the standard sizes of eye-bars may be found in all of the handbooks. Sometimes, in order to secure stiffness in the panels of short spans requiring no counter bracing, the eye-bars are connected by riveting an angle to each bar and uniting the angles with lattice bars. In the panels which require counterbracing the same result is secured by using two pairs of angles laced together to form an I-section. (See Fig. 86 and Plate III.) In members with larger sectional areas a solid web plate is substituted for the lacing.

When the main ties are eye-bars the counters in the same panel consist either of an adjustable eye-bar, or of a square bar with loop eyes, when the required section is small. When laced angles are used for the main ties, the counters have the same composition.

Another method of securing greater stiffness has been adopted to some extent in which the counter ties are omitted and the main diagonals designed to take both tension and compression. The member is then made up either of two rolled channels laced together or of built-up channels, each one being composed of a web plate and two angles. The bridge over the Missouri river at Bellefontaine, Mo., may be mentioned as a prominent example in which counterbraced diagonals are used, whose composition is the one mentioned last.

The larger vibration due to adjustable counters and the great difficulty in keeping them in proper adjustment has led to the design of the other forms, and so far as they have been compared under traffic, there is little or no difference between the action of Pratt trusses having counterbraced diagonals which take both tension and compression and those in which both main and counter ties are riveted members.

ART. 76. SUSPENDERS.

In the through Pratt truss the suspender or hip-vertical is the vertical tie which connects the upper end of the inclined end post and the second panel point of the lower chord. In the Baltimore and Pettit trusses there are not only the long suspenders, but a number of short ones whose duties are similar. These members have all the forms of section which were mentioned for the diagonals, whether counterbraced or not. If eye-bars are used, they are frequently connected by bent bars instead of by angles and the ordinary forms of lattice bars. (See Fig. 111, Art. 82.) When channels are employed, the flanges may either be turned in or out, and the same is true when the channel section is built up. The sectional area of built-up channels is increased sometimes by using double webs. When the I-section is used in a large truss, two flange plates are added to the two pairs of angles. For examples of the forms mentioned see Plates III and V, and Engineering Record, vol. 41, page 516, June 2, 1900, and vol. 37, page 384, April 2, 1898.

As the suspender in a through Pratt truss receives its stress only from loads in the first two panels, its stress changes more rapidly than that of any other member, and it also receives its

impact more directly. In order to reduce the excessive vibration thus produced some railroads require the suspender to be made of a riveted post section in all cases. This arrangement also prevents rising driftwood from buckling the floor and pulling the bridge off the pier.

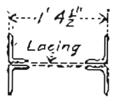
In a deck Pratt truss with inclined end posts the only duty of the suspender is to support the lower chord members, and hence in this case it is made of a square bar with either upset or loop-welded eyes, or of two angles laced together so as to form a member about as wide transversely as the intermediate posts. The stiff member is preferable.

ART. 77. LOWER CHORD MEMBERS.

In simple pin-connected steel trusses the lower chord members are very seldom made of anything else than eye-bars, except in the two panels at each end. The depth of eye-bars used in trusses of ordinary span generally does not exceed 8 inches. On the other hand, the smaller depths are not now used to such a great extent as formerly, since it is considered desirable to use few comparatively heavy bars rather than a larger number of light ones. (See Plate IV.)

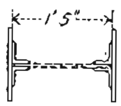
The largest eye-bars that have been used in any simple truss bridge in this country are those of the Delaware river bridge on the Pennsylvania Railroad, their depth being 12 inches, the greatest thickness $2\frac{1}{2}$ inches, and maximum finished weight of one bar, 56 feet long, 5500 pounds. Eye-bars 10 inches deep are used in the Louisville, Bellefontaine, Alton, and Rankin bridges, the greatest thickness being respectively $2\frac{1}{2}$, $2\frac{5}{8}$, $2\frac{11}{16}$, and $2\frac{1}{8}$ inches. In the Bellefontaine bridge the bars extend over two panels of the Baltimore trusses, being 55 feet long between centers of pins. In Fig. 111 are shown two pairs of eye-bars $51' \ 3\frac{3}{4}''$ long, the inner ones being riveted to the suspender and the outer ones resting on the horizontal legs of a pair of connecting angles.

In the best practice the lower chord members in the first two panels at each end of the span are designed to resist both ten-



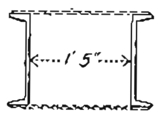
4 Ls 5″×3½″×⅝″

Fig. 87.



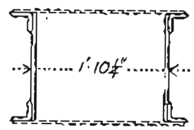
4 Ls 3½″×3½″×⅜″
2 Pls. 12″×½″

Fig. 88.



2-15″ [s 45 lbs.

Fig. 89.

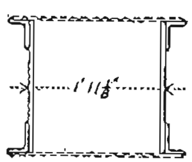


4 Ls 3½″×3½″×7/16″; 2 Pls. 18″×9/16″

Fig. 90.

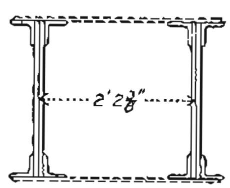
sion and compression. This construction enables the lower chord to resist the compression caused by the traction load when the brakes are applied to the train, or the thrust of a derailed car on the bridge, or that caused by a derailed car striking the end of the truss. It also reduces vibration, and increases the stiffness of the truss, especially in short spans. The principal forms of section are shown in Figs. 87 to 93 inclusive. Fig. 91 gives the section used in the end panels of the Alton bridge, and Fig. 92 those in the Bellefontaine bridge.

In a few cases the lower chord of pin-connected trusses is constructed with plates and angles from end to end. Fig. 93



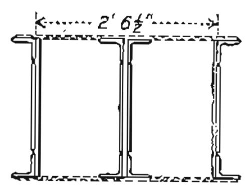
4 Ls 5″×3½″×½″
2 Pls. 22″×11/16″

Fig. 91.



8 Ls 4″×4″×⅝″; 4 Pls. 26″×¾″

Fig. 92.

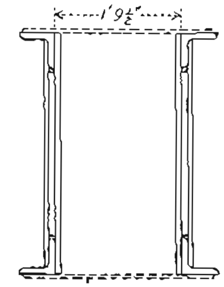


4 Ls 4″×4″×⅝″; 4 Ls 4″×4″×½″
3 Pls 23″×11/16″; 4 Pls. 14¾″×⅝″

Fig. 93.

gives the section in a panel toward the middle of one of the fixed spans of the United States bridge at Rock Island. In the end panels of the bridge only two webs are employed. Fig. 94

gives the section of the lower chord of the International bridge at Buffalo. The chord is made very deep in order to resist the flexure caused by the floor beams, which are spaced only half the distance between the panel points of the trusses. This construction was used to secure a shallow floor. The floor beams consist of 24-inch I-beams, and the stringers of 4 lines of 15-inch I-beams. See Engineering Record, vol. 43, page 567, June 15, 1901.



4 Ls 6″×6″×⅝″; 2 Pls. 40″×⅞″
2 Pls. 27½″×⅞″; 2 Pls 38″×7/16″

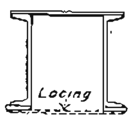
Fig. 94.

The bridge department of the Baltimore and Ohio Railroad has designed some spans in the vicinity of 150 feet in which the use of eye-bars is restricted to the end ties and the entire bottom chord, all the bars being laced together in order to eliminate as far as possible the vibration of these members. Sometimes the eye-bars in the end panels only are laced instead of using members composed of plates and shapes, as shown in Fig. 111, Art. 82. The use of bottom chords which are stiff throughout is also referred to in Chapter XI.

ART. 78. UPPER CHORD AND END POSTS.

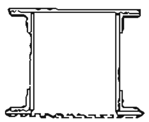
One of the simplest sections of an upper chord member is shown in Fig. 95. The flats below the channels are used to balance the section about a horizontal axis passing through the centers of the channel webs. These are often omitted, but unbalanced sections are not regarded favorably by the best designers. When the section is so small that the required thickness of the metal is less than the minimum allowed, the cover plate and flats are omitted and then the top of the member is laced as well as the bottom.

The compositions indicated in the two examples given in Figs. 96 and 97 are much more frequently employed for ordinary spans. In the one case the section is balanced by means of flats, while in the other the lower angles are increased in size for the same purpose. The former method is preferred, as it simplifies the construction at the joints where pin plates



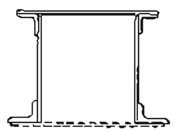
1 Cover Plate $19'' \times \frac{3}{8}''$
2–15" [s 33 lbs.
2 Flats $4'' \times \frac{3}{4}''$

Fig. 95.



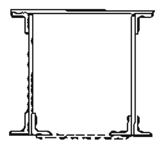
1 Cover Pl. $22'' \times \frac{3}{8}''$
2 Web Pls $16'' \times \frac{3}{8}''$
4 Ls $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$
2 Flats $4\frac{1}{2}'' \times \frac{3}{4}''$

Fig. 96.



1 Cover Plate $24'' \times \frac{7}{16}''$
2 Web Plates $18'' \times \frac{1}{2}''$
2 Angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$
2 Angles $5'' \times 3\frac{1}{2}'' \times \frac{7}{8}''$

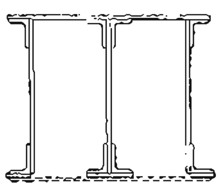
Fig. 97.



1 Cover Plate $24'' \times \frac{7}{16}''$
2 Web Plates $20'' \times \frac{5}{8}''$
4 Angles $3\frac{1}{2}'' \times 3'' \times \frac{7}{16}''$
2 Ls (inside) $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$

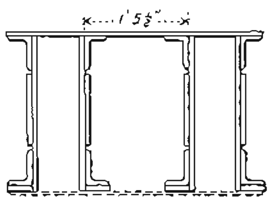
Fig. 98.

must be attached to the sides in order to secure sufficient bearing on the pins. In Fig. 98 the section is balanced by using two angles instead of one at the bottom of each web plate. At the panel points the horizontal legs of the inner angles are cut to afford the necessary clearance for the posts and diagonals. The latticing is connected to the inner angles only. This section is taken from the Northern Pacific Railway's standard plan for a 200-foot through pin bridge. (See Plate IV.)



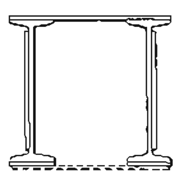
1 Cover Plate $34'' \times \frac{3}{8}''$
2 Web Plates $26'' \times \frac{9}{16}''$
1 Web Plate $26'' \times \frac{3}{4}''$
8 Angles $5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$
2 Flats $4'' \times \frac{5}{8}''$
1 Flat $8'' \times \frac{5}{8}''$

Fig 99



1 Cover Plate $43'' \times \frac{1}{2}''$
2 Web Plates (inside) $25\frac{1}{4}'' \times \frac{7}{8}''$
2 Web Plates (outside) $25\frac{1}{4}'' \times \frac{3}{4}''$
4 Plates $12'' \times \frac{3}{4}''$
8 Angles $6'' \times 4'' \times \frac{3}{4}''$
4 Flats $4'' \times 1''$

Fig 100.



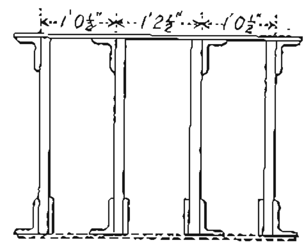
1 Cover Plate $27\frac{1}{2}'' \times \frac{5}{8}''$
2–24" I-Beams 100 lbs
2 Web Plates $20 \times \frac{5}{8}''$
2 Flats $7\frac{1}{2}'' \times \frac{5}{8}''$

Fig. 101.

Additional area is obtained not only by increasing the thickness of the plates and shapes, but also by putting additional

web plates in the clear space between the angles or by placing a web plate of the full depth inside of each of the others. Fig. 99 shows a section containing three webs, and in this case also the outer webs are strengthened in the manner just described. The maximum upper chord section of the Bellefontaine bridge is given in Fig. 100. That of the Delaware river bridge is similar to this except that the inner upper angles are placed on the outside of the inner webs as indicated on Plate V, which shows some details of another bridge on the same division of the Pennsylvania Railroad.

Fig. 102 gives the composition of the largest section of the upper chord of the Monongahela river bridge at Rankin, Pa., its sectional area being 334.52 square inches. It is the largest chord section of any simple truss in existence. It will be noticed that the flats are placed opposite the vertical legs of the angles instead of being riveted to their horizontal legs. The chords of the heavy truss in the Monongahela river bridge at Port Perry, Pa., are a little wider, but the depth and area are less. The composition is as follows: 1 cover plate, $50'' \times \frac{5}{8}''$; 2 pairs of outer web plates, $30'' \times \frac{13}{16}''$; 2 pairs of inner web plates, $30'' \times \frac{5}{8}''$; 4 upper angles, $4'' \times 4'' \times \frac{5}{8}''$; 2 outer lower angles, $6'' \times 4'' \times \frac{3}{4}''$; 2 inner lower angles, $6'' \times 6'' \times \frac{7}{8}''$; 2 outer flats, $6'' \times \frac{1}{2}''$; and 2 pairs of inner flats, $6'' \times \frac{5}{8}''$. The arrangement of the shapes is similar to that in Fig. 102, except that the outer flats are placed between the outer angles and the web plates. Five intermediate lines of rivets, with a large pitch, are used to connect the several pairs of web plates. The light truss in the same bridge has only three webs. In both bridges the



1 Cover Plate $48'' \times \frac{5}{8}''$
4 Web Plates $33'' \times 1\frac{1}{4}''$
8 Angles $6'' \times 4'' \times \frac{3}{8}''$
4 Flats $6'' \times \frac{3}{4}''$

Fig. 102.

ends of the chords and end posts where pin bearing is required have short angles placed opposite the upper angles and extended the full length of the pin plates.

The new trusses of the International bridge at Buffalo, erected in 1901, have upper chords of a very unusual section, shown in Fig. 101. Toward the ends of the span the side plates are reduced, and finally omitted. The lacing at the bottom consists of $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$ angles. At the panel points portions of the inner flanges of the I-beams are cut away to provide the clearance needed to pack the web members.

When the chords and end posts have either three or four webs, it is important that their ends be prevented from shifting their relative position after the pin holes are bored, or else trouble is caused in erection. The same conditions apply to the sections where the chords are spliced. This is accomplished by means of transverse diaphragms, as indicated on Plate V. It will be noticed that between the inner and outer webs the diaphragms consist simply of two angles, while between the inner webs plates are also used.

The construction of end posts is usually the same as that of the chords in the same span, the variations rarely being more than those between the upper chord members in different panels. Occasionally the width of the end post may be different from that of the upper chord, but this is rather exceptional.

ART. 79. LATERAL BRACING.

Formerly the upper lateral ties of through bridges consisted of adjustable square bars or round rods connected either to the top of the upper chord or to the middle of its inner web by means of connecting plates and pins. In long spans two sets of ties were often used connected to the top and bottom of the chord respectively. This construction is now seldom employed,

nearly all the standard specifications for railroad bridges stating that stiff members are preferred for the lateral bracing. Those who still use the adjustable members claim that they are not only much lighter, but that the upper chord can be more thoroughly lined up by this means. The object of the stiff laterals is to secure greater lateral stiffness in the bridge, as well as to avoid the difficulty of maintaining the rods in proper adjustment. Many specifications state that it is preferable to avoid altogether the use of adjustable members in trusses, lateral and sway bracing.

Stiff lateral diagonals are most frequently composed of single angles as illustrated in Plate III, and Fig. 111. Sometimes two angles placed, back to back are employed. In order to give greater vertical stiffness to these members a section like Fig. 86 is used, consisting of two pairs of angles laced together, the depth of the section being equal to that of the upper chord so that the connection with it may be made on both top and bottom. (See Plate VII, Chap. XI.) Laterals of this type are used in the Delaware river bridge. Occasionally in short spans the composition is modified by latticing two single angles instead of two pairs of angles. This form is used in the Monongahela river bridge at Rankin, Pa., the size of both angles being $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$. The long span over the same river at Port Perry has adjustable rods.

The various sections described are used also for lower laterals of through bridges. In many cases where adjustable laterals are still used in the upper system, stiff members are employed in the lower system. (See Plate IV.) This statement also applies to the bridge at Port Perry, whose lower laterals consist of two pairs of angles latticed together. As these laterals are not connected to the stringers, they are stiffened in a horizontal direction by means of four horizontal members of similar composition which are connected at their extremities to the laterals at their quarter points, thus forming a rectangle in plan. The laterals stiffen each other also by the connection at their centers. Attention is called to the forms of splices used for both upper and lower laterals on Plates III and VII.

The connections of the lower laterals to through trusses is often very eccentric, causing large horizontal bending moments in the ends of the floor beams. This is avoided, in the best designs, by using larger connecting plates, and by incurring the cost of somewhat greater inconvenience in field riveting. In the upper lateral system the effect of eccentricity is not so serious, since the stresses are smaller and the connection is made to the stiff upper chord. Let the student observe the character of the lateral connections in this respect on Plates III, IV, VI, and VII.

The construction of the upper lateral system in deck bridges is practically the same as that of the lower system in through bridges, and that of the lower system of deck bridges the same as that of the upper system of through bridges. Sometimes the lower laterals are omitted in alternate panels, while in other cases they are omitted entirely. The latter arrangement is adopted in the standard plans for pin-connected deck bridges on the Northern Pacific Railway.

The lateral struts which are perpendicular to the upper chords of through trusses form also a part of the transverse or sway bracing. Sometimes the rest of the sway bracing consists merely of brackets connecting the lateral struts to the posts of the trusses, while at other times this is connected to a lower or intermediate strut by means of two or more web members as shown in the next article. In short spans the lateral strut is composed of two pairs of angles placed back to back and laced together as in Fig. 86, its depth being equal to that of the upper chord to whose upper and lower flanges it

is riveted by connecting plates. (See Plates III and VII.) Occasionally the upper angles are placed with their horizontal flanges on the lower side, extended across the top of the chord and riveted directly to it. Where the upper chord is rather deep and the trusses are separated by double tracks, the angles are often placed in the corners of a rectangle as in Fig. 82, Art. 74, and laced on the four sides. Two channels laced together are occasionally used. Another form of section is that in which the lacing of the first form mentioned is replaced by a solid web, forming practically a small plate girder.

When the web connections of the sway bracing are rather close together, the lateral strut is sometimes reduced to a single pair of angles (Plate IV) or to one pair of angles with a web plate between, the latter form being shown on Plate VII. In double-track bridges this section is increased in stiffness horizontally by using bulb angles instead of the ordinary angles.

The composition of lower lateral struts in deck bridges comprises all the forms mentioned above except those containing a solid web plate with either one or two pairs of flange angles.

ART. 80. PORTAL AND SWAY BRACING.

When the required clearance extends to within two or three feet of the top of the lateral strut, the intermediate sway bracing of a through bridge consists merely in connecting the strut to the post at each end by means of a bracket or knee brace. (See Plates III and VI.) When there is more head room one of the simplest styles of bracing consists of a lattice girder, with a double system of webbing, as shown on Plate IV. The lower flange is placed as low as the head room will allow. With increasing depth four systems of webbing may be used, an example of which is given on Plate VII. For other examples, see the inset of the Engineering News, Jan. 11, 1900. Where the depth is large, the lower strut is sometimes made like the upper or lateral strut. It will be noticed that the bracing on Plate VII also contains a small bracket. The use of brackets is generally confined to cases where the depth is small.

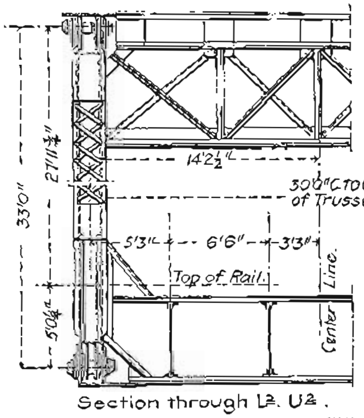


Fig. 103

Another type is shown in outline in Fig. 103, and its details are given in Fig. 104. Sometimes the verticals are omitted in the webbing, thus reducing it to the Warren type of truss. The number of panels depends on the depth of the bracing and on the width of the bridge. An example of this form may be seen in Engineering Record, vol. 37, page 386, April 2, 1898.

The small connecting plates shown in the top view and section are intended to connect with a longitudinal strut which helps to stiffen the lateral struts in a horizontal direction, since it is also attached to the lateral diagonals at their intersection.

Fig. 105 shows two forms of intermediate sway bracing, one between the long posts of the trusses in which a quadruple system of diagonals is used, and the other between the sub-vertical struts with only two diagonals. In both cases the upper and lower struts are composed of a plate and a pair of bulb angles. In some cases the single pair of diagonals is used throughout the span, and occasionally a sub-vertical is suspended from the intersection of the diagonals to support the center of the lower strut. With further increase in depth the sway bracing is sometimes divided into two panels, one above the other, by means of an intermediate horizontal strut. In the Engineering

Record, vol. 44, page 467, Nov. 16, 1901, may be found an illustration of the sway bracing at the middle of the span of the Rankin bridge. The lateral strut consists of two pairs of angles $6'' \times 4'' \times \frac{3}{8}''$, connected by a system of double intersection lacing with $3'' \times 2'' \times \frac{5}{16}''$ angles; the other two struts consist

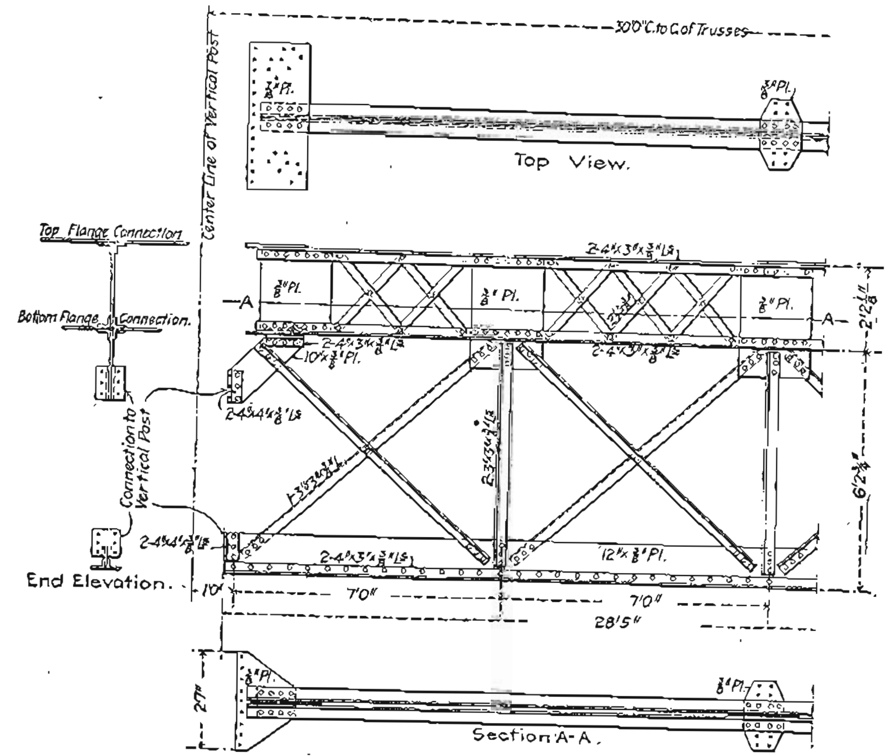


Fig. 104.

of two pairs of $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles laced with bars so as to be 18 inches deep, and the two diagonals in each panel are composed of two $3'' \times 3'' \times \frac{3}{8}''$ angles placed back to back. Toward the end of the span the bracing has only one panel, and at some intermediate points a single pair of diagonals crosses the two panels intersecting the middle strut at its center.

In the shorter spans of the Victoria Jubilee bridge at Montreal the upper strut is made up of two pairs of $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles laced, the lower strut of one web plate, $10'' \times \frac{3}{8}''$, and two flange angles, $6'' \times 3\frac{1}{2}''$, each of the two intersecting diagonals of one angle, $4'' \times 3'' \times \frac{3}{8}''$, and the sub-vertical of one angle, $3'' \times 3'' \times \frac{3}{8}''$. In the long span the composition is the same except that the angles and plate are increased in size.

The general character of the sway bracing of deck bridges is about the same as for through bridges. One example of both the intermediate and the end sway bracing is shown on Plate V. Another example may be found on the inset of the Engineering News, Nov. 29, 1900, and a third one in the Engineering Record, vol. 41, pages 125 and 126, Feb. 10, 1900.

Adjustable rods are still used to some extent in the sway bracing of both through and deck bridges, but the practice is not generally regarded with favor.

A number of the forms employed for intermediate sway bracing are also used in portal bracing, the details being made stronger, however, on account of the greater duty of the latter. Plate IV shows a portal having flanges with unequal-legged angles of ample size, and with deep plates to receive the connections of the web members, which consist of two systems of diagonals. The wide plates are continued around the ends of the portal, and extended into the bracket, so as to make a very rigid connection with the inner sides of the end posts. As indicated on the plate, this is a standard design of the Northern Pacific Railway. In the reference to Engineering Record mentioned in the preceding paragraph may be seen the view and details of a portal only about $4\frac{1}{2}$ feet deep at the middle, with double intersection webbing. The lower flange is curved down at the ends to form the flanges of the brackets, and solid web plates form the bracing in the end panels. The standard portal of the

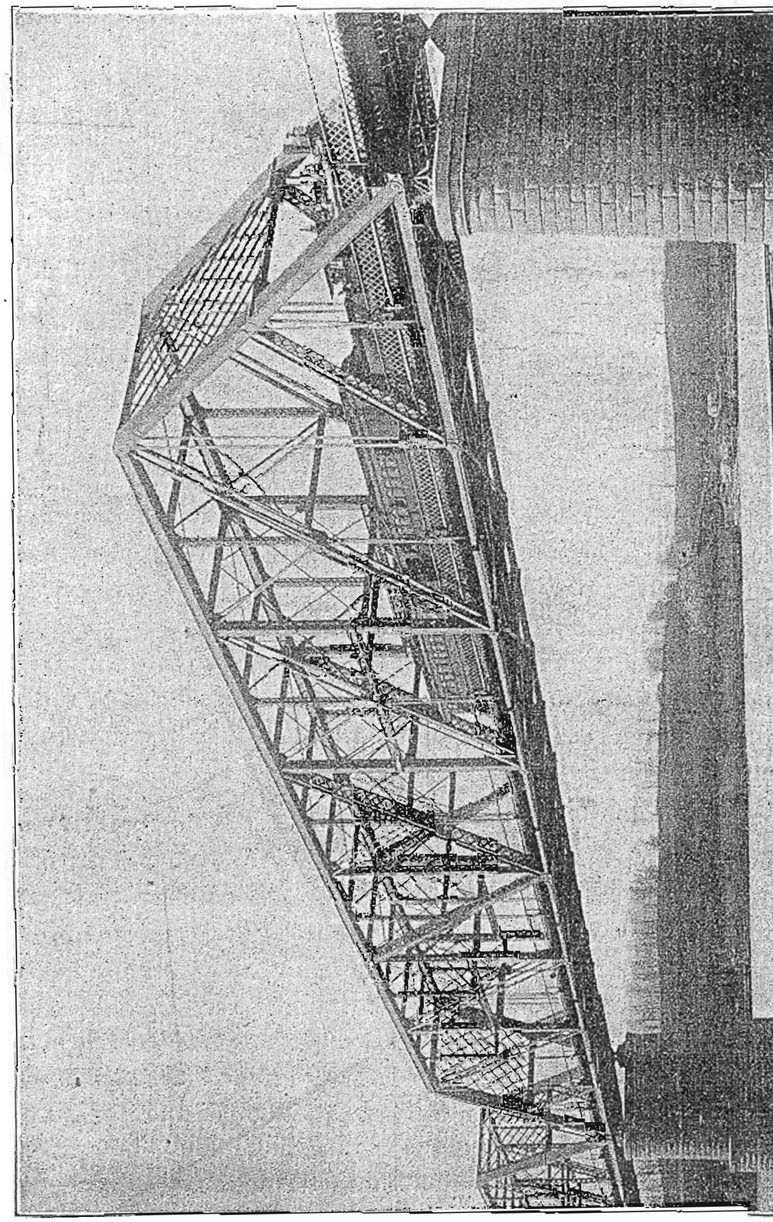


Fig. 105. Double-track Through Bridge over the Missouri River at Bellefontaine, Mo.

New York Central and Hudson River Railroad is given on Plate VII , Chap. XI.

In some cases where the head room is limited the portal bracing consists practically of two complete double intersection lattice girders with their connecting end brackets, one riveted to the top and the other to the bottom flanges of the end posts, the corresponding flanges of both girders being united by lacing. (See Fig. 6, Art. 3.) A plate is sometimes substituted for the upper lacing. An example of a double portal bracing, but of somewhat different design, is shown on Plate VI . Under similar conditions of limited head room the portal bracing is occasionally composed simply of a plate girder and of brackets with solid webs.

Perhaps the best illustration of the application of a lattice portal bracing to a bridge of long span is that of the Bellefontaine bridge shown in Fig. 105. The top strut consists of a web plate $30'' \times \frac{1}{2}''$ and two bulb angles $9'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, the lower strut of one plate $27'' \times \frac{1}{2}''$, one angle $4'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, and one bulb angle $9'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, and each of the twenty diagonals of one angle $5'' \times 5'' \times \frac{1}{2}''$ angle. The plates extend around both sides of the bracing similar to that on Plate VII , and with neatly rounded corners.

The present practice in the design of portals for bridges whose depth affords adequate room consists in using relatively few members with sufficient strength to secure that degree of lateral rigidity which is now regarded as so essential. The members are all made of the same depth as the end posts, so as to permit them to be riveted to both the top and bottom flanges of the end posts. An excellent example of such a design is the portal of the United States bridge at Rock Island, Ill. The view given in Fig. 106 is that of the portal of the draw span, but it has the same construction as those used on the fixed spans. In the $216\frac{1}{2}$-foot fixed spans the lower strut has one

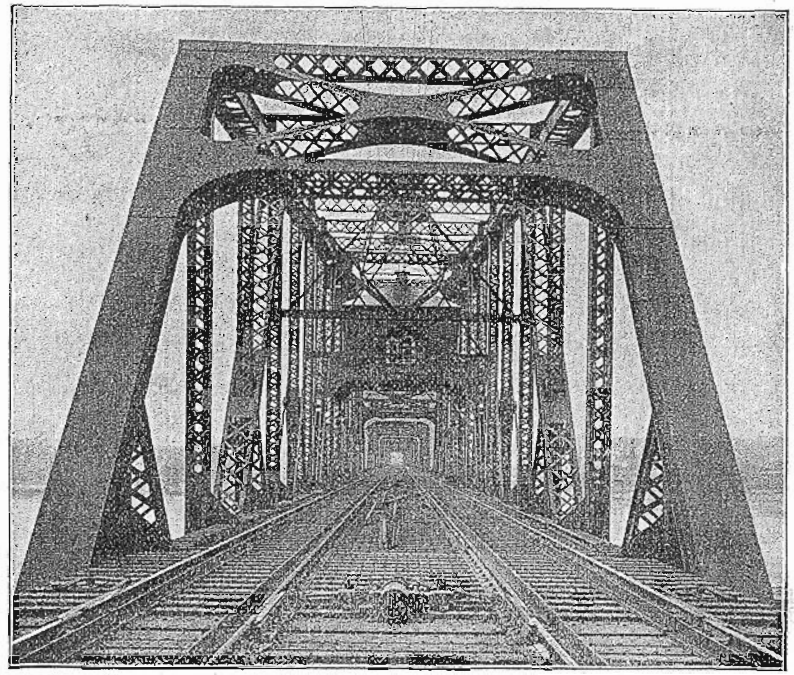


Fig. 106. Portal of United States Bridge at Rock Island, Ill.

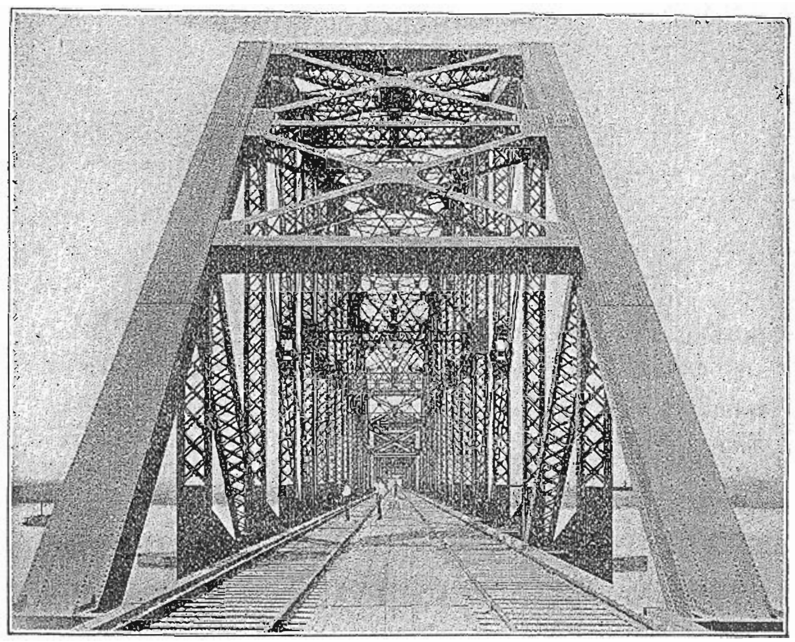


Fig. 107. Portal of Delaware River Bridge, Philadelphia.

cover plate $16\frac{1}{2}'' \times \frac{3}{8}''$, four angles $3'' \times 3'' \times \frac{3}{8}''$, and is laced on three sides. The diagonals have two pairs of angles $5'' \times 3'' \times \frac{3}{8}''$, with one line of lacing. The upper strut has an upper cover plate $17\frac{1}{4}'' \times \frac{3}{8}''$, three angles $3'' \times 3'' \times \frac{3}{8}''$, one angle $4'' \times 4'' \times \frac{3}{8}''$, and a lower cover plate $7'' \times \frac{3}{8}''$. It is laced on two sides, one side being perpendicular to the flanges of the end post, and the other in the plane of the beveled end of the end post. The provision of connecting plates with curved edges indicate that some attention was paid to æsthetic considerations in this design.

The portal of the Delaware river bridge near Philadelphia is divided into two panels, one above the other. Fig. 107 indicates that the lower strut is practically a plate girder whose depth equals that of the end posts, while the middle strut and the diagonals consist of two pairs of angles laced together. The top strut is of novel design. In composition it resembles that of an upper chord member, but the two web plates are respectively perpendicular to the flanges of the end post and of the upper chord, and both the cover plate and the lower lacing are bent to the angle made by the end post with the adjoining upper chord member. Square connections could thus be made on one side with the portal diagonals and on the other side with the top laterals, which also consist of two pairs of angles laced as deep as the chords.

Another portal containing some new details is that of the Union Railroad Bridge at Rankin, Pa., shown in Fig. 108. Both the upper and the lower struts consist practically of two plate girders whose flanges, each having only one angle, are extended across the end posts, and riveted to them on the upper and lower sides respectively. The girders have their corresponding flanges laced together with a single system of diagonals composed of single angles. Double triangular brackets with solid webs and connecting plates are also used. In addition to the diagonals of the portal, a strut of the same composition as

the diagonals connects the middle of each horizontal strut with the intersection of the diagonals.

The portal bracing of the bridge erected by the same railroad at Port Perry differs from this one by substituting for the strut

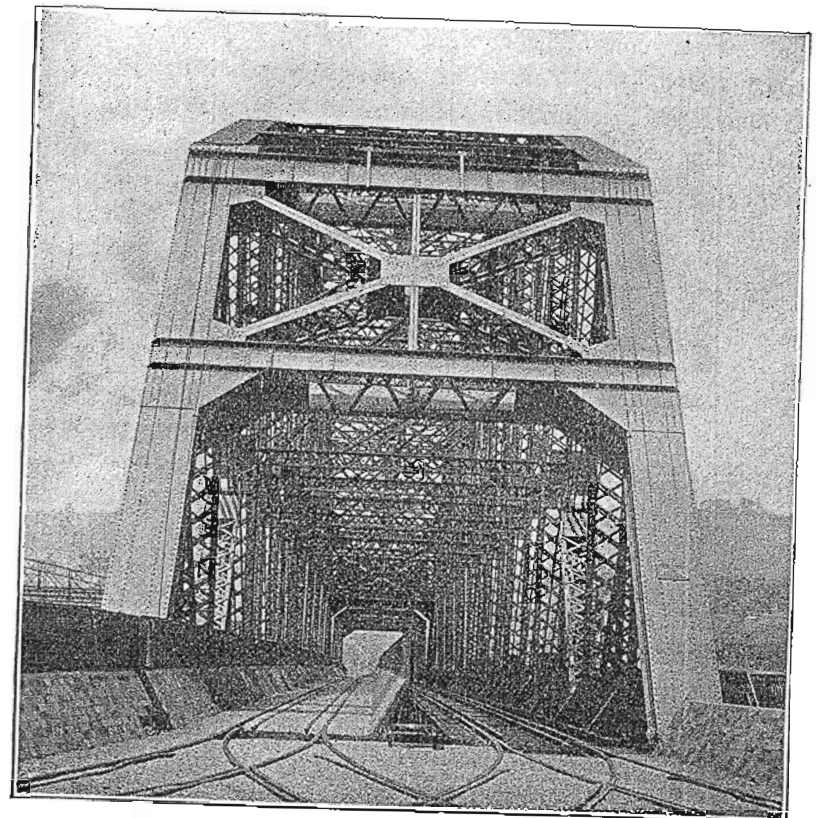


Fig. 108. Portal of Rankin Bridge.

just mentioned one of only two angles laced in a similar way, but extending horizontally across the intersection of the diagonals, and riveted at each end to the top and bottom of the end post. The upper strut is also different in containing only four angles laced on the four sides.

This bridge contains the unusual feature of a double plate-girder portal bracing, connecting the feet of the end posts on their top and bottom flanges. The web plates of each of these girders are not continuous, but are connected by angles to the webs of the stringers, and thus to each other. The flanges, however, are continuous and are field-riveted to the webs. They consist of single angles. The function of this bracing is performed in many other bridges by an end floor beam riveted to the end posts.

The composition of the sway and portal bracing of the Victoria Jubilee bridge is given in Engineering Record, vol. 38, page 488, Nov. 5, 1898. Each of these contains only two struts and two intersecting diagonals.

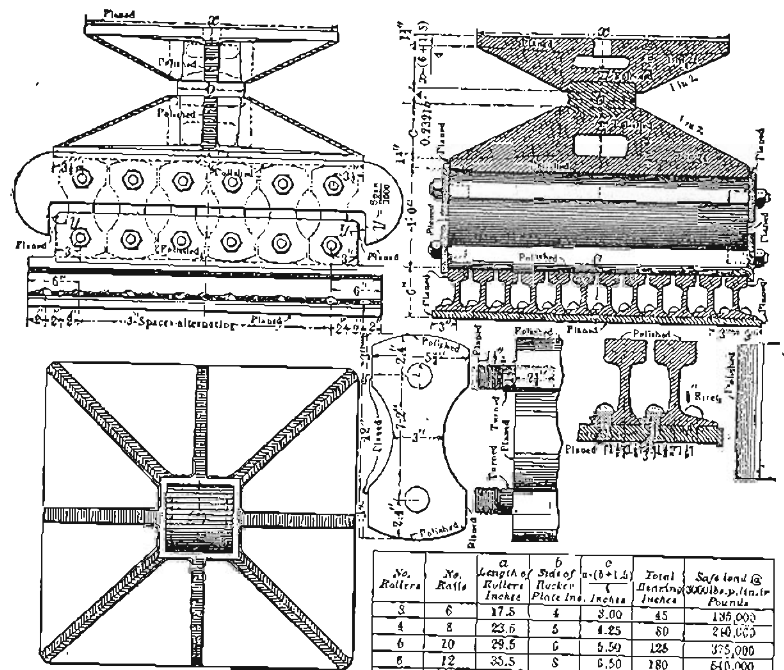
ART. 81. EXPANSION BEARINGS.

Pedestals, friction rollers, and bed plates, similar to those described in Art. 44, are used also for truss bridges. Two examples of expansion bearings containing cylindrical rollers are given on Plate III and in Fig. 111.

Complete detail drawings of pedestals, nests of cylindrical rollers, and rail plates, together with the castings for the fixed bearings, may be found on the insets of the Engineering News for Jan. 5 and Feb. 2, 1899. The nests contain 7 rollers each, their diameters being $4\frac{1}{2}$ and $4\frac{7}{8}$ inches respectively.

Fig. 109 shows the details of a standard expansion bearing, designed by GEORGE S. MORISON, which contains some valuable improvements over those employed in Europe. The steel rollers are 12 inches in diameter and spaced 6 inches between centers. The sides are parallel near the top and bottom, and hollowed out along the middle to facilitate cleaning with a brush. Contact between the parallel sides of the rollers prevents them from tipping over, but an additional provision against it is afforded

by means of the side plates, which engage stud bolts screwed into the ends of the rollers. The clearance between the hook of the upper plate and the square ends of the lower plate allows a linear movement of y = span /3000 in both directions from the mean position. The rollers rest on a rail plate consisting of



No. Rollers	No. Rails	a Length of Rollers Inches	b Side of Rocker Plate Ins.	c $\frac{a-(b+1.5)}{4}$ Inches	Total Bearing Inches	Safe load @ 3000 lbs. p. lin. in. Pounds
3	6	17.5	4	3.00	45	135,000
4	8	23.5	5	4.25	80	240,000
5	10	29.5	6	5.50	125	375,000
6	12	35.5	8	6.50	180	540,000
7	14	41.5	9	7.75	245	735,000
8	16	47.5	10	9.00	320	960,000
9	18	53.5	11	10.25	405	1215,000
10	20	59.5	12	11.50	500	1500,000
11	22	65.5	13	12.75	605	1815,000
12	24	71.5	14	14.00	720	2160,000

Fig. 109.

T-rails riveted to a plate, with their tops afterward planed and polished. In large bridges the rail plate is bolted to the cast base, which is directly supported by the pier masonry. As the dust accumulates it is readily removed by passing a long-handled brush between the rails.

On top of the rollers rests a steel casting with its lower surface polished, and this in turn supports another casting by means of a block of polished steel called a rocker plate. The rocker plate fits into a socket in each casting, the surfaces of contact being segments of horizontal circular cylinders, whose axes are respectively parallel and perpendicular to the direction of the rollers. The radius of curvature in each case equals the length of one side of the square rocker plate. The upper casting sustains the pedestal, which in turn supports the end pin of the truss, and the connecting bolts pass also through the flanges of the stiff lower chord. The object of the rocker plate is to allow the bridge to adjust itself when erected, so that the bearing on the rollers, and hence also that on the bed plate, may be uniform. This eliminates the unequal distribution of load, which would otherwise be caused by imperfect workmanship in the construction of the truss and its supports. To secure the transverse stiffness of the lower ends of the end posts, they are preferably connected by an end floor beam which is riveted to them after the bridge is swung. The side plates project above and below the rollers respectively, thus acting as guides to prevent any lateral movement of rollers or casting.

In the vertical line of dimensions in Fig. 109 the next to the highest one should read $\frac{x-(b+1.5)}{4}$, while the value 0.2321 b does not belong to c, but to the dimension directly above c. The safe load given in the table is that recommended by the designer, no addition being made to the live load for impact. The allowance for impact is included in the unit stress adopted. See Transactions of American Society of Mechanical Engineers, vol. 15, page 153, 1894, and vol. 16, page 724, 1895, for an account of the evolution of this bearing and some illustrations of its application. The description and detail drawings of a modified form of this bearing, in which a pin casting takes the place of the usual bolster and of the top casting and rocker plate, thus materially reducing the height required, may also be found in Engineering Record, vol. 32, page 93, July 6, 1895.

In order to avoid the danger of the rollers getting out of place under the frequent jars to which the lighter bridges are subject, another improvement has been added by fitting steel plates into grooves cut in the ends of the middle roller, the plates projecting beyond the surface of the roller and forming teeth to engage spaces cut into the rail plate below and the bearing plate above. The details of this device may be seen on Plate II.

A side elevation of the fixed and expansion bearings of the Davenport, Rock Island and Northwestern Railroad bridge at Rock Island, Ill., may be seen in the inset of Engineering News, Jan. 11, 1900. This is a different type of bearing from the standard described above. The I-beams extend under both bearings over the full width of the pier.

The 6-inch segmental rollers of the International bridge at Buffalo are illustrated in Engineering Record, vol. 43, page 567, June 15, 1901. They are 3 inches wide, but have cylindrical spaces 6 inches long cored out of their sides and separated by $\frac{3}{4}$-inch vertical webs. The rollers are 3 feet 5 inches long.

Fig. 110 gives the details of the 12-inch rockers with parallel sides used in the truss shown on Plate V. The center roller has a spur or gear tooth at each end on both top and bottom, and these enter slots in the roller bed plate and the shoe respectively, thus retaining the rollers in their proper position. Grooves in the centers of the rollers engage longitudinal center strips to prevent the trusses from shifting sideways. The figure also indicates the construction of the pedestal and bed plate at the fixed end of the span.

In the Delaware river bridge seven segmental cast-steel rollers 18 inches in diameter are used to take care of a truss reaction of 1200 tons. The rollers are of the same type as those mentioned in the preceding paragraph, and are $8\frac{1}{2}$ inches wide and 8 feet $2\frac{1}{2}$ inches long. The gear teeth on the middle roller are 7 inches long. A view of the fixed and expansion bearings on one pier is shown in Engineering Record, vol. 40, page 596, Nov. 25, 1899.

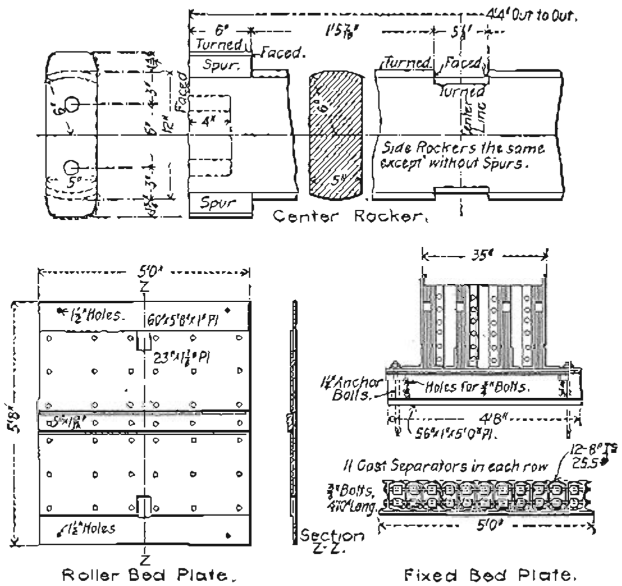
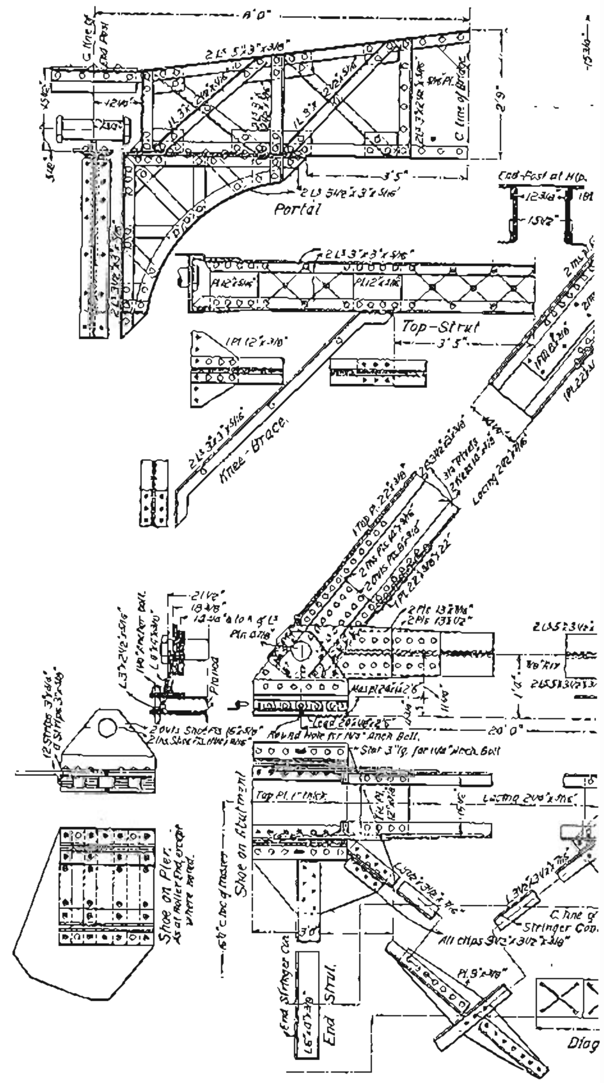


Fig. 110.

ART. 82. RAILROAD PIN BRIDGES—REFERENCES.

Variations in the composition of members throughout the span, the relations between the forms and dimensions of connecting members, and many of the smaller details related to the connections at the joints can best be studied by consulting the general drawings of different trusses. The following references are given to enable the student to become familiar with recent practice in these respects, no reference being included whose date precedes 1895, and only a few that are earlier than 1898.

Bridge No. 14, I



DETAILS OF A SINGLE-TRACK TI

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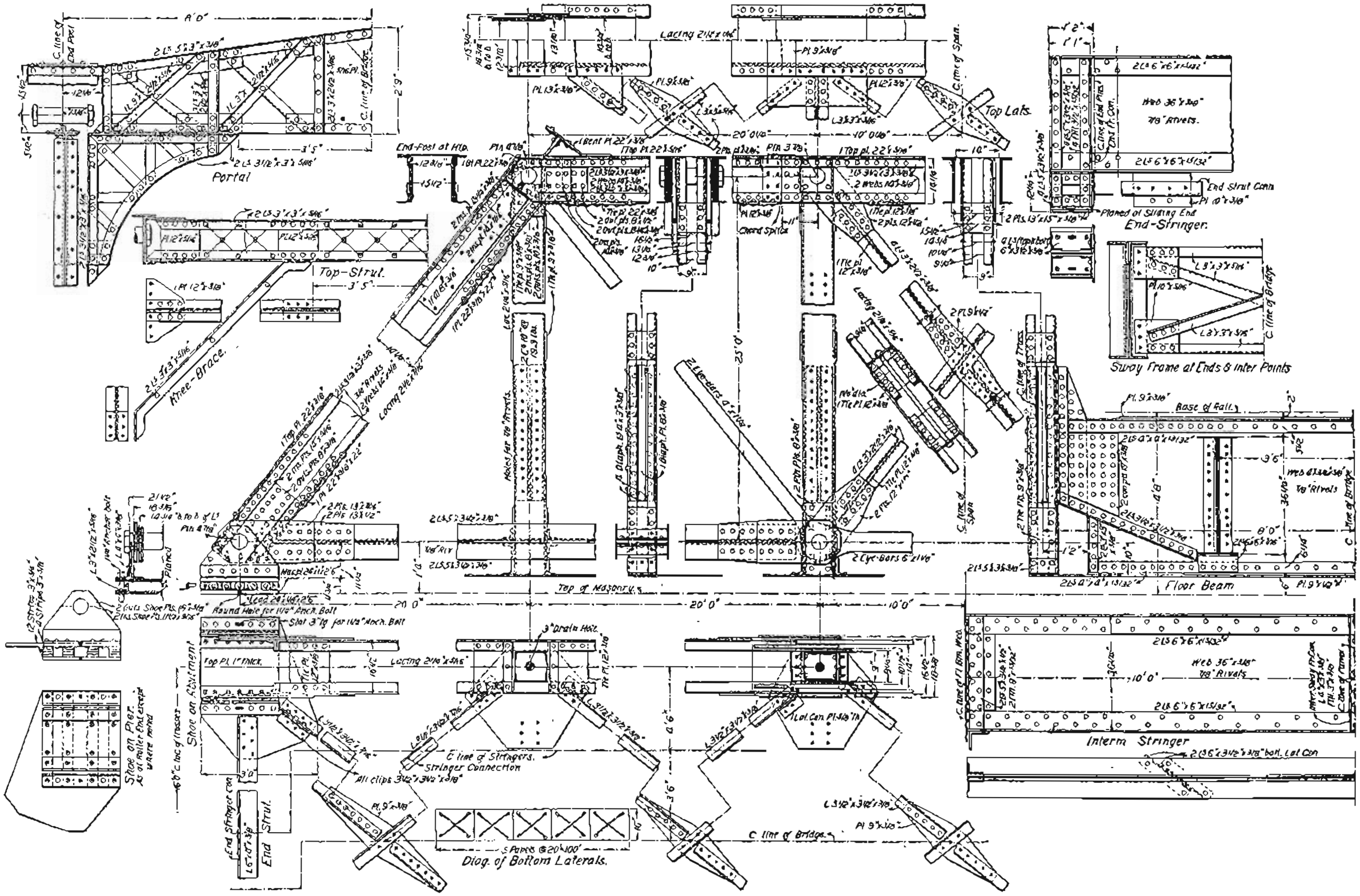
See references t

Bridge No. 14, Midland Division, Baltimore and Ohio Railroad.



DETAILS OF A SINGLE-TRACK THROUGH BRIDGE OVER DEER CREEK, EAST OF MT. STERLING, OHIO.

SPAN, 100 FEET. ERECTED IN DECEMBER, 1894.

See references to separate details in Arts. 71, 73, 74, 75, 76, 79, 80, and 81.

Recent small bridges on the Baltimore and Ohio Railroad. Railroad Gazette, vol. 27, page 34, Jan. 18, 1895.

General plan of Bridge No. 14, Midland Division. (See Plate III.) Single-track through Pratt-truss bridge, span 100 feet. The article contains extracts from the specifications which indicate the character of the structures.

Bridge Work on the Baltimore and Ohio Railroad. Engineering Record, vol. 41, page 271, March 24, 1900.

A description of the characteristic features of short-span bridges designed by the bridge department for extensive recent improvements where old bridges were replaced by heavier and stiffer structures. Several views, but no drawings.

Standard Plans for 120-foot Pony-truss Bridges, Northern Pacific Railway. Engineering News, vol. 41, page 14, Jan. 5, 1899. Standard Plans for 130-foot Through Truss Bridges, Northern Pacific Railway. Engineering News, vol. 41, page 69, Feb. 2, 1899.

Complete detail drawings. These standards have been superseded by those referred to below, but they will furnish the student a good opportunity for comparative study. The 120-foot truss is replaced in the new standards by a riveted truss.

A Trunk Line Deck Bridge. Engineering Record, vol. 33, page 58, Dec. 28, 1895.

Span, 127′ 11⅝″. Partial plans of a single-track deck bridge across Big Pipe Creek on the New York, Lake Erie and Western Railroad.

Short-span Railroad Bridges. Engineering Record, vol. 40, page 717, Dec. 30, 1899.

Extracts from specifications giving unit stresses; description of a few special details of spans of 125 and 150 feet of the Oregon Railroad and Navigation Co.; illustrations of floor-beam connection to the hanger at the first panel point, and of templets for reaming the connections of the floor system.

The Terminal Improvements of the Chesapeake and Ohio Railway, at Richmond, Va. Engineering News, vol. 44, page 379, Nov. 29, 1900.

Span, 133′ 7½″. The drawings show the details of one of the deck spans of the river viaduct.

Northern Pacific Standard Bridge Plans. By RALPH MODJESKI. Journal of Western Society of Engineers, vol. 6, page 51, Feb., 1901.

The plates accompanying this paper include the general plans of a 160-foot deck span and of a 200-foot through span, both being single-track bridges. (See Plate IV.)

Special Bridge and Viaduct Construction in Western Pennsylvania. Engineering Record, vol. 41, page 465, May 19, 1900.

Span, 177′ 3″. Description and illustration of some of the principal details of the Union Railroad bridge which crosses the Pittsburg, Virginia and Charleston Railroad at Port Perry, Pa. (See illustrations of details in Figs. 77, 103, 104, and 111.)

The Lehigh Valley Railroad Bridge at Easton, Pa. Engineering Record, vol. 41, page 124, Feb. 10, 1900.

Description and drawings showing the details of the outside trusses of two of the spans in which the track is on an eight-degree curve. One is a through span 187 feet long, and the other a deck span 215′ $1\frac{1}{2}$″ long. There is also a general view of the bridge and its approach. The bridge is noted for the special features of construction required by its difficulties of alignment, grade, etc.

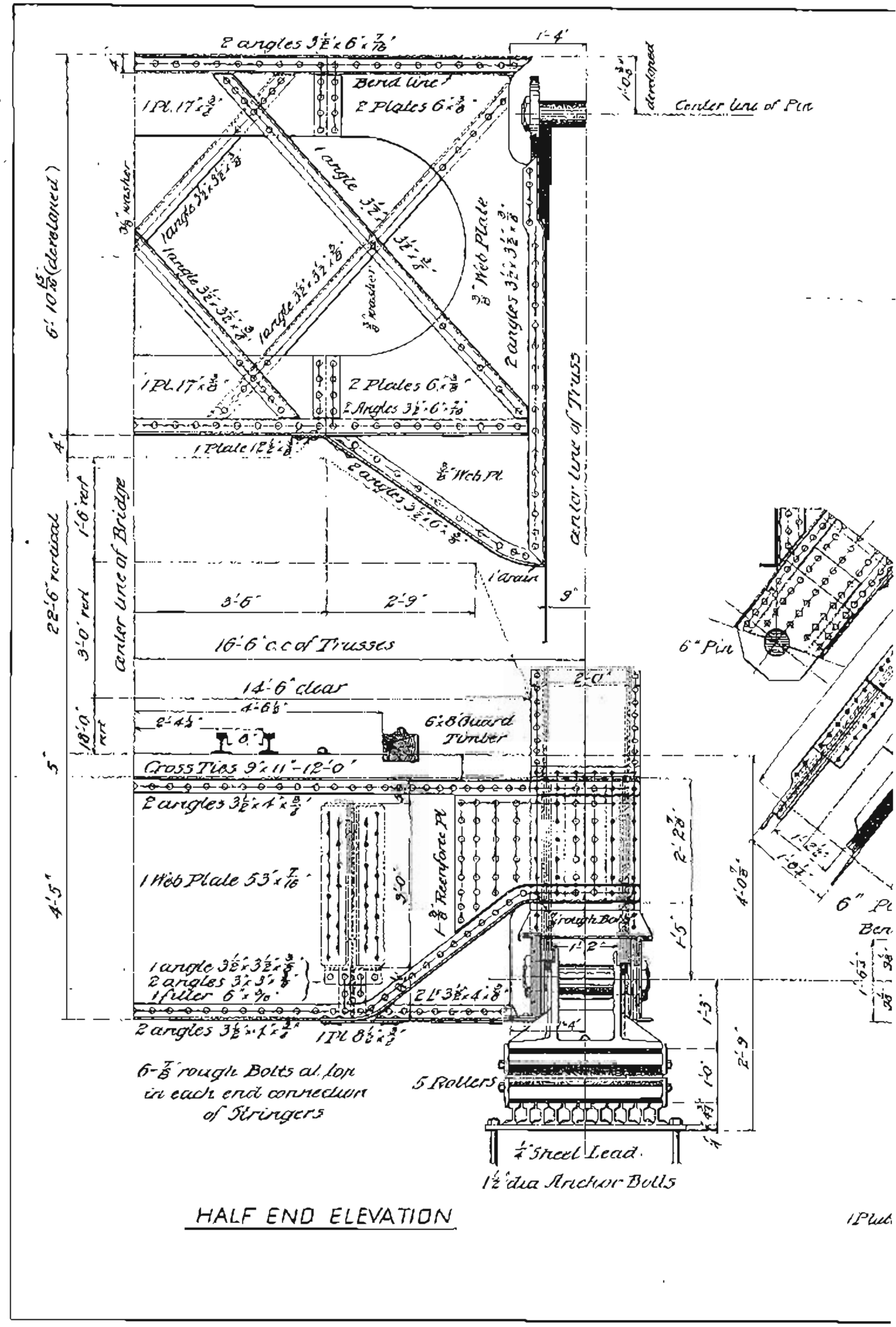
The Newport and Cincinnati Bridge. Engineering Record, vol. 37, page 448, April 23, 1898.

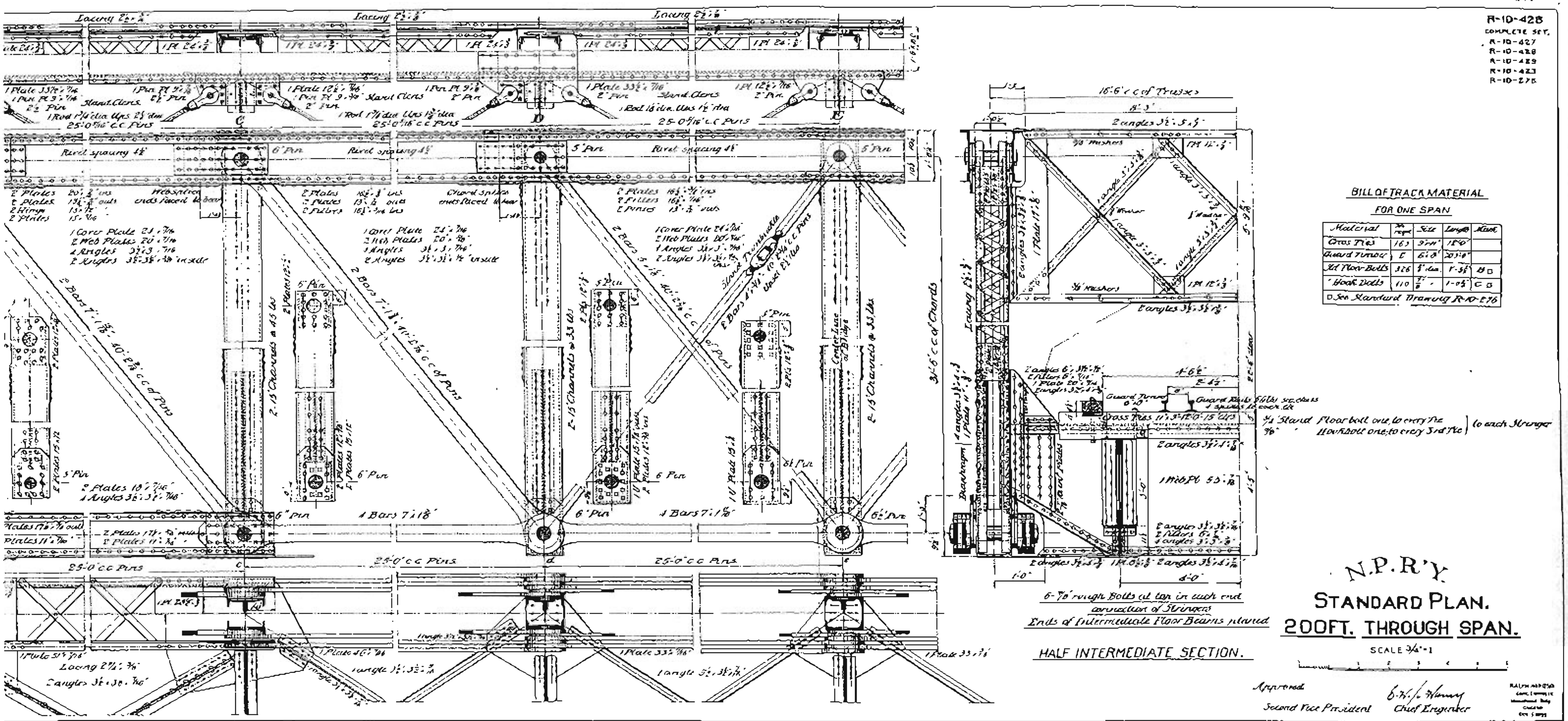
Span, 198′ 6″. Partial plans showing the details of one of the short Pratt truss through spans of this four-truss bridge. The upper chord is curved. The bridge accommodates a single-track railroad, two street railroad tracks, a roadway, and a sidewalk.

The New United States Rock Island Bridge. Engineering Record, vol. 37, page 384, April 2, 1898.

Span, 216′ $6\frac{3}{4}$″. Description and detail drawing of one of the ten-panel Baltimore trusses of a double-deck through bridge. The double-track railroad is on the upper deck. The portal and intermediate sway bracing of the 258-foot span are also shown on small-scale sections. (See also Engineering News, vol. 36, page 406, Dec. 17, 1896.)

Bridge 69, New York Division, Pennsylvania Railroad. Engineering Record, vol. 39, page 371, March 25, 1899.




R-10-428
COMPLETE SET.
R-10-427
R-10-428
R-10-429
R-10-423
R-10-276
16'-6" c.c. of Trusses
HALF INTERMEDIATE SECTION.
BILL OF TRACK MATERIAL
FOR ONE SPAN
Material | No. req'd | Size | Length | Mark
Cross Ties | 163 | 9"x11" | 18'-0"
Guard Timber | 2 | 6"x8" | 203'-0"
Std. Floor Bolts | 326 | 3/4" dia. | 1'-3 1/4" | B □
" Hook Bolts | 110 | 7/8" " | 1'-0 1/2" | C □
□ See Standard Drawing R-10-276
N.P.R'Y.
STANDARD PLAN.
200FT. THROUGH SPAN.
SCALE 3/4" = 1
Approved
Second Vice President
Chief Engineer

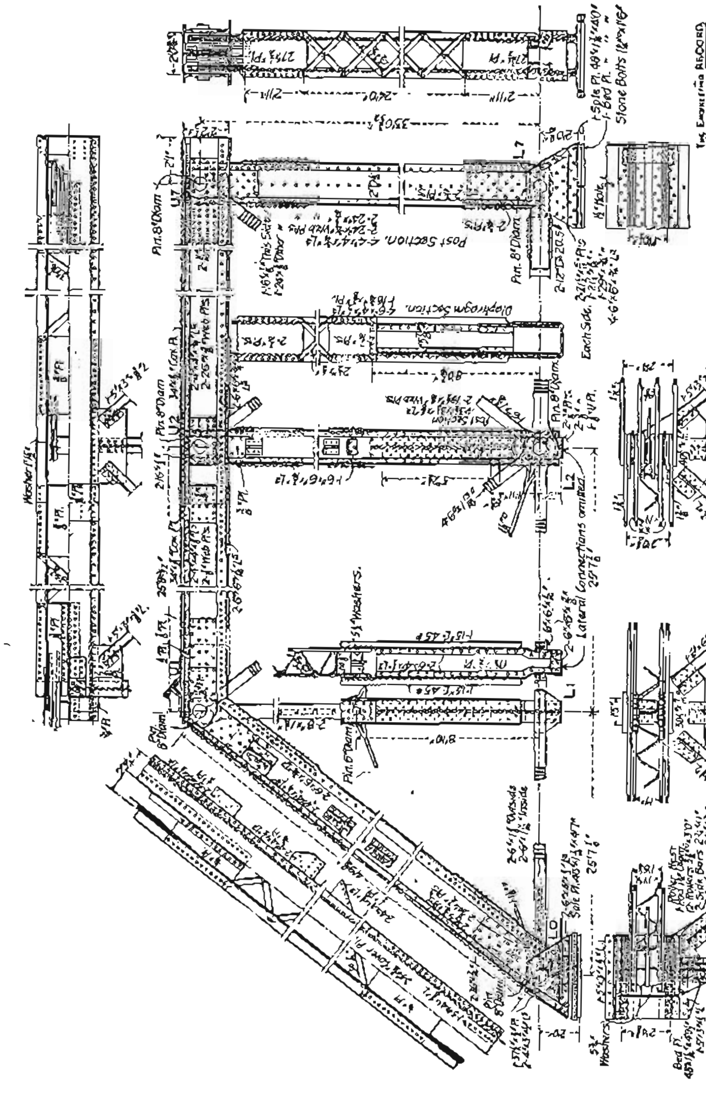


Fig. 111. Union Railroad Double-track Through Bridge over the Pittsburg, Virginia and Charleston Railroad at Port Perry, Pa. Span, 177¾ feet. Built in 1897. See references to various details in Arts. 73, 74, 77, 79, 81, and 96.

Span, 235′ 7″. Description and partial detail drawings of one of the spans of the double-track deck bridge over the Schuylkill river near Girard Avenue, Philadelphia. The illustrations are reproduced in Plate V and in Fig. 110. Many of the details are referred to in the preceding articles of this chapter. The structure was designed for heavy traffic under comparatively high speeds. (See Proceedings of the Engineers' Club of Philadelphia, vol. 14, page 302, Jan., 1898, for an account by JOSEPH T. RICHARDS of the operation of moving aside the old Whipple truss bridge and putting this new bridge in its place in two minutes and twenty-eight seconds, on Oct. 17, 1897.)

The International Bridge, Buffalo. Engineering Record, vol. 43, page 566, June 15, 1901.

Description of the characteristic details of this single-track through bridge with some of their dimensions for the trusses whose span is 244′ 7″. The only details shown in the drawings are splices in the upper and lower chords, the connection of the floor beam with the stiff lower chord, and one of the segmental rollers.

The Victoria Jubilee Bridge at Montreal; Grand Trunk Railway. Engineering News, vol. 38, page 130, Aug. 26, 1897.

The small-scale elevation, section, and plan give the stresses in the members of the Pratt truss whose span is 253′ 11½″, and the composition of the truss members and of the lateral and sway bracing. The cantilever ends of the floor beams of this double-track through bridge support a roadway and sidewalk on each side.

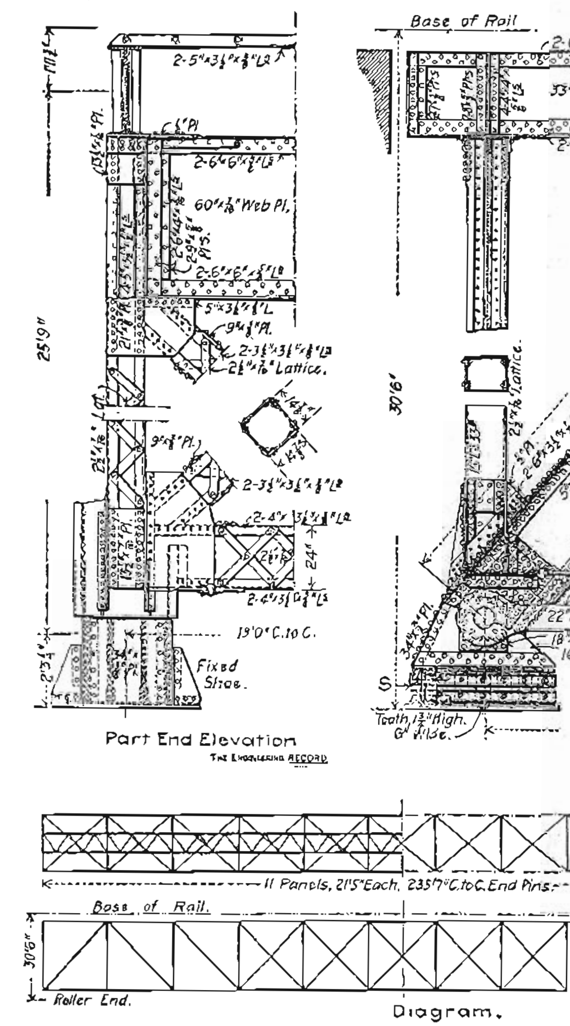
The Reconstruction of the Glasgow Bridge on the Chicago and Alton Railway. By W. D. TAYLOR. Engineering Record, vol. 43, page 241, March 16, 1901.

Description and detached drawings of several characteristic details of the through Pratt truss with curved upper chord whose span is 339¼′, and of the deck Pratt truss whose span is 139′ 5⅛″. The deck truss is supported at the top chord. The bridge is a single-track structure. Views of the completed bridge and of the falsework are given.

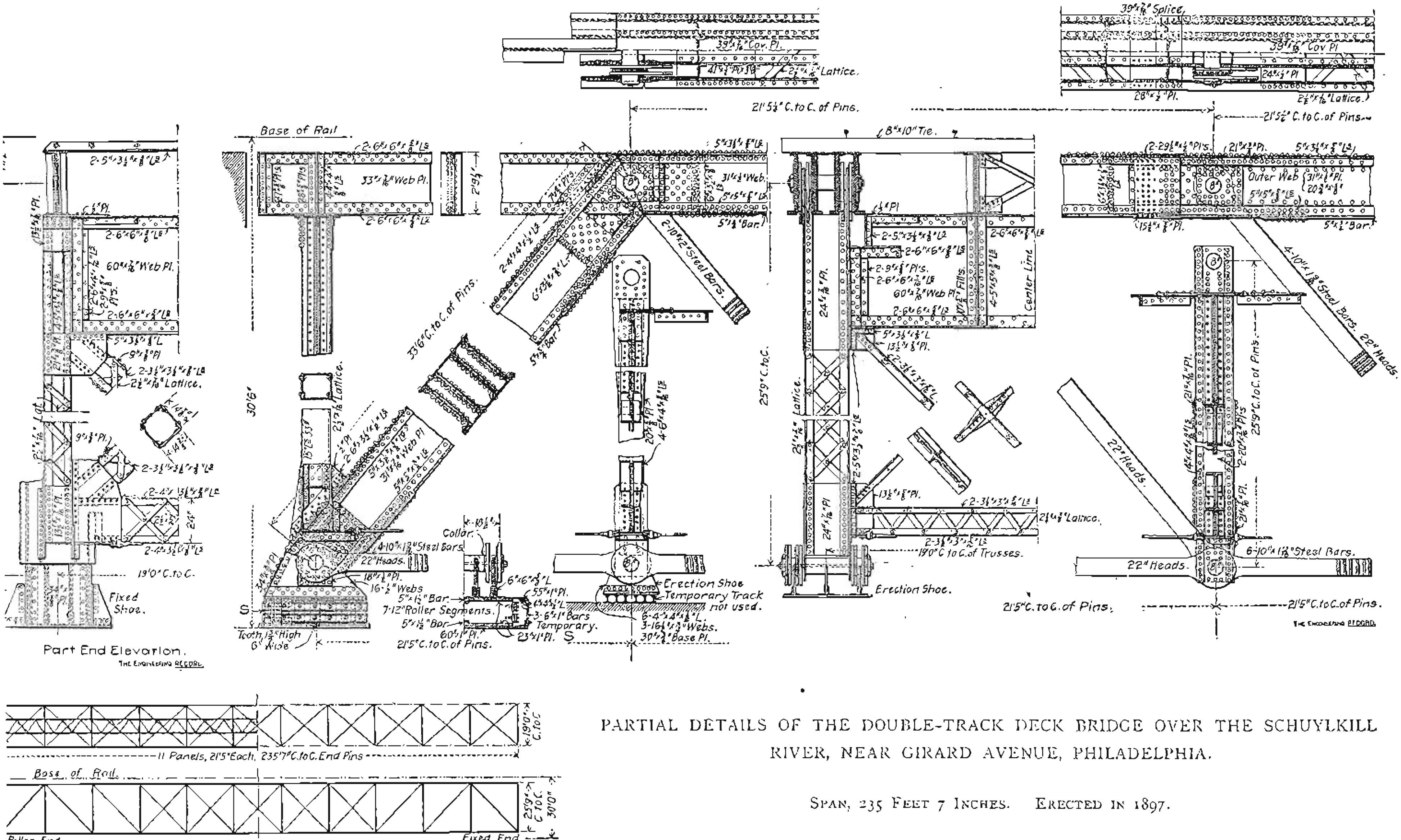
The Victoria Bridge at Montreal. Engineering Record, vol. 38, page 488, Nov. 5, 1898.

Span, 347′ 11½″. The composition of the truss members of the lateral and sway bracing and of the floor are marked on the small-scale drawings of a part of the Pettit truss. Similar data are also given for a part of the smaller Pratt truss spans. (See one of the preceding references.)

B



Bridge 69, New York Division, Pennsylvania Railroad.



PARTIAL DETAILS OF THE DOUBLE-TRACK DECK BRIDGE OVER THE SCHUYLKILL RIVER, NEAR GIRARD AVENUE, PHILADELPHIA.

SPAN, 235 FEET 7 INCHES. ERECTED IN 1897.

See references to separate details in Arts. 71, 73, 74, 77, 78, 80, and 81.

The Davenport and Rock Island Bridge over the Mississippi River. Engineering News, vol. 43, page 26, Jan. 11, 1900.

Span, 361 feet. The general character of the single-track through Pettit truss bridge is shown by a very small scale drawing, on which are marked the composition of most of the tension members. The article relates principally to the swing span.

Special Bridge and Viaduct Construction in Western Pennsylvania. Engineering Record, vol. 41, page 516, June 12, 1900.

Span, 396′ 8″. Double-track through Pettit truss bridge of the Union Railroad over the Monongahela river at Port Perry, Pa. Built in 1897. Description and partial detail drawings of the heavy truss, showing its most important features. Provision was made for adding a third truss later. One of the tracks is a hot-metal route with special fireproof protection.

The Rankin Bridge. Engineering Record, vol. 44, page 465, Nov. 16, 1901.

Span, 495′ $8\frac{3}{8}$″. Double-track through Pettit truss bridge of the Union Railroad over the Monongahela river at Rankin, Pa. Built in 1900. Description and partial illustration of details. One of the tracks is a hot-metal route. Both of these bridges of the Union Railroad were designed for the heaviest live load, consisting of two $192\frac{1}{2}$-ton locomotives followed by a uniform load of 5000 pounds per foot per track. The total weight of the steel work alone in one of the long spans of the Rankin bridge is about 2800 tons. (See Fig. 108.)

Superstructure of the Delaware River Bridge at Bridesburg, Philadelphia, for the Pennsylvania and New Jersey Railroad Company. By PAUL L. WÖLFEL. Proceedings of Engineers' Club of Philadelphia, vol. 14, page 154, 1897.

Span, 533 feet. Description of several special features in the design of this double-track through bridge with subdivided panels and curved upper chord. Built in 1896.

The Delaware River Bridge at Bridesburg. Engineering Record, vol. 40, page 594, Nov. 25, 1899.

Description of the principal details. Two of the fine views relate to the fixed spans. Some of this description was reprinted from Mr. WÖLFEL'S paper.

Die Brücke der Pennsylvania-Eisenbahn über den Delaware bei Philadelphia. Von F. C. KUNZ. Allgemeine Bauzeitung, Wien, Heft 1, 1901.

Description of the design, construction, and erection, illustrated by numerous views and a number of large plates showing many of the details of the fixed and swing spans. Analyses of their weights are included. (See Figs. 10 and 107.)

CHAPTER IX.

DESIGN OF A PIN TRUSS BRIDGE.

ART. 83. SPECIFICATIONS.

Let it be required to design a single-track through railroad bridge whose trusses are of the Pratt type and whose span is 175 feet between centers of end pins. The cross-ties, foot planks, and guard timbers are to be of long-leaf Southern yellow pine, the truss pins of medium steel, and the rest of the structure of soft rolled steel.

The trusses are to be spaced 17 feet center to center and the clear opening is not to be less than that shown in Fig. 112.

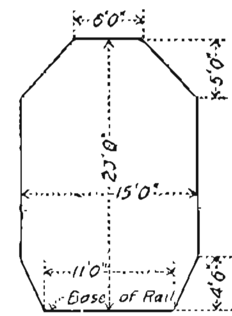


Fig. 112.

LOADS.

The live load is to be class Q of WADDELL'S compromise standard system (Art. 32), but the equivalent live loads given on the diagrams in WADDELL'S specifications are to be used instead of the actual wheel concentrations.

The effect of impact and vibration shall be added to the maximum stresses resulting from the above live load, and is to be determined by the following formula:

$$I = S\left(\frac{300}{L + 300}\right),$$

in which I is the impact, S the computed maximum live-load stress, and L the length of the loaded distance in feet which produces the maximum stress in the member.

To provide for wind stresses due to the pressure of the wind on the truss and train, as well as for lateral vibrations from high-speed trains, the wind load

CHAPTER X.

DESIGN AND DETAILING OF A HIGHWAY BRIDGE.[1]

ART. 104. DATA OF THE DESIGN.

It is proposed to design a highway bridge of 140 feet span which shall, in addition to highway traffic, carry that of trolley cars. The roadway is to be 18 feet in width from center to center of trusses. The trolley track is to be on one side of the roadway and its center 5 feet clear of the truss. On the opposite side of the roadway, on the outer side of the truss, will be a foot-walk 5 feet wide, supported on cantilever brackets attached to the posts. Judging from the rapid increase in trolley wheel loads during the past few years, it is advisable to construct all trolley bridges of such strength that they shall be sufficient to carry the traffic of heavy interurban cars. This bridge will accordingly be designed and detailed in accordance with Class B of COOPER's General Specifications for Steel Highway and Electric Railway Bridges and Viaducts (edition of 1901), with two exceptions as follows:

First, omit § 3 and § 63; second, insert these clauses in § 48: 'When the wind-load stress is taken into account, together with live-load stress in any truss member, two-thirds of it shall be considered as live-load equivalent and is to be added to the live-load stress in computing the total stress.' 'When a post is fixed at its ends the flexural stress caused by the wind shall be computed by considering the ends fixed, but in computing the total stress due to combined loads it shall be considered hinged at both ends.

On account of the uniformity of chord stresses and the small web stresses in the Bowstring truss, this form will be used.

[1] By F. O. DUFOUR, C.E., Instructor in Civil Engineering in Lehigh University.

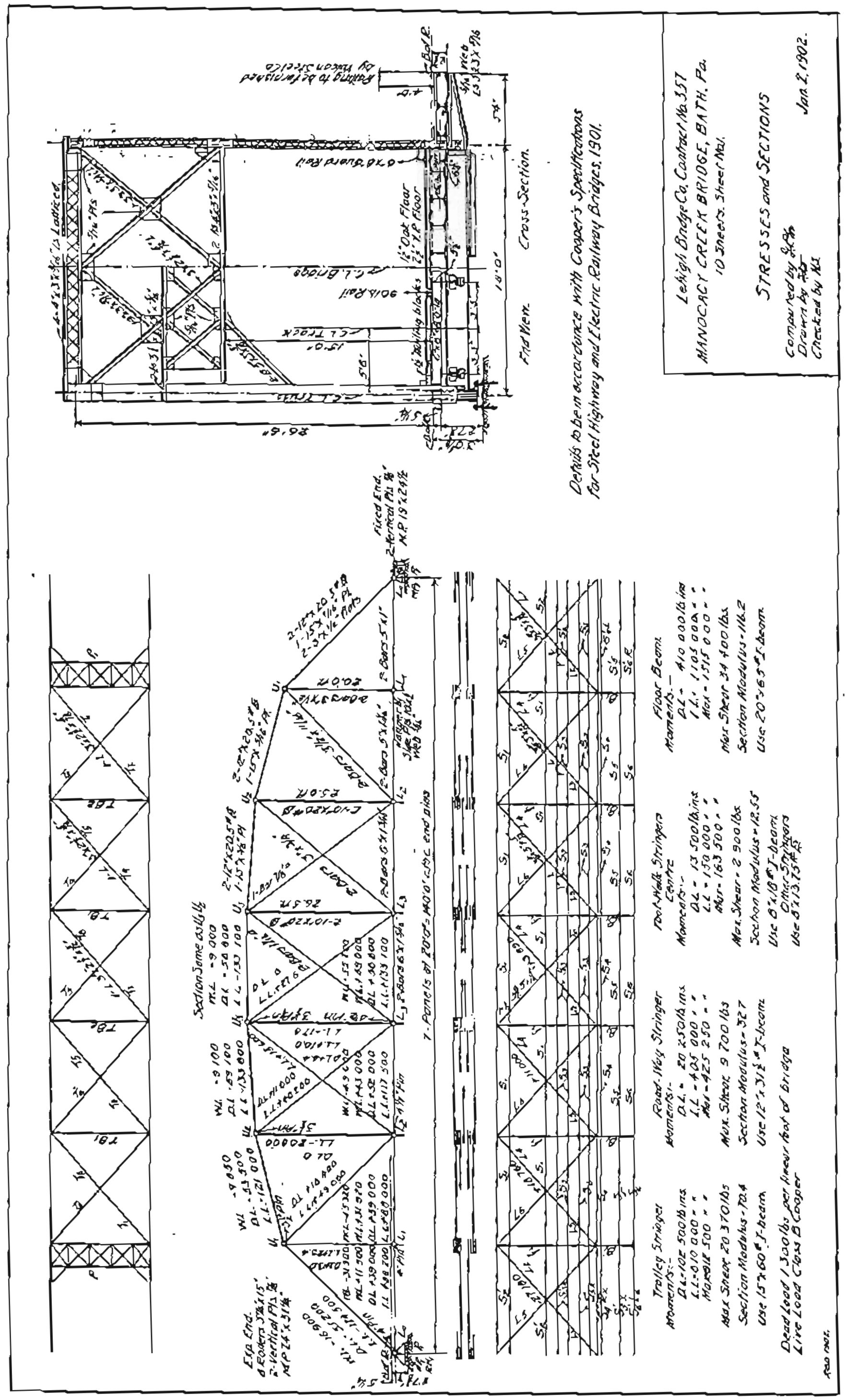


Fig. 137.

This truss has its upper chord pins approximately on the arc of a parabola which passes through the end pins and the middle top panel points. The general dimensions of the truss are as given on the stress sheet (Fig. 137).

The dead load per linear foot is estimated as 1300 pounds. This estimate is based upon the weight of a similar bridge which has been erected, and it is more likely to be correct than any value derived from empirical formulas. The live load for the truss is taken from table A (§ 38, Cooper). The panel length will be 20 feet, thus making 7 panels per truss. The dead panel load is $(1300/2)20 = 13\,000$ pounds. Taking the trolley track load as covering 8 feet of roadway, the roadway load as covering the remaining 10 feet, and the foot-walk as loaded with the roadway load, the live panel load for each truss will be 29 400 pounds. The dead, live, and wind-load stresses are now computed by the methods of Part I, and recorded on a stress sheet (Fig. 137).

A careful consideration of the entire set of specifications should now be made, as much misunderstanding in the design will thus be avoided. The spacing of stringers and the general arrangement of the floor system should be decided upon and placed on the stress sheet.

ART. 105. STRINGERS.

TROLLEY STRINGERS. — The dead load on these stringers is the weight of the 4-inch floor plus the weight of the ties and the rails. The rail is assumed to weigh 30 pounds per linear foot. The oak ties, 6×8 inches and 8 feet long, weigh $48/12\,(8 \times 4\frac{1}{2}) = 144$ pounds each, and if they are spaced 14 inches from center to center, there are 17.14 ties to a panel, or $(17.14 \times 144)/(2 \times 20) = 62$ pounds per linear foot. Considering the stringer nearest to the truss, and estimating the width of a post as 12 inches, the floor plank will weigh $4\frac{1}{2} \times 4 \times 5 = 90$ pounds per linear foot. The total dead load per linear foot is hence $30 + 62 + 90 = 182$ pounds, and the maximum bending moment due to this, since the length of the stringer is 20 feet, is

$$\tfrac{1}{8}(182 \times 20 \times 20 \times 12) = 109\,200 \text{ pound-inches.}$$

In order to obtain the maximum live-load moment, the two loads of 12 000 pounds each should be so placed that the center of the stringers is midway between the center of gravity of the loads and one load (Part I, Art. 91).

From this rule it is found that this load comes at $7\frac{1}{2}$ feet from the end. The maximum bending moment occurs under this load, and its value is,

$$\tfrac{1}{20} \times 12\,000\,(2\tfrac{1}{2} + 12\tfrac{1}{2})\,7\tfrac{1}{2} \times 12 = 810\,000 \text{ pound-inches.}$$

The dead-load moment under this wheel is, $(182 \times 20 \times 7.5 - 7.5^2 \times 182)\,12/2 = 102\,500$ pound-inches. The total moment is the sum of the dead and live load moments, or 912 500 pound-inches. Then $912\,500/13\,000 = 70.4$ inches^3 is the section modulus necessary. A 15-inch 60-pound I-beam has a section modulus of 81.2 inches^3, and is hence stronger than required, but it will be used. The maximum shear is readily seen to be $12\,000(1 + \frac{1}{2}) + (182 + 55)\,20/2 = 20\,370$ pounds. The moment due to the weight of the beam itself is, $(55 \times 20 \times 20 \times 12)/8 = 33\,000$ pound-inches, but as this is less than $\frac{1}{10}\,(912\,500)$ it need not be considered (§ 55, Cooper).

ROADWAY STRINGERS. — The dead load for these stringers consists of only the weight of floor covering, which is $2 \times 4 \times 4\frac{1}{2} = 36$ pounds per linear foot. The bending moment due to this weight at $7\frac{1}{2}$ feet from the end is $\frac{1}{2}\,(36 \times 20 \times 90 - 36 \times 7\frac{1}{2}) = 20\,250$ pound-inches. This is the dead-load moment under the wheel that causes the maximum live-load moment. Here the loads are placed in the same position as for the trolley stringer. The concentration being 6000 pounds,

$\frac{6000}{20}(2.5 + 12.5) \times 7.5 \times 12 = 405\,000$ pound-inches is the maximum live-load moment. The total bending moment is $20\,250 + 405\,000 = 425\,300$ pound-inches, which requires a section modulus of $425\,300/13\,000 = 32.7$ inches³. A 12-inch $31\frac{1}{2}$-pound I-beam satisfies this condition and will be used. The maximum shear is $6000\ (1 + \frac{1}{2}) + (36 + 31.5)\ 20/2 = 9700$ pounds. The moment in the beam due to its own weight is $\frac{1}{8}(31.5 \times 20 \times 20 \times 12) = 18\,900$ pound-inches, but as this is less than $\frac{1}{10}(425\,300)$, it need not be considered.

FOOT-WALK STRINGERS. — The bending moment due to the uniform load of 100 pounds per square foot is, for the center stringer,

$$\tfrac{1}{8}(2.5 \times 100 \times 20 \times 20 \times 12) = 150\,000 \text{ pound-inches.}$$

The moment due to dead load, which consists of the weight of flooring only, is $\frac{1}{8}(2.5 \times 2 \times 4\frac{1}{2} \times 20 \times 20 \times 12) = 13\,500$ pound-inches. The maximum moment is the sum of these, or 163 500 pound-inches, and then $163\,500/13\,000 = 12.55$ inches³ is the section modulus required. An 8-inch 18-pound I-beam will be used, as it satisfies the conditions (§ 57, Cooper). The moment due to the weight of the beam itself is $\frac{1}{8}(18 \times 20 \times 20 \times 12) = 10\,800$ pound-inches, and as this is less than $\frac{1}{10}(163\,500)$, it can be neglected.

The outer and inner stringers of the foot-walk will consist of 8-inch channels. The inner one must resist one-half the above moment, while the outer one will, in addition to this, sustain a railing estimated to weigh 60 pounds per linear foot. The railing will be connected to the stringers at the ends and at the one-third points. Both inner and outer stringer will be made the same on account of economy of construction. The moment

due to the railing is $\frac{1}{3}(20 \times 60 \times 3 \times 20 \times 12) = 32\,000$ pound-inches. The total moment which the outer stringer must stand is $32\,000 + 163\,500/2 = 113\,750$ pound-inches, and this requires a section modulus of $113\,750/13\,000 = 8.75$ inches³. An 8-inch 13.75-pound channel must be used (§ 57, Cooper). The moment due to the weight of the stringer may be neglected. The maximum shear for the inner stringer is, $\frac{1}{2}(13.75 \times 20 \times 2.5 \times 100 \times 20/2 + 2.5 \times 2 \times 4\frac{1}{2} \times 20/2) = 1499$ pounds. In the same manner the maximum shear for the middle stringer is found to be 2905 pounds, and for the outer stringer, railing included, 2100 pounds.

The masonry plates for the end stringers can now be computed (§ 126 and § 130, Cooper). They are as follows: For the trolley stringer, area $= 20\,370/250 = 81.8$ square inches, and length $= 81.8/6 = 13.63$ inches, since the width of flange of trolley stringer is 6 inches; in like manner for the roadway stringer the plate must be 5×7.8 inches, for the center foot-walk stringer 2.35×3.6 inches, and for the channel stringers 4×3 inches. These dimensions should not be taken as final, as in all probability they will be changed in order to secure good details. The thickness of all plates should be one-half inch.

ART. 106. FLOOR BEAMS.

The live load must be so placed on the stringers that the sum of the reactions for two adjacent stringers shall be a maximum. The trolley live load will be assumed to be the only live load acting. The dead-load concentration under the first trolley stringer is as follows:

Due to flooring, $5 \times 4 \times 4\frac{1}{2} \times 20$	= 1800 pounds.
Due to trolley stringer, 60×20	= 1200 pounds.
Due to trolley rail, 30×20	= 600 pounds.
Due to cross-ties, 62×20	= 1240 pounds.
Total for trolley stringer	= 4840 pounds.

The concentration under second trolley stringer is slightly less, but will be considered the same. The dead-load concentration under a roadway stringer is:

Due to flooring, $2 \times 4 \times 4\frac{1}{2} \times 20 =$ 720 pounds.
Due to stringer, $31.5 \times 20 =$ 630 pounds.
Total for roadway stringer = 1350 pounds.

All roadway concentrations are considered equal. The left reaction, due to these concentrations, is:

$$\tfrac{1}{17}\,[4840\,(8.75+15.25)+1350\,(0.75+2.75+4.75+6.75)]$$

= 8020 pounds, and the dead-load bending moment for the floor beam is $8020 \times 8.25 \times 12 - 4840 \times 6.5 \times 12 = 416\,460$ pound-inches. The maximum live-load concentration is 18 000 pounds, and the left reaction due to this is $(2 \times 18\,000)/17 =$ 25 400 pounds. The width of posts being assumed as 12 inches, the length of the floor beam is 17 feet. The live-load moment is $(25\,400 \times 8.25 - 18\,000 \times 6.5)\,12 = 1\,105\,000$ pound-inches. The maximum bending moment now is $1\,105\,000 + 416\,460 = 1\,521\,460$ pound-inches, which requires a section modulus of $1\,521\,460/13\,000 = 117$ inches^3. A 20-inch 65-pound I-beam will be used. The moment due to the weight of the beam itself is $\frac{1}{8}(65 \times 20 \times 20 \times 12) = 39\,000$ pound-inches, which can be neglected, as it is less than $1\,521\,460/10$ (§ 55, Cooper). The maximum shear is $(25\,400 + 7900 + 17 \times 65) = 34\,400$ pounds. The reaction at the other end will be less, but the same connections will be used at each end, as at some future date the track may be changed.

FOOT-WALK BRACKET. — This is the floor beam for the sidewalk. The concentrations are (see Art. 105 under foot-walk stringers) as follows:

At inner stringer, $2 \times 1500 = 3000$ pounds.
At center stringer, $2 \times 2900 = 5800$ pounds.
At outer stringer, $2 \times 2100 = 4200$ pounds.

The bending moment at the post is $(5800 \times 2\frac{1}{2} + 4200 \times 5)12 = 426\,000$ pound-inches. By reference to the stress sheet it will be seen that the top of bracket is $19\frac{1}{8}$ inches from the bottom of the post. Assuming that the center of gravity of the angle to be used is $\frac{1}{2}$ inch from its back, and such assumption is near enough for this computation, the effective depth of the bracket at the post is $18\frac{1}{2}$ inches, or say 18 inches. Then the stress in the top flange is $426\,000/18 = 23\,700$ pounds, and this demands a net area of $23\,700/13\,000 = 1.82$ square inches. One angle will then require 0.91 square inch net area. It is specified that two $\frac{7}{8}$-inch rivet holes shall be deducted from the angle section. A $3 \times 3 \times \frac{5}{16}$ inch angle will be used, this having a net area of $1.78 - (0.875 + 0.125)0.625 = 1.15$ square inches, which is greatly in excess of the required area, but the smallest obtainable satisfying the conditions. The stress in the bottom flange is slightly less, but the same angles will be used as for the top flange. A solid $\frac{5}{16}$-inch web will be used throughout. The connection to the post will be made by the detail shown on the stress sheet.

ART. 107. TENSION MEMBERS.

The eye-bars should not be greater in width than four-thirds the diameter of the pin to which they are attached (§ 104, Cooper), and in general the thickness of a bar should not be less than one-sixth its width. Pins safe in bending are liable to be deficient in bearing. For this reason it is advisable to design the bars so that they will not be deficient in bearing on pins of minimum diameter. A relation satisfying this condition will now be deduced.

Let t be the thickness of the bar, W its width, P the total stress it is required to sustain, D the diameter of the smallest pin, and S the allowable unit bearing stress. Then $SDt = P$. But $D = \frac{3}{4}W$, and $W = 6t$. Substituting these values and reducing, $9\,St = 2\,P$. Here $S = 18\,000$ pounds per square inch, and hence $t = 0.00352\,P^{\frac{1}{2}}$ is the minimum allowable thickness of a bar. Guided by this, and knowing that bars under six inches should be ordered in variations of one-half inch and bars over six inches should be ordered in variations of one inch, it is now easy to find the sizes of the eye-bars whose maximum stress is known. The maximum stress in the web members is readily found by adding one-half of the dead-load stress to the live-load stress (§ 45, Cooper). This sum divided by the number of bars which are to carry it, gives the load P above. The minimum thickness is then computed. Next the area and the maximum width are found, and lastly the final size. For web members, the widths should generally decrease from the ends toward the middle of the truss. For lower chord members the widths should generally increase from the ends toward the middle. According to § 52 of the Specifications the wind stresses must be considered in designing these sections.

The maximum stress in L_0L_1 is,

$$88\,200 + 39\,000 + 31\,900 \times 0.8 = 152\,720 \text{ pounds.}$$

The maximum stress in L_1L_2 is,

$$88\,200 + 39\,000 + 45\,320 \times 0.8 = 163\,450 \text{ pounds.}$$

The maximum stress in L_2L_3 is,

$$117\,500 + 52\,000 + 43\,000 = 212\,500 \text{ pounds.}$$

The maximum stress in L_3L_3 is,

$$133\,100 + 58\,000 + 49\,000 = 240\,900 \text{ pounds.}$$

In computing these stresses eight-tenths of the negative wind load is added only when it is greater than the positive wind load (§ 55, Cooper).

The number of bars to be taken for any member is a matter of choice in some respects. An even number should, of course, always be taken, except when only one is needed. They should be so chosen and packed that the flexure of the pin is a minimum. A large number decreases this flexure while the reverse increases it. It costs almost the same to forge a large eye-bar as it does a small one, while the manufacture of large pins is much more costly than the manufacture of small ones.

The problem resolves itself into this form, namely, that the cost of eye-bars and pins shall be a minimum. The shop practice of different plants modifies the results obtained as to the number of bars which satisfy these conditions, and therefore no hard and fast rule can be given. The stress in the heaviest eye-bar due to the weight of the bar itself is readily computed to be 2100 pounds per square inch, which need not be considered. A table can now be formed as follows:

MEMBER.	P POUNDS.	NUMBER OF BARS.	t INCHES.	UNIT STRESS. POUNDS/SQ.IN.	AREA REQUIRED IN SQ. INCHES.	MAXIMUM WIDTH INCHES.	BARS USED. INCHES.
L_0L_1	76 360	2 eye	0.88*	15 625	4.90	5.57	5×1
L_1L_2	81 730	2 eye	0.91*	15 625	5.23	5.75	$5 \times 1\frac{1}{16}$
L_2L_3	106 250	2 eye	1.03*	15 625	6.80	6.60	$6 \times 1\frac{3}{16}$
L_3L_3	120 450	2 eye	1.09*	15 625	7.70	7.06	$6 \times 1\frac{5}{16}$
U_1L_2	29 100	2 eye	0.60	12 500	2.33	3.88	$3\frac{1}{2} \times \frac{11}{16}$
U_2L_3	27 850	2 eye	0.59	12 500	2.23	3.80	$3 \times \frac{3}{4}$
U_3L_3	13 800	2 loop		12 500	1.10	1.05	$1\frac{1}{16} \times 1\frac{1}{16}$
U_3L_2	10 100	1 loop		12 500	0.81	1.00	$\frac{7}{8} \times \frac{7}{8}$
U_1L_1	17 950	2 eye	0.47	12 500	1.44	3.06	$3 \times \frac{1}{2}$

* $s = \frac{5}{4} \times 18\,000 = 22\,500$, and hence $t = 0.00316\,P^{\frac{1}{2}}$.

The counter U_3L_2 and one set of the main diagonals U_3L_3 are to be adjustable, turnbuckles being used, and the bars are to be upset according to § 105 of the Specifications.

ART. 108. VERTICAL POSTS.

POST U_2L_2.—From § 49 of the Specifications it is seen that the radius of gyration cannot be less than $(25 \times 12)/100 = 3$ inches, and the thickness of metal must be $\frac{5}{16}$ inch thick or more (§ 75, Cooper). The first condition precludes the possibility of using four angles latticed in pairs back to back, as their area would be greatly in excess of that required for a radius of gyration of 3 inches. A channel section is the most economical, although even in this case the area will be greatly in excess of that required. Remembering that the radius of gyration cannot be less than 3 inches and that the web cannot be less than $\frac{5}{16}$ inch thick, and noting that if the 3-inch legs of the floor beam connecting angles are used on the channel the width cannot be less than 8 inches, it is found that a 10-inch 20-pound channel is the section which can be used, and this satisfies all conditions.

The maximum stress in this post is live load only, as the dead-load stress is equal to zero, and its value is 20 800 pounds. The unit load allowable is, from the Specifications, $P = 10\,000 - 45\frac{l}{r}$. If the length of the post in feet be used, this formula becomes $P = 10\,000 - 540\frac{L}{r}$, where L is the length in feet. Here $L = 25$ feet, $r = 3.66$ inches, and the area $A = 2 \times 5.88 = 11.76$ square inches. Then $P = 10\,000 - 540\frac{25}{3.66} = 6300$ pounds per square inch, and $20\,800/6300 = 3.29$ square inches, are required. Thus it will be seen that, in order to meet the conditions of § 48 and § 75, the area is greatly in excess of that required by the given formula. A 6-inch 8-pound channel could be used if it were required to satisfy the conditions for unit load only.

POST U_3L_3.—For this post the total load is $17\,800 + (10\,000 + 4400/2)0.8 = 27\,560$ pounds (§ 51, Cooper). The channels used above will be tried. Here $L = 26.5$ feet, the unit load is

$P = 10\,000 - 540\frac{26.5}{3.66} = 6100$ pounds per square inch, and $27\,560/6100 = 4.52$ square inches are required. These channels will be used. An 8-inch 11.25-pound channel could be used, if the unit load formula alone were to be satisfied, and still would be in excess.

WIND ON VERTICAL POSTS.—The channels for these posts should be placed a certain distance from back to back, which will not only insure safety against the compression of the vertical load, but also that due to the effect of bending at the point where the transverse wind bracing is connected. This latter bending is caused by the wind. Fifteen feet of head room being required (§ 10, Cooper), and, considering the lower end of the post to be at the center line of the pin, the bending moment is $(150 \times 20 \times 15 \times 12)/4 = 135\,000$ pound-inches. This regards the post as fixed in direction at the ends and the upper lateral bracing as not in action, the point of contra-flexure being taken as half-way between the end of the post and the wind-bracing connection. It is the case of a member under compression and flexure. The following is an approximate method of determining the relation between the properties of the section and the loads which it may safely carry.

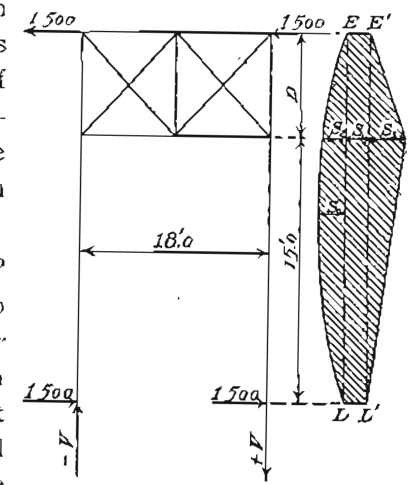


Fig. 138.

Let l be length of post, P the total load or stress it is to carry, A its sectional area, r the least radius of gyration of that section, P/A the direct unit stress S_1 due to P, and S the allowable compressive unit stress for a short block. Let S_0 be the unit stress at the middle of the post, due to the bending that may be caused by P. Then $S = S_0 + P/A$ is the unit stress on the concave side at the middle. Now the straight line column formula is

$$S_1 = P/A = S - k\frac{l}{r},$$

in which $S = 10\,000$, $k = 45$ for the post in hand (§ 48, Cooper). Here the term kl/r is the value of S_0. Let S_2 be the flexural unit stress at any distance x from the end of the post. If the curve of bending moments or stresses be regarded as a parabola, this unit stress is

$$S_2 = \frac{4(l-x)x}{l^2}S_0 = \frac{4k(l-x)x}{lr}.$$

Now let M be the bending moment at the point x due to a wind force acting normal to the axis of the post. The unit stress caused by this force on the outer fiber of the post is

$$S_3 = \frac{Mc}{I},$$

in which I is the moment of inertia of the section and c is one-half its width normal to the channel webs. In order that the post shall be safe at the point considered it is necessary that $S_1 + S_2 + S_3$ shall not exceed the allowable unit stress S, or

$$\frac{P}{A} + \frac{4k(l-x)x}{lr} + \frac{Mc}{I} = S$$

is the equation to be satisfied by the properties of the section. Substituting for I its value Ar^2, and solving for c,

$$c = \frac{Ar^2}{M}\left(S - \frac{P}{A} - \frac{4k(l-x)x}{lr}\right), \qquad (1)$$

from which c can be computed for any assumed value of r.

Let b be one-half the distance from back to back of the channels, and g the distance from the back of a channel to an axis through the center of gravity of the channel section and parallel to its web. Let I_1 be the moment of inertia and r_1 be the radius of gyration of one channel section referred to this axis. Now $I = 2\,I_1 + Ah^2$, in which h is the distance from the axis of the post to the axis for which the moment of inertia is I_1. When the flanges of the channels are turned out, $h = b + g$, and when they are turned in, $h = b - g$. Substituting for h its value and for I_1 its value $\frac{1}{2}Ar_1^2$, there results

$$b = \pm g + \sqrt{r^2 - r_1^2},$$

in which the plus sign is to be used for flanges turned in, and the minus sign for flanges turned out. When flanges are turned in, as is to be the case with the posts of this bridge, the distance c is the same as b, and hence

$$r^2 = r_1^2 + (c-g)^2 \qquad (2)$$

is another relation between c and r.

The method of spacing the channels of the intermediate post is hence as follows: Assume a value of c, and compute r from (2); then insert this value of r in (1) and compute c. If the assumed and computed values of c do not agree, a new value is to be assumed and the computation repeated. Usually only two trials are necessary to bring the computed and assumed values within $2\frac{1}{2}$ per cent. The value of c to be assumed in the first computation should be a little greater than that value which gives the post section equal radii of gyration with respect to the two rectangular axes. If x and l are taken in feet, then $k = 12 \times 45 = 540$. For the case in hand (see Sheet 1, Fig. 137), $l = 26.5$ feet, $x = 11.5$ feet, $A = 11.76$ square inches, $P/A = 27\,560/11.76$ pounds per square inch, and $M = 135\,000 \times 2/3 = 90\,000$ pound-inches when reduced to live-load equivalent (see additional clause, Art. 104). Assuming $c = 3.5$ inches, and taking $g = 0.61$ inch and $r_1 = 0.70$ inch from the handbooks for the given channel, there is found from (2) $r = 2.98$ inches; inserting this in (1), there is then found $c = 3.39$ inches. Repeating the computation by assuming c as 3.55 inches, (2) gives $r = 3.02$ inches; inserting this in (1) gives 3.55 as assumed. The post channels will hence be placed $7\frac{1}{4}$ inches from back to back. If the channels should have their flanges turned out, the value of c would be 5.55 inches and the distance from back to back would be $5\frac{5}{8}$ inches.

The above is not an exact analysis, as the post is considered hinged at both ends when the unit load is computed and when the action of combined loads is considered, but in computing the bending moment due to wind it is considered fixed at one end. The assumption that the lateral bracing is not in action is also incorrect. The above, however, is a good approximate guide in aiding the designer to stiffen the post under wind.

ART. 109. HANGER AT THE HIP VERTICAL.

As the vertical L_1U_1 is a tension member, some provision must be made at L_1 to connect L_1U_1 to the floor beam. This is done by means of a hanger, which will consist of two side plates connected by a web and four angles. The function of this web is to transmit one-half the floor-beam reaction to the outer side, and on the foot-walk side to carry one-half the foot-walk reaction to the inner side. It must therefore resist a shear of $\frac{1}{2}(3000 + 5800 + 4200 + 34\,400) = 23\,700$ pounds. Its net thickness, assuming the depth the same as that of the floor beam, will be $23\,700/(20 \times 0.8 \times 10\,000) = 0.15$ inch, but by § 59 of the Specifications it must be at least $\frac{5}{16}$ inch thick, and this value will therefore be adopted.

The side plates should have a net area at the pin of 23 700/8000 or 2.96 square inches. As the pin is 4 inches in diameter, the thickness of the plates, the width being taken as 10 inches, is $2.96/(10 - 4) = 0.493$ inch, and they will be made $\frac{1}{2}$ inch thick. The thickness of the angles must not be less than $\frac{5}{16}$ inch, the exact size to be determined by the conditions of detailing, but not less than $3 \times 2\frac{1}{2}$ inches.

ART. 110. END POSTS.

The section will consist of two channels, flanges turned out, a cover plate, and two flats. The flats will be riveted to the lower flanges, thus increasing the section, and at the same time keeping the neutral axis near the center of the web. The posts are spaced $7\frac{1}{4}$ inches from back to back, and, as all diagonals are packed inside the post, the top chord channels cannot be less than $7\frac{1}{4}$ inches from back to back. The distance from back to back will be made such that the channels of the posts will clear the rivet heads of the pin plates of the chord sections. A $\frac{7}{8}$-inch rivet head is $\frac{5}{8}$ inch high, and consequently $(7\frac{1}{4} + 2 \times \frac{5}{8})$ or $8\frac{1}{2}$ inches is the distance required. It is well to add at least $\frac{1}{4}$ inch for clearance, thus making the final distance from back to back of channels equal to $8\frac{3}{4}$ inches. The width of a 12-inch $20\frac{1}{2}$-pound channel flange is 3 inches, and using this, the cover plate width must be at least $14\frac{3}{4}$ inches. It will be taken $\frac{7}{16}$ inch thick and 15 inches wide, as plates over 6 inches should be ordered in variations of one inch in width. The flats will be taken as $3 \times \frac{1}{2}$ inches.

This section will now be investigated in order to determine if it fulfills the conditions, and does not give an excess or deficiency of area. The center of gravity is computed by taking moments about an axis through the center of top plate and parallel to its width. It is best to arrange the principal quantities in tabular form, A representing the area of any part in square inches, and l its lever arm in inches with respect to the axis mentioned above.

PIECE.	A	l	Al
2 channels	12.06	6.218	75.20
1 plate	6.56	0.0	0.0
2 flats	3.00	12.218	36.66
Sums	21.62		111.86

Then the distance from center of cover plate to the center of gravity of section is

$$g = \Sigma Al / \Sigma A = 5.21 \text{ inches},$$

and the eccentricity of the section, or distance from center of channel web to neutral axis is $e = (12/2 + \frac{7}{16}/2) - 5.21 = 1.008$ inches. The moment of inertia of the section is now computed, neglecting the moments of inertia of the plates about their own axes parallel to their width; thus

PIECE.	A	I'	h	Ah^2
2 channels	12.06	0	5.210	150
1 plate	6.56	256	1.008	12
2 flats	3.00	0	7.008	146
Sums	21.62	256		308

whence $I = \Sigma(I' + Ah^2) = 564$ inches^4, and the radius of gyration of the section is

$$r = (564/21.62)^{\frac{1}{2}} = 5.1 \text{ inches}.$$

Lastly, by the column formula of the Specifications,

$$P = 10\,000 - 540\frac{28.28}{5.1} = 7060 \text{ pounds per square inch},$$

which is the safe unit load for the assumed section. As the stress on the post is 152 100 pounds, the area required is 152 100/7060 = 21.54 square inches, which is practically the same as that assumed, and accordingly the latter may be used.

By § 97 of the Specifications, it will be seen that the thickness of the channel webs is less than $\frac{5}{16}$ inch. The discrepancy being small, however, their use will be allowed, as much economy in quantity of material results.

The end post is also subjected to bending, due to wind, at a point where the knee brace of the portal strut joins it. If the portal strut be taken as six feet deep and the knee brace as joining the post six feet lower, the bending moment is

$$12\,[(28.3 - 12) \times 3 \times 20 \times 150]/2 = 146\,700 \text{ pound-inches},$$

when the post is free at lower end, or 146 700/2 = 73 350 pound-inches if the post is fixed at the lower end, the point of contra-flexure being considered as half-way between the end and the knee-brace connection.

The end post may be regarded as fixed if the moment of the wind acting with a lever arm equal to the distance from center to center of end pins is less than the moment caused by one-half the stress in the end post acting with a lever arm equal to the distance from center to center of the bearings of the pin at the lower end. For this computation the length of the end post may be considered as 28.3 feet and the distance from center to center of bearings as slightly more than $8\frac{3}{4}$ inches, say $9\frac{1}{2}$ inches. The moment for the first case is $3000 \times 3 \times 28.3 \times 12 = 3\,060\,000$ pound-inches, and the moment for the second case is $\frac{1}{2}(9\frac{1}{2} \times 152\,700) = 725\,000$ pound-inches. As the first of these values is greater than the second, the post will be considered as having free ends and will be required to stand a bending moment of 146 700 pound-inches or $\frac{2}{3}\,146\,700 = 98\,000$ pound-inches when reduced to live-load equivalent (Art. 104, additional clause to § 48, Cooper).

The moment of inertia of the section with reference to an axis perpendicular to the cover plate is now computed and is found to be 547 inches^4, thus giving a radius of gyration of 5.02

inches, and an allowable unit load of 6960 pounds per square inch. Hence, 152 100/6960 = 21.8 square inches are required, but the assumed section will not be changed, as it is less than one percent in deficiency. Referred to the above axis the section can stand a live-load moment of

$$M = \frac{SI}{c} = \frac{10\,000 \times 547}{7.5} = 731\,000 \text{ pound-inches.}$$

Since the moment due to the wind, 98 000 pound-inches, is less than one-fourth of 731 000, it need not be considered (§ 52, Cooper).

ART. 111. TOP CHORD SECTIONS.

For the chord U_1U_2 a 5/16 × 15 inch cover plate and two 12-inch $20\frac{1}{2}$-pound channels will be used (§ 90, Cooper). Here, proceeding as in the case of the end post, $g = 4.44$ inches, $e = 1.72$ inches, $I = 384.5$ inches^4, $r = 4.79$ inches, and the total area is 16.75 square inches. Then (§ 48, Cooper),

$$P = 12\,000 - \frac{660 \times 20.6}{4.79} = 9140 \text{ pounds per square inch,}$$

and 147 750/9140 = 16.20 square inches is the area of section required. The moment of inertia referred to an axis through the middle of the section and perpendicular to the cover plate is 406.9 inches^4, and hence the assumed section is amply safe in that direction. The wind stresses are not considered in any of the top chord sections (§ 52, Cooper).

The same section will be used for both U_3U_2 and U_3U_3, and will be designed for the greatest stress, which is 163 350 pounds. A $\frac{3}{8}$-inch cover plate and two 12-inch 20-pound channels will be tried. Here, computing as before, $g = 4.22$, $e = 1.965$, $I = 404$ inches^4, $r = 4.77$ inches, and the total area is 17.69 square inches. The unit load allowable is

$$P = 12\,000 - \frac{660 \times 20}{4.77} = 9240 \text{ pounds per square inch,}$$

and 163 350/9240 = 17.7 square inches is the area required. The moment of inertia referred to an axis through the center of the section and perpendicular to the cover plate is 444.5 inches^4, which shows the section to be safe for that axis, and hence it will be used.

ART. 112. CENTER LINE OF PINS.

Pins are not placed at the centers of gravity of the sections, nor on the center line of the web of channels. They are placed at such a distance below the center of gravity that the direct stress acting along the neutral axis will produce a moment neutralizing the moment due to the weight of the member itself. Let this distance be denoted by p, let W be the total weight of the member in pounds, l the length in inches, and P the total stress in the member, which in this case is the sum of the dead and live load stresses. Then

$$Pp = \tfrac{1}{8}\,Wl, \text{ or } p = \tfrac{1}{8}\,Wl/P.$$

Let d be the distance of center line of pins above the center of web of channels. Then

$$d = e - p.$$

To determine the weight per linear foot of a member for this computation, the weight of material in the section is taken and 20 percent added for the weight of batten plates, lattice bars, rivet heads, and pin plates. For example, for the end post L_0U_1 the weight per linear foot is,

2 channels, 12 inch × $20\frac{1}{2}$ pounds =	41 pounds,
1 plate, 15 × $\frac{7}{16}$ inches =	22 pounds,
2 flats, 3 × $\frac{1}{2}$ inches =	10 pounds,

and the sum of these plus 20 percent is 100 pounds nearly. Here the component which causes bending is 100/1.414 or 71 pounds, and the total weight W is 71×28.3 pounds. The distance p is

$$p = \frac{71 \times 28.3 \times 28.3 \times 12}{8 \times 179\,700} = 0.475 \text{ inch},$$

and hence $d = 1.008 - 0.475 = 0.523$ inch is the correct distance of the center line of pins above the center line of web of channels. In like manner are found $p = 0.25$ inch, and $d = 1.47$ inches for U_1U_2, while for U_2U_3 and U_3U_3 there results $p = 0.23$ inch, and $d = 1.74$ inches. As loads increase or decrease, d increases or decreases. The center line of pins must also be the same distance from the center of the web throughout for constructive reasons. It is not advisable to use the highest or the lowest values of d, but an average value, say $1\frac{1}{4}$ inches, should be taken.

ART. 113. DESIGN OF PINS.

The pin at each joint should be designed to resist bending, bearing, and shear, and also to satisfy § 104 of the Specifications. As an example a pin will be designed for the point L_3 of the lower chord. Here the large eye-bars being the members carrying the largest stress, the greatest bending moment will occur when they take the maximum stress, which will be when the bridge is entirely loaded. The maximum stress will be taken as the live-load equivalent (§ 48, Cooper). For this loading the stress in L_3U_3 is zero; the stress in U_2L_3 is $+11\,000$ from dead and $+24\,900$ from live load; the stress in U_3L_3 is $+4400$ from dead and $+10\,000$ from live load; and the floor beam exerts a downward pull of 1300 pounds from dead and 29 400 pounds from live load. The wind load is taken as 48 000 pounds in each member, in order to balance the horizontal components.

A table should now be prepared giving the horizontal and vertical components of these stresses for the point L_3. It is to

	HORIZONTAL COMPONENTS, POUNDS.			VERTICAL COMPONENTS, POUNDS.		
	L_2L_3	L_3L_3	U_2L_3	U_3U_3	U_2L_3	Lower end of U_3L_3
Live	$-117\,500$	$+133\,100$	$-15\,550$	$+10\,000$	$+19\,400$	$-29\,400$
$\frac{1}{2}$ dead	$-26\,000$	$+29\,400$	$-3\,450$	$+2\,200$	$+4\,300$	$-6\,500$
$\frac{2}{3}$ wind	$+32\,000$	$+32\,000$	$-\ \ 000$	$+\ \ 000$	$+\ \ 000$	$-\ \ 000$
Sum	$-175\,500$	$+194\,500$	$-19\,000$	$+12\,200$	$+23\,700$	$-35\,900$

be noted that the sum of the horizontal components and the sum of the vertical components are each equal to zero. This serves as a check on the computations.

Taking the packing from the stress sheet (Fig. 137) and assuming the total thickness of the bearing surface of U_3L_3 to be $\frac{1}{2}$ inch, and cutting the flanges of the channels to within $1\frac{1}{4}$ inches of the backs, the horizontal bending moment at the center of each bar is computed by $M = M + V'x$ (Mechanics of Materials, Art. 47). Thus the horizontal bending moments are found as follows:

MEMBER.	STRESS.	V'	x	$V'x$	M
L_2L_3	$-87\,750$				
		$-87\,750$	1.313	$-115\,000$	
L_3L_3	$+97\,250$				$-115\,000$
		$+\ 9\,500$	2.345	$+\ 22\,000$	
U_2L_3	$-\ 9\,500$				$-\ 92\,800$
		$\pm\ \ 000$	——		

while the vertical bending moments are:

MEMBER.	STRESS.	V'	x	$V'x$	M
U_3L_3	$+11\,850$				
		$+1850$	1.438	$+17\,100$	
U_2L_3	$-11\,850$				$+17\,100$
		$\pm\ 000$	——	$\pm\ \ 000$	

The resultant bending moment under U_2L_3 is

$$(92\ 800^2 + 17\ 100^2)^{\frac{1}{2}} = 93\ 600 \text{ pound-inches.}$$

As this is less than 115 000, the bending moment under L_3L_3, the maximum is therefore 115 000 pound-inches, which occurs under the large eye-bar L_3L_3. The size of the pin can now be computed according to methods given in text-books on mechanics of materials, or by reference to tables in manufacturers' hand-books, and it will be found that a 4-inch pin is needed to resist the bending moment. By § 104 of the Specifications the pin is required to be $6 \times \frac{3}{4} = 4\frac{1}{2}$ inches in diameter. The maximum unit shear is $87\ 750/15.9 = 5500$ pounds per square inch. The unit stress per square inch for bearing for L_2L_3 is $87\ 750/(4.5 \times 1\frac{3}{16}) = 16\ 400$ pounds; in the same manner that for L_3L_3 is 16 350 pounds, and that for U_2L_3 is 8200 pounds, all of which show the pin to be safe. Hence the $4\frac{1}{2}$-inch pin will be used at L_3 and L_2.

Upon computing pins for other joints it is found that a $3\frac{3}{4}$-inch pin can be used at L_1, that a 4-inch pin is required at L_0, a $3\frac{1}{4}$-inch pin at U_1, a $3\frac{1}{2}$-inch pin at U_2, and a 3-inch pin at U_3. A $4\frac{1}{2}$-inch pin will be used at L_3 and L_2, a 4-inch pin at L_0 and L_1, and a $3\frac{1}{2}$-inch pin at U_1, U_2, and U_3.

As the center line of pins is $1\frac{1}{4}$ inches above the center line of the webs of the channels, and as the eye-bars should have a section through the center of pins of 40 percent excess over the body of the bar, the head of a 5-inch eye-bar is $5 + 4 + (0.4 \times 5) = 11$ inches wide. The radius of the head is therefore $5\frac{1}{2}$ inches, which shows that the head of the bar will strike the cover plate. If the cover plate is stopped off a few inches from the end, this obstruction will be cleared, and in doing so the strength of the member will in nowise be lessened, as at the ends the allowable unit load is the allowable unit stress in bearing, or 18 000 pounds per square inch, and the 15.06 square inches left in the

post is capable of carrying $18\ 000 \times 15.06 = 270\ 100$ pounds, the pin plates not being considered. The cover plate will be so arranged in detailing.

ART. 114. PEDESTALS AND ROLLER NESTS.

The vertical plates as seen on the stress sheet will go inside of the end post. The maximum reaction is equal to $3\frac{1}{2}$ times the dead panel load divided by 2, plus 3 times the live panel load. The half panel live load that comes at L_0 is transferred directly to the abutments by the end stringers, there being no floor beam at the end. The maximum reaction is, therefore,

$$\tfrac{1}{2}(3\tfrac{1}{2} \times 13\ 000) + 3 \times 29\ 400 = 115\ 950 \text{ pounds.}$$

The design of the pedestals for the fixed end will be made first. The bearing area required is $115\ 950/18\ 000 = 6.45$ square inches, and $\frac{1}{4}(6.45) = 1.612$ inches is the width of the bearing area on a 4-inch pin. Two vertical bearing plates each $\frac{7}{8}$ inch thick will be used. The inside connection angles will be $5 \times 3 \times \frac{1}{2}$ inches, the 5-inch leg vertical, and the outer ones will be $5 \times 3\frac{1}{2} \times \frac{1}{2}$ inch (§§ 130–132, Cooper). The masonry plates cannot be less than $8\frac{3}{4} + 2 \times 3\frac{1}{2} = 15\frac{3}{4}$ inches in width, say 16 inches. The bearing area required is $115\ 950/250 = 465$ square inches.

The length of the masonry plate must be $465/16 = 29$ inches. The bearing plate will be the same area and thickness. Both bearing and masonry plates will be ordered $\frac{13}{16}$ inch thick and finished on one side to $\frac{3}{4}$ inch. The pedestal will be anchored to the masonry by $1\frac{1}{4}$-inch anchor bolts securely fox-bolted in the masonry to a depth of 12 inches.

The design of the pedestal and roller nest for the free end is as follows: The vertical plates and connection angles will be the same as at the fixed end. The width cannot be less than $15\frac{3}{4}$ inches, as before, nor can it be greater than $21\frac{3}{4}$ inches, since the bearing plate should not extend, unsupported, beyond the edges of the connecting angles for a distance greater than 3 inches. If it extends further than 3 inches, the remainder cannot be considered in taking up the bearing. The allowable load for rollers per linear inch is 300 d (§ 27, Cooper). Here d is $2\frac{7}{8} + 0.4 \times 1 = 2\frac{7}{8} + \frac{3}{8} = 3\frac{1}{4}$ inches, which makes the load per linear inch equal to 975 pounds. Hence, 115 950/975 = 119 linear inches are required. If each roller be 15 inches long, eight will be needed. Allowing for a small guide bar 2 inches wide at the middle, the rollers will be 17 inches long. If a tie bar $\frac{1}{2}$ inch thick be used on each side and guide angles 3 × 3 inches, the masonry plate, allowing $\frac{1}{16}$ inch clearance between members, will be at least $(17 + 4 \times \frac{1}{16} + 2 \times \frac{1}{2} + 2 \times 3) = 24\frac{1}{4}$ inches wide, and it will be taken as 25 inches. If a $\frac{1}{8}$-inch space be allowed between each roller, and $\frac{3}{4}$-inch tie rods be used, and a variation of 150° in temperature be assumed, the length of the plate will be found to be $31\frac{1}{4}$ inches. The bearing and masonry plates should be ordered $\frac{13}{16}$ inches thick and finished on one face to $\frac{3}{4}$ inch. The dimensions of the bearing plate should be the same as the masonry plate, the extra width being required in order that room may be provided to allow slotted holes to be cut for the anchor bolts. In detailing, care should be exercised to extend the bearing plates properly and to make the under side the same distance below the center line of the pins as the bottom of floor beam, plus a $\frac{5}{16}$-inch connection plate, in order to allow for the connection of the angles of the lower lateral bracing.

ART. 115. LATERAL AND TRANSVERSE BRACING.

According to § 4 of the Specifications all laterals must be of shapes capable of resisting compression. Angles will be used, but for a tension member the section will be determined from the tensile stresses as computed.

LOWER LATERALS. — Here 18 000 pounds per square inch is the allowable unit stress, and dividing this into the computed stresses, it is found that sectional areas of 1.51, 1.04, 0.62, and 0.22 square inches are needed in the first, second, third, and fourth panels respectively. The angles must, however, have a net area of not less than $\frac{3}{4}$ of a square inch (§ 97, Cooper). A $3\frac{1}{2} \times 3 \times \frac{5}{16}$ inch angle gives a net area of 1.55 square inches, after allowing for one $\frac{7}{8}$-inch rivet taken out of the section. The $3\frac{1}{2}$-inch leg will be placed vertically downward. It may be here stated that the vertical leg of an angle should always be placed downward when possible, for the water will run off quicker, and dust and dirt do not accumulate and hasten deterioration as they do when the leg is upwards, while a trough carrying the rain and dirt into the connection at the ends of angle is also avoided.

UPPER LATERALS. — The computed stresses being small, in all cases requiring less than $\frac{3}{4}$ of a square inch, $3 \times 2\frac{1}{2} \times \frac{5}{16}$ inch angles will be used and connected to the top chord by $\frac{5}{16}$-inch plates and $\frac{3}{4}$-inch rivets.

INTERMEDIATE TRANSVERSE BRACING. — By § 121 of the Specifications transverse or sway bracing is required. The computed stress for its lower chord is (3000 × 26.5)/(2 × 11.5) = 3500 pounds. The radius of gyration cannot be less than (12 × 18)/120 = 1.8 inches. Two $4 \times 3 \times \frac{5}{16}$ inch angles, placed $\frac{3}{8}$ inch from back to back, give a radius of gyration of 1.9 inches and conform to the Specifications in regard to thickness. They will be used, although they give an excess of about one-half a square inch of area over that required. The top chord of the intermediate bracing will consist of four of these angles latticed, and the bottom chord will consist of two. The longer leg is placed outward in each case. These two chords will be connected by two panels of latticing consisting of $3 \times 3 \times \frac{5}{16}$ inch angles. These angles, designed to resist the vertical shear, require a much smaller section, but by § 97 of the Specifications they must be used.

ART. 116. PORTAL BRACING.

In order to give the required head room, the portal bracing can be 7 feet deep. It will, however, be taken as 6 feet, with a knee brace joining the post 6 feet farther down. The wind load at the hip is $150 \times 20 \times 3 = 9000$ pounds, and the vertical shear is $\frac{1}{18}[3000(26.5+25+20)1.414] = 16\,850$ pounds. The moment in the portal strut at the point where the knee brace joins it, is $9000 \times 72 + 4500 \times 267.6 - 16\,850 \times 66 = 738\,000$ pound-inches, and its stress, taking 6 feet as effective depth, is $738\,000/72 = 10\,200$ pounds. By § 48, the radius of gyration cannot be less than 1.8 inches and, by § 97, the thickness of the angles cannot be less than $\frac{5}{16}$ inch. For the bottom chord, the stress, where the knee brace joins it, is $(4500 \times 28.3 \times 12)/72 = 16\,600$ pounds. Two angles $3\frac{1}{2} \times 3 \times \frac{3}{4}$ inches, spaced $\frac{3}{4}$ inch from back to back, will be tried. Here $r = 1.90$ inches, and the net area $= 7.12$ square inches. The unit load is $13\,000 - 720\,l/r$, l being in feet. This gives 6150 pounds per square inch for the unit load, and dividing this into the total stress gives 2.7 square inches as the area required, which shows that the angles are too heavy. Two $3\frac{1}{2} \times 3 \times \frac{5}{16}$ inch angles have a radius of gyration of 1.8 inches. Here $P = 5800$ pounds per square inch and 2.86 square inches area required. These angles give a gross area of 3.86 square inches and a net area of 3.22 square inches, and they will be used. Both flanges will be made of the same section.

The knee brace of the portal strut will now be investigated. Considering that it is to be placed at an angle of 45 degrees, the length is $1.414 \times 6 = 8.5$ feet. The stress in it is $(4500 \times 22.3)/4.25 = 23\,700$ pounds, and two $5 \times 3 \times \frac{5}{16}$ inch angles give a least radius of gyration of 0.85 inch. Hence, $P = 5800$ pounds per square inch and the net area required is 4.10 square inches. As these angles give a gross area of 4.80 and a net area of 4.18 square inches, they will be used.

ART. 117. THE STRESS SHEET.

The work of the designer is now finished, and the results, with sketches representing the general arrangement of stringers, floor beams, eye-bar packing, and details, are handed to a draughtsman in his office, who makes the stress sheet (Fig. 137), often improperly called a strain sheet.

On this sheet the stresses in the members, together with their sections, are noted on outline diagrams of the truss and lateral systems drawn to a small scale. Another view, one-half of which shows an end elevation of the truss and the other half a section taken near the middle of the span, is drawn to a larger scale. On this view are shown as many of the details and arrangements of the floor system and truss members as possible. A diagram of the packing of the eye-bars is also given. The eye-bars of L_1U_1 are placed outside of the chord at the hip U_1 in order to reduce the bending stress on the pin. For the same reason the largest eye-bars at the middle of the span are packed nearest to the posts, and the diagonals inside of the posts.

The roadway is arranged according to §§ 17–21 of the Specifications. The sidewalk has been placed below the level of the roadway in order to lessen the tendency of persons to step from the sidewalk to the roadway while crossing. Bridge companies do not have the facilities for the manufacture of ornamental railings, but these are usually bought from firms doing that class of work. These firms furnish the bridge companies with sketches, showing the location, size, and number of holes for the bolts or rivets which connect the railing to the bridge, from which the details of the outer foot-walk stringers and the bracket may be correctly made.

The distance from base of rail to the masonry and from base of rail to the center line of pins, together with clearances and general dimensions of the truss and floor system, are also placed on the stress sheet.

This stress sheet may also be used as a marking diagram, using the following notation. The stringers are marked with the letters S_1, S_2, S_3, etc., and if one differs from the other it should be marked S_1' or S_1x. Should two be similar in all respects except that one is right-handed and the other left-handed, they are marked S_1R and S_1L. Top laterals should be marked T_1, T_2, etc., bottom laterals L_1, L_2, etc., transverse bracing TV_1, TV_2, etc., and plates P_1, P_2, etc., the same rules in regard to subscripts, primes, and rights and lefts applying here also. It is best to give the moments and shears for the floor system in order to save their re-computation by the draughtsman who details the bridge. A good clear stress sheet is indispensable to enable the details to be made correctly and quickly.

This sheet, together with a copy of the Specifications, is now sent to the detailing room, where it is placed in the hands of a draughtsman, who makes the details according to these specifications and the practice of the bridge company.

ART. 118. DETAILING THE BRIDGE.

The full set of drawings for this highway bridge comprises ten sheets, of which only five are here published. The size of the original drawings between border lines was 15 × 25 inches, the lettering being made somewhat larger than usual in order to permit of satisfactory reduction. These drawings are as follows:

Sheet 1. Stresses and Sections (Fig. 137).
Sheet 2. Stringers and Floor Beams (Fig. 139).
Sheet 3. Intermediate Posts (Fig. 140).

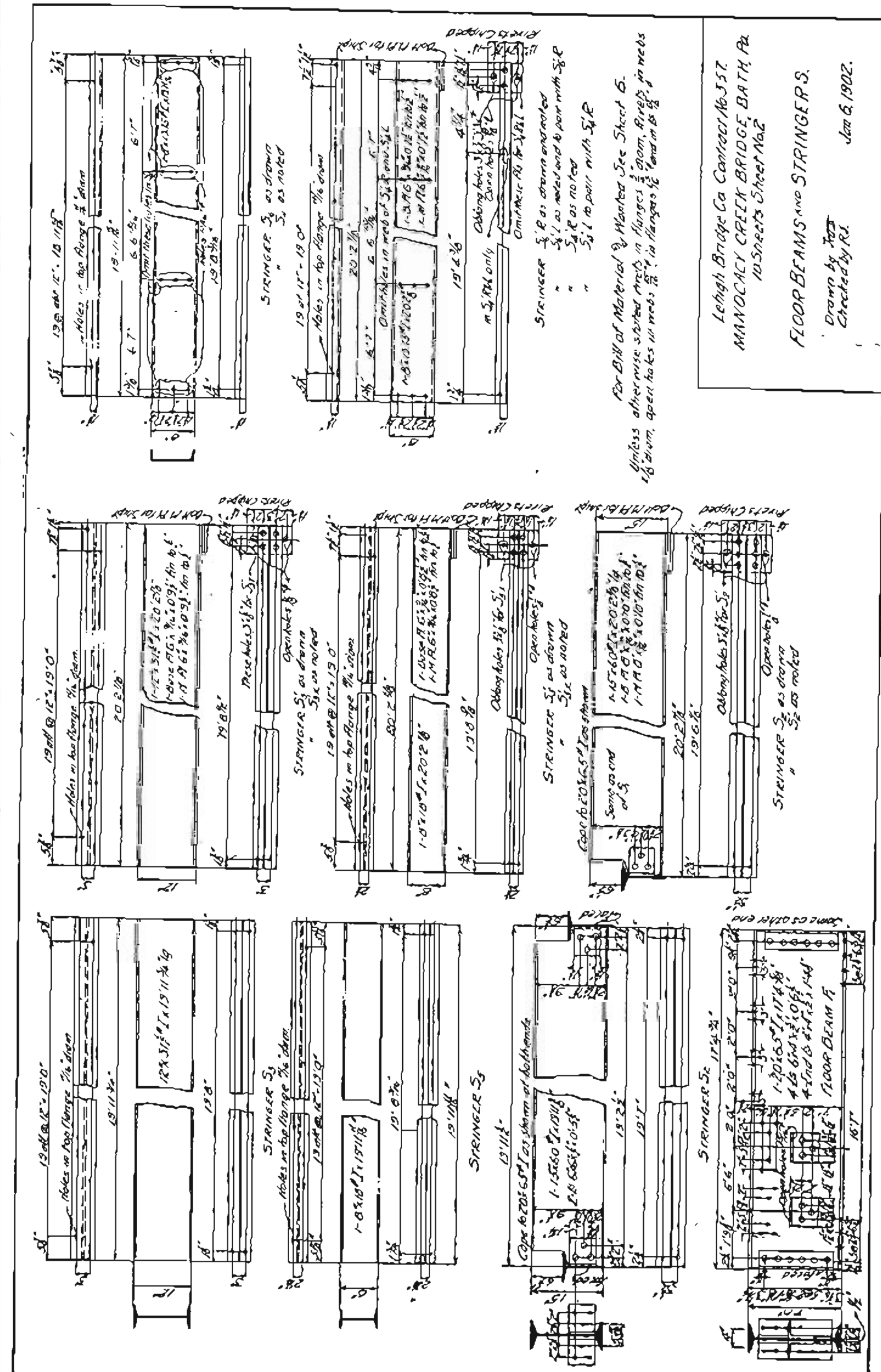


Fig. 139.

Sheet 4. End Posts and Top Chords U_3U_3 (Fig. 141).

Sheet 5. Top Chords U_1U_2 and U_2U_3.

Sheet 6. Portal Bracing and Foot-walk Bracket.

Sheet 7. Pedestals and Roller Nests (Fig. 142).

Sheets 8, 9, and 10. Top Laterals, Bottom Laterals, and Transverse Bracing.

In addition to these drawings bills for eye-bars, loop and adjustable eye-bars, field rivets, and bolts are prepared on printed forms, on which are noted the final dimensions and lengths, and also the additional lengths needed to make the heads of bars. If the bar is an adjustable member, the additional length for the upset for the screw is given and a note is made stating whether a turnbuckle or a sleeve nut is to be used. The field-rivet bill gives the number required of each diameter and also the length of shank required for the necessary grip.

On each sheet of detail drawings there is usually placed a Bill of Material and a "Wanted" list, both in tabular form, but such tables are omitted on Figs. 140–142 on account of lack of space. When a sheet is crowded a note may be made, as seen on Fig. 139, that these lists are given on one of the following sheets.

SHEET 2. FLOOR BEAMS AND STRINGERS. — Unless otherwise stated, the rivets used in the flanges are $\frac{3}{4}$ inch and those in the webs $\frac{7}{8}$ inch in diameter. The beams are drawn to scale in depth and width, but are shortened in the direction of length in order to place a large number on one sheet. For each beam there is shown the top view, side elevation, and transverse and longitudinal sections. The holes in the top flanges are for $\frac{5}{8}$-inch bolts, with which the $1\frac{1}{2}$-inch nailing strips are connected. The holes at the ends of the lower flanges are for rivets which connect the stringers to the floor beams. A clearance of $\frac{1}{4}$ inch is allowed between the ends of roadway stringers, and

$\frac{5}{16}$ inch is allowed between the ends of foot-walk stringers. All end stringers, except that of the inner foot-walk, have masonry and bearing plates. For the fixed end of the bridge open holes $\frac{3}{4}$ inch in diameter are left in these plates for the anchor bolts; for the roller end slotted holes are cut to allow for movement due to temperature. The rivets shown, which of course only go through the bearing plate, are countersunk and chipped on the side next to the masonry plate. The end inner foot-walk stringer is seen connected to the end post by a bent plate. This arrangement is necessary, as the shoe of the truss prevents the masonry of the pier from being built close enough to allow the end of the stringer to bear upon it. The distance of the rivet hole from the end of the stringer can either be computed or the detail laid out to a large scale and measured off directly. The masonry plate is usually shipped bolted to the bearing plate as noted. The size of the plates will, in most cases, exceed those previously calculated.

The trolley stringers, being of greater depth than those of the roadway, are connected in the manner shown in order to bring the tops to the same elevation. The top flange and part of the web is cut to allow the flange of floor beam to fit in; this operation is called coping. The reaction of the trolley stringer is 20 370 pounds, and there are six field and three $\frac{7}{8}$-inch shop rivets in single shear. Their aggregate strength is $3200 \times 6 + 4800 \times 3 = 33\,600$ pounds, which shows the joint to be amply strong in shear. The strength of the joint in bearing is that of the three shop rivets in web of stringer and the three in web of floor beam, and is $3 \times 6280 + 3 \times 0.59 \times 0.875 \times 14\,400 = 43\,840$ pounds, showing it to be safe in bearing. The thickness of the angles must be sufficient to take the bearing stress. It is not the best practice to connect a leg of an angle with only two rivets, hence three are used, although giving an excess of strength. The small shelf angle should always be used, if possible, even if not required for strength, as it is of great convenience in erection, enabling the field rivets to be driven without blocking up the stringer. It is to be noted that where connection angles are used the beam is cut $\frac{1}{16}$ inch short at each end and the angles faced off true to length (§ 116, Cooper). The end reaction of the floor beam is 34 400 pounds. The number of shop rivets required in bearing in the $\frac{1}{2}$-inch web of the floor beam is $34\,400/6280 = 6$; the number of field rivets required in the end connection in single shear is $34\,400/3200 = 11$; the number of field rivets required in bearing in the 0.382-inch web of the post channels is $34\,400/3200 = 11$, but 12 will be used. In the bottom flange of the floor beam are holes to receive the plate of the lower lateral connection; this connection should be plotted to scale to determine the size of the plate. The component of stress in the end panel tension member parallel to the floor beam is $18 \times 27\,180/26.9 = 18\,200$ pounds, and as a $\frac{3}{8}$-inch plate is used, the joint will be weak in shear and $18\,200/2360 \times 1.4 = 6$ is the number of $\frac{3}{4}$-inch rivets required in single shear. On account of the large size of the plate 8 rivets are used in order that the unsupported width shall not be too great.

SHEET 3. INTERMEDIATE POSTS AND HANGERS. — As the posts U_2L_2 and U_3L_3 only differ in length, they are detailed together. The details, such as batten plates and lattices, are the same, and thus only two sets of dimensions are needed, one drawing doing for both. The batten plates and lattices are determined by § 111 of the Specifications. The batten plates should be so placed that they do not interfere with the diagonals. The maximum stress which can come on the pin at the lower end is $20\,000 + 29\,400 + 1300/2 = 56\,780$ pounds. By § 54 of the Specifications $56\,780/(18\,000 \times 4.5) = 0.692$ inch is the width of bearing area required on a 4.5-inch pin. As the thickness of the channel web

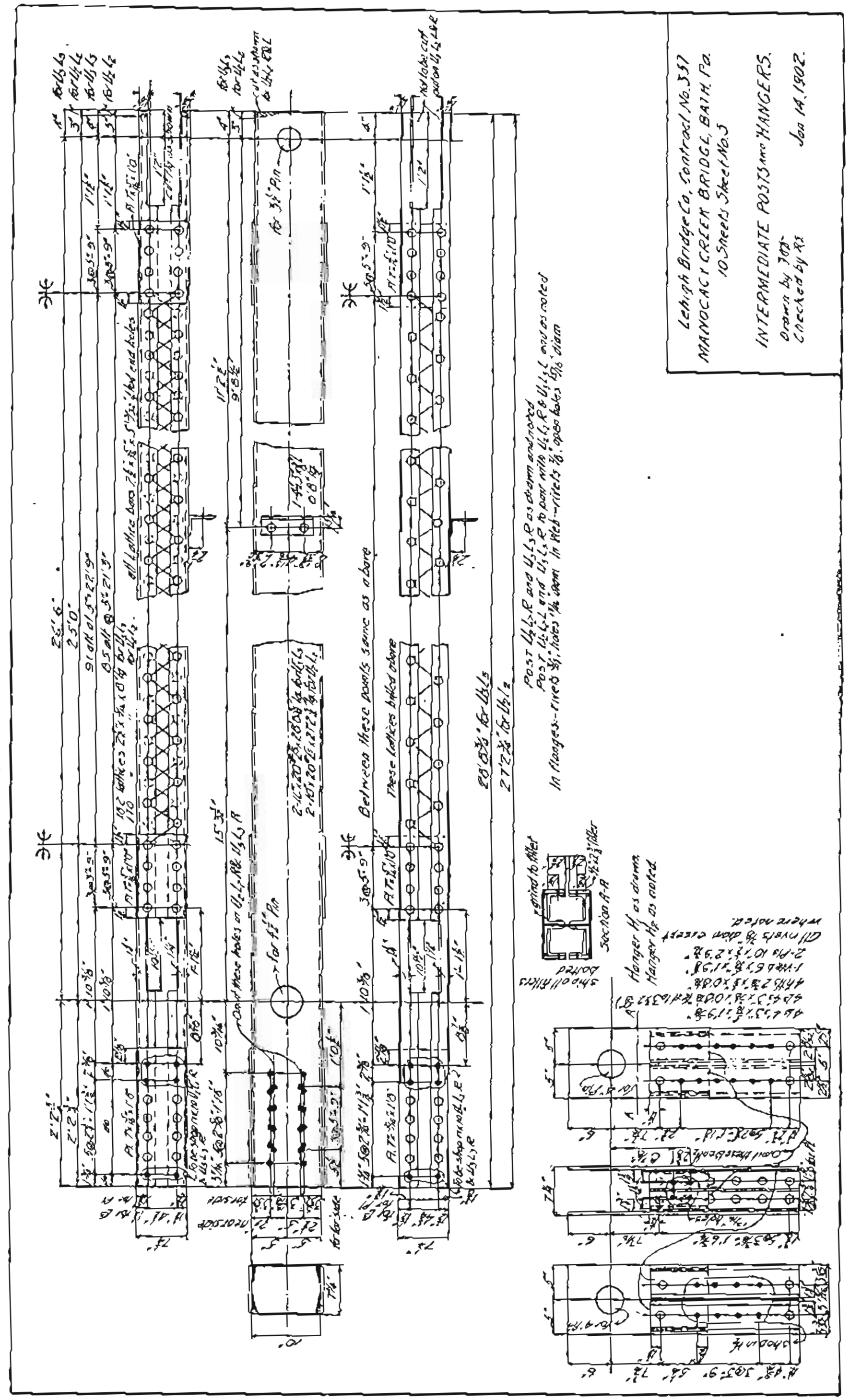


Fig. 140.

is 0.764 inch, no pin plates are required. The maximum stress on the $3\frac{1}{2}$-inch pin at the top is 20 800 pounds, and by a similar computation 0 32 inch is the total thickness required, showing that no pin plates are required. The flanges of the channels are cut to allow the diagonals to be placed closer to the channel web, and thus reduce the bending on the pins. This of course weakens the section, and an investigation as to strength should be made. The maximum tensile strength which U_3L_3 must be designed to resist (§ 51, Cooper) is $(10\,400 + 4400/2) \times 1.8 =$ 22 000 pounds. By cutting off the flanges the area is reduced by 2.2 square inches, and this is further reduced by 2.68 square inches on taking out the section of the $3\frac{1}{2}$-inch pin. This leaves 6.88 square inches, which is capable of standing $6.88 \times 12\,500$ $= 86\,000$ pounds, showing the section to be safe.

At the lower end two $\frac{6}{16}$-inch plates are riveted to the post, one on each side. These take the place of a diaphragm, and their function is to cause both channels of the post to take an equal amount of load. The total number of rivets required, on the foot-walk side, in one side of both plates, must be sufficient to transfer one-half the vertical stress in the post from one side to the other. This shear is equal to one-half the sum of the maximum shears of the foot-walk and floor beam, or $\frac{1}{2}$ (13 000 + 34 400) = 23 700 pounds. This requires seven $\frac{3}{4}$-inch shop rivets, or 4 on each side of each side plate, but more are used to keep the rivet spacing less than 5 inches. These plates should be as long as the depth of the floor beam, and the top should be even with the top of floor beam, and clearing the heads of the eye-bars of lower chord. In these plates are holes for the connection of floor-beam and foot-walk bracket; the posts opposite the foot-walk side of bridge have these foot-walk holes omitted. The number of rivets for the floor-beam connection was computed in Art. 106. The number required for the shear of the foot-walk bracket is six $\frac{3}{4}$-inch field rivets in single bearing in the $\frac{6}{16}$-inch plate. The open holes in these plates, in the flanges of channels, and in the posts on the foot-walk side of bridge, are for the connection of foot-walk bracket, which consists of 3×3 inch angles at bottom and 6×3 inch angles at top, riveted to the flanges of channels, in order to have rivets in shear only, and not in tension, as is too often the case. The small angle on the inner side of the post is for the connection of the transverse bracing, and should have holes to take the standard gage of the bottom flange angles of the bracing. It should also be placed at a distance of 15 feet above base of rail (§ 10, Cooper).

The hanger shown is simply a counterpart of the lower end of the post, except the side plates. Its function is the same as the end of post, the diaphragm web transmitting one-half the shear. Here $\frac{7}{8}$-inch instead of $\frac{3}{4}$-inch rivets are used. The required thickness of web is $23\,700/(8000 \times 20) = 0.148$ inch (§ 53, Cooper). A $\frac{5}{16}$-inch web must be used (§ 97, Cooper). The number of rivets required in bearing in this web, to connect the angles to the diaphragm web, is $23\,700/3930 = 6$. The number of rivets required to connect angles to side plates is $23\,700/3930 = 6$. More than the required number are used on each side, in order to allow for the floor-beam and foot-walk bracket connections, and in order to keep the rivet spacing within the allowable limit. The connections of the floor beam and foot-walk should be so arranged that their tops may be the same distance below the center line of pins as those which are connected to the posts. To allow for the foot-walk connection, angles are riveted on and brought out level to the side plates by means of small fillers; these will of course be omitted on the other side of the bridge. The number of rivets required in the flange connection of the foot-walk is determined as follows. The maximum top-flange stress in the foot-walk bracket is 23 700 pounds, and the bearing value of a $\frac{3}{4}$-inch field rivet in a $\frac{5}{16}$-inch plate at a bearing value of 12 000 pounds per square inch is 2810 pounds.

The number of rivets required is $23\,700/2810 = 8.5$, and 8 will be used. At the bottom four rivets are used to keep the bracket from lateral motion.

The box form of post is expensive from a shop point of view on account of the difficulty in riveting, but it is better in appearance, and has no outstanding edges to interfere with traffic. It also has the advantage over others of throwing the material in the web nearer to the outer surface of post, and thus allowing a closer compliance with § 102 of the Specifications.

SHEET 4. END POSTS AND TOP CHORD U_3U_3.—The end post will first be considered. According to § 48 and § 133, the end post is not a top chord, and hence camber will not be considered. The batten plates are required to be $1\frac{1}{2}(12+\frac{1}{2}+\frac{7}{16}) = 19\frac{3}{4}$ inches long and $\frac{5}{16}$ inch thick (§ 97 and § 111, Cooper), and they will be $15 \times \frac{5}{16} \times 21$ inches. The distance between gage lines is $8\frac{3}{4}+2\times 1\frac{3}{4}=12\frac{1}{4}$ inches, and $12.25 \times \tan 60° = 7.1$ inches, which is the maximum allowable spacing for single lattices. Seven-inch spacings will be used, thus making $3\frac{1}{2}$-inch spacing for the flats. The unsupported lengths of lattices is $(7^2+12.25^2)^{\frac{1}{2}} = 14.1$ inches (§ 111, Cooper); the thickness must be $14.1/40 = 0.353$ inch $= \frac{3}{8}$ inch, and the width $2\frac{1}{4}$ inches. By § 110 of the Specifications the pitch of rivets in cover plate must not be greater than $4 \times \frac{3}{4}$ inch $= 3$ inches for a length of $2 \times 15 = 30$ inches. The pitch of the remainder should not exceed 5 inches. Care must be taken that the batten plates do not interfere with the lower chord and the hip vertical U_1L_1. The lower flanges of channels are cut back to allow U_1L_1 to be placed close to the web, and thus reduce bending stress on pin. The webs of the channels are cut back from the miter line $\frac{1}{8}$ inch, thus allowing a $\frac{1}{4}$-inch clearance for rotation around the pin. The top plate at the hip takes no stress, it being simply to prevent lateral motion and also to prevent dust and water from entering the joint. To give room for the inner foot-

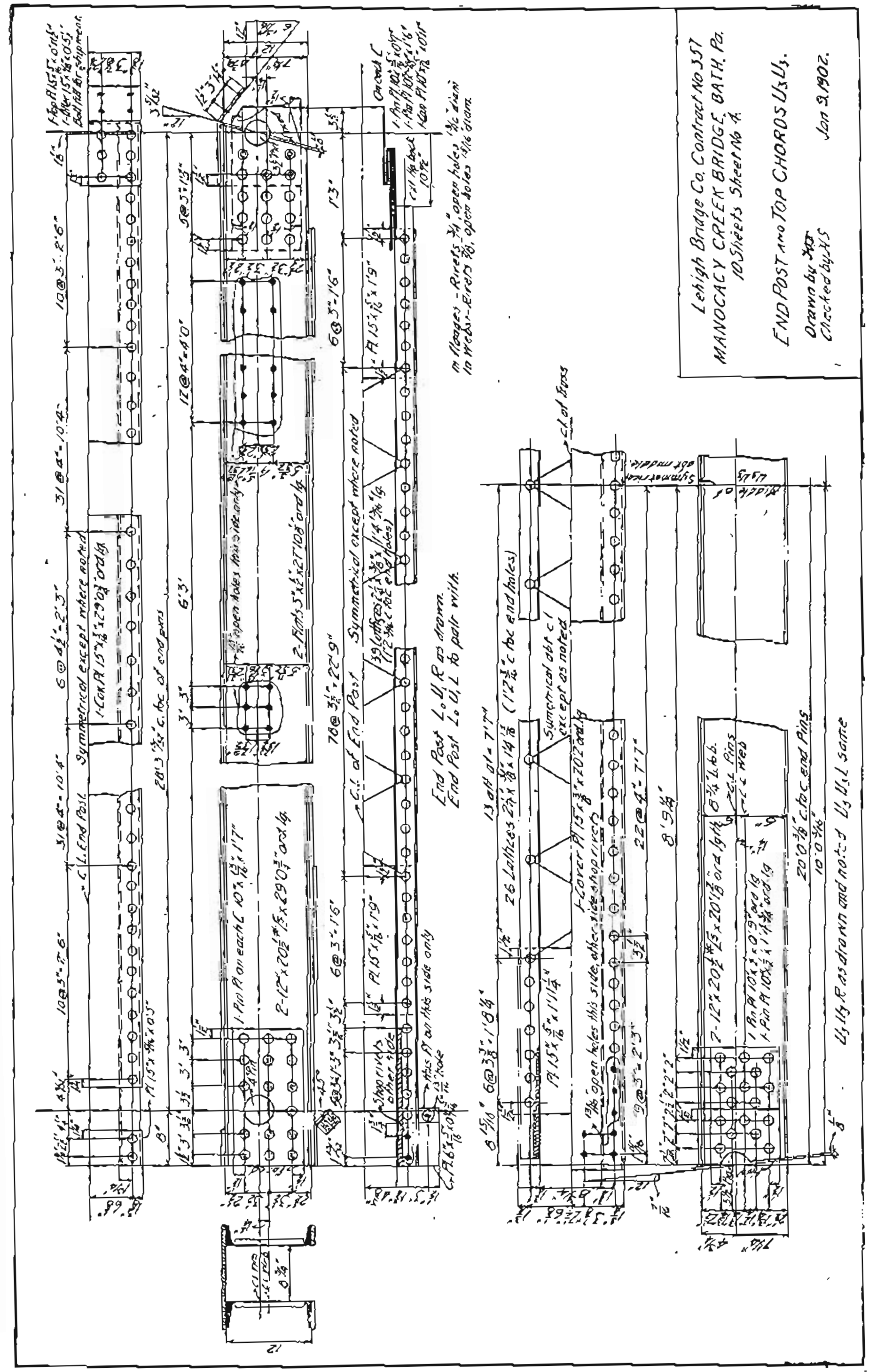


FIG. 141.

walk stringer connection, it was necessary to extend the lower end of the end post 8 inches beyond the center of pin.

The pin plates, together with the required number of rivets, will now be computed (see Chap. IX). The total stress is 152 100 pounds, which is transferred to the pin equally by the two channel webs. The total bearing area required on each side is 76 050/18 000 = 4.21 square inches, and the total thickness of bearing area on one side on a $3\frac{1}{2}$-inch pin is 4.21/3.50 = 1.21 inches. The channel web being 0.28 inch thick, the pin plates must be 1.21 − 0.28 = 0.93 inch thick. They will consist of one $\frac{5}{8}$-inch and one $\frac{5}{16}$-inch plate, both placed outside so as to allow U_1L_1 to fit closely against the side of the web and thus reduce the bending stress on the pin. An additional $\frac{5}{16}$-inch plate with full pin hole is placed on top of last plate to facilitate erection.

The $\frac{5}{8}$-inch pin plate carries $0.625 \times 3\frac{1}{2} \times 18\,000 = 39\,400$ pounds stress, the web $0.28 \times 3\frac{1}{2} \times 18\,000 = 17\,600$ pounds stress, and the $\frac{5}{16}$-inch pin plate $76\,050 - (39\,400 + 17\,600) = 19\,050$ pounds stress. The value of a $\frac{7}{8}$-inch shop rivet in bearing in a 0.28-inch plate is 4400 pounds, and the number of rivets needed in bearing in the 0.28-inch web is $(39\,400 + 19\,050)/4400 = 13.2$, or say 13, as it is within $2\frac{1}{2}$ percent variation (§ 162, Cooper). There are required 58 450/6010 = 10 rivets in shear to connect both plates to web, 58 450/9850 = 6 rivets in bearing in the $\frac{5}{8}$-inch plate, 19 050/4920 = 4 rivets in bearing in the $\frac{5}{16}$-inch plate, and 19 050/6010 = 4 rivets in shear to connect the $\frac{5}{16}$-inch to the $\frac{5}{8}$-inch plate. From this it will be seen that the joint will fail first in bearing in the web, and that 13 rivets are required, 4 of which must pass through the $\frac{5}{16}$-inch plate.

The pin plates for the lower end, or joint L_0, will next be computed. Here both plates are placed on the outer side of

the post in order to allow the vertical plates of the shoe to be packed as closely to the backs of the webs of the channels as possible. A 4-inch pin being used, it is found, in the same manner as before, that one $1\frac{3}{16}$-inch plate is needed on each channel. Thirteen rivets are needed in bearing in the channel web to make the connection safe. Rivets are countersunk and chipped, where necessary, in order to allow members to fit close up to the web. When the views of a member are symmetrical about an axis, or nearly so, it is usually the practice to draw only one-half of such views; this occurs here, one-half views being drawn and exceptions noted. The top chord section U_3U_3 being symmetrical about two axes is drawn as shown, and, in order to economize space, the one-quarter top views and longitudinal sections are placed together. Open holes are shown on top to take the connections of the transverse and the top lateral bracings and a top plate. By § 133 of Specifications $\frac{3}{8}$ inch must be added to the length from center to center of pins. In this case, a $3\frac{1}{2}$-inch pin being used at both ends, the pin plates will be the same. The maximum stress is 133 100 + 29 400 = 162 500 pounds, one-half of which is transmitted to the pin by each side of the post. Following the same method as employed in the end post, the bearing area needed on one side is 1.285 inches, and the thickness of the pin plates 1.005 inches. Two $\frac{1}{2}$-inch plates will be used, both on the outside, as the post will not allow clearance enough to be placed on the inside. Fifteen $\frac{7}{8}$-inch rivets are required in the channel web, 5 passing through the outer plate. One more than the required number is used in order to make spacing symmetrical. Rivets in the top cover plate are spaced according to § 66 and § 110 of the Specifications. It will be noticed that, in the pin plates, the first row of rivets next to the edge of the smaller plate is closer than $1\frac{1}{4}$ inches, the distance required by the shop in order to drive a rivet. This is not, however, a violation of shop practice, as that row of rivets can be driven before the smaller plate is put on, thus saving material in the length of the pin plate.

SHEET 5. TOP CHORD SECTIONS U_1U_2 AND U_2U_3. — The detailing of these presents no new features. Camber, rivet spacing, batten plates, and lattices are the same as before. Plates with full pin holes and top plates should be placed at the upper ends of each chord section. Open holes should be placed at the ends in order to take the connections of transverse and lateral bracings, and of top plates.

SHEET 6. PORTAL BRACING AND FOOT-WALK BRACKET. — As the portal bracing is symmetrical about its middle, only one half is required to be drawn. The connection plates are $\frac{5}{16}$ inch from back to back (Art. 116). At intermediate points rivets are spaced every 8 or 10 inches, and the angles are kept apart by two round washers $\frac{3}{8}$ inch thick and $2\frac{1}{4}$ inches in diameter. In the connection of the knee brace the rivet holes for the connection to the end posts and top chords U_1U_2 correspond. The maximum shear is 16 850 pounds (see Art. 116), which requires (§ 53, Cooper) 16 850/4600 = four $\frac{7}{8}$-inch field rivets in bearing in the $\frac{5}{16}$-inch angles connecting the portal strut to the end post. The stress in the knee brace is 23 700 pounds. Its vertical and horizontal components are 16 850 pounds. To connect the knee brace to the end post four $\frac{7}{8}$-inch field rivets are required in bearing in $\frac{5}{16}$-inch angles, and to connect it to the portal strut three $\frac{7}{8}$-inch shop rivets in bearing are needed in the $\frac{5}{16}$-inch plate. Four $\frac{7}{8}$-inch field rivets are required for bearing in the channel web.

Each panel of the portal strut being about 4 feet, the length of a diagonal from center to center of gravity of flange angles is $(6^2 + 4^2)^{\frac{1}{2}}$, or about 7.2 feet, the secant of the angle which it makes with the vertical is $7.2/6 = 1.2$, and the stress in a diagonal is $1.2 \times 16\,850 = 20\,150$ pounds, the diagonals being considered as taking tension only. The number of $\frac{7}{8}$-inch shop rivets required for bearing in the ends is $20\,150/6900 = 3$. In detailing the diagonals the distance from center to center of end holes is given and the ordered length is billed on them.

The depth of the foot-walk bracket at its outer end is sufficient to allow a small clearance between flange angles. Plates $\frac{3}{8}$ inch thick are placed on the top flange to evenly distribute the reaction of stringers. Holes are placed in these corresponding to those in the ends of foot-walk stringers. The inner foot-walk stringer must clear the eye-bars and pins. The inner plate also takes the angles of the connection to the post. This plate has six $\frac{7}{8}$-inch shop rivets in it, in addition to the two field of the stringer connection. The strength of the joint in bearing in the $\frac{5}{16}$-inch angles is $(6 \times 3930 + 2 \times 2620) = 28\,820$ pounds, while the flange stress is only 23 700 pounds, thus showing it to be safe. The flange stress at the end is zero, the moment under the middle stringer is $4200 \times 2.5 \times 12 = 126\,000$ pound-inches, and taking the effective depth here as 11 inches, the flange stress is 11 400 pounds. The flange stress at the post, as previously computed, is 23 700 pounds. The difference of the flange stress between the end and the center is 11 400 pounds, requiring three $\frac{7}{8}$-inch shop rivets in bearing in the $\frac{5}{16}$-inch web. The difference in bending moments between the center and post is 12 300 pound-inches, requiring 4 rivets.

SHEET 7. PEDESTALS AND ROLLER NEST. — By § 130 of the Specifications, the vertical webs must be connected transversely when of sufficient height. This is done by means of angles and plates. The plates in this case should not be the full width of $8\frac{1}{2}$ inches, but less, in order not to interfere with the channels of the end post. The distance out to out of vertical plates is made $\frac{1}{4}$ inch less than the distance from back to back of channels for the same reason. The elevation of the under surface of the bearing plate should be of the same elevation as that of the under surface of a $\frac{5}{16}$-inch lateral connection plate on the bottom of the floor beams. This is necessary so that the angles of the lower lateral system may be horizontal. The $3\frac{1}{2}$-inch vertical leg of these angles should clear the masonry. This requires that the distance from the under side of the bearing plate to the masonry shall be greater than $3\frac{1}{2}$ inches. The rollers at the roller end render this distance sufficient, but at the fixed end the pedestal must be built up by flats. The open holes in the bearing plates are for the lower lateral connection. The stress in the end lateral is 27 180 pounds, and (§ 53, Cooper) six $\frac{7}{8}$-inch field rivets are needed in bearing in the $\frac{5}{16}$-inch plate. The number of rivets in the vertical legs of connection angles must be sufficient to bear the entire reaction of 115 950 pounds, as no reliance can be placed in the bearing of plates. This joint is weak in double shear, requiring a total of fourteen $\frac{7}{8}$-inch shop rivets. It is thought best to space the rivets near the minimum spacing limit. This gives 15 required at the roller, and 13 required at the fixed end. The number in the horizontal leg cannot be computed except from the horizontal thrust of bridge. This gives a total required number of six $\frac{7}{8}$-inch rivets in single shear, friction being neglected. In all cases where rivets extend to a surface required to be flat, those heads are countersunk and chipped. The rollers (§ 127, Cooper) are of machinery steel. They should be ordered $3\frac{3}{8}$ inches in diameter and finished to the sizes given. The small flats riveted on the roller sides of masonry and bearing plates at the roller end prevent lateral motion to a slight extent and keep the rollers in alignment. They should be finished so as to give a $\frac{1}{16}$-inch clearance on all three sides between the roller and itself. The holes for anchor bolts (§ 131, Cooper) in both the masonry plates and in the bearing plate of the fixed end must be $1\frac{3}{8}$ inches in diameter. For the roller end those in bearing plate are oblong in order

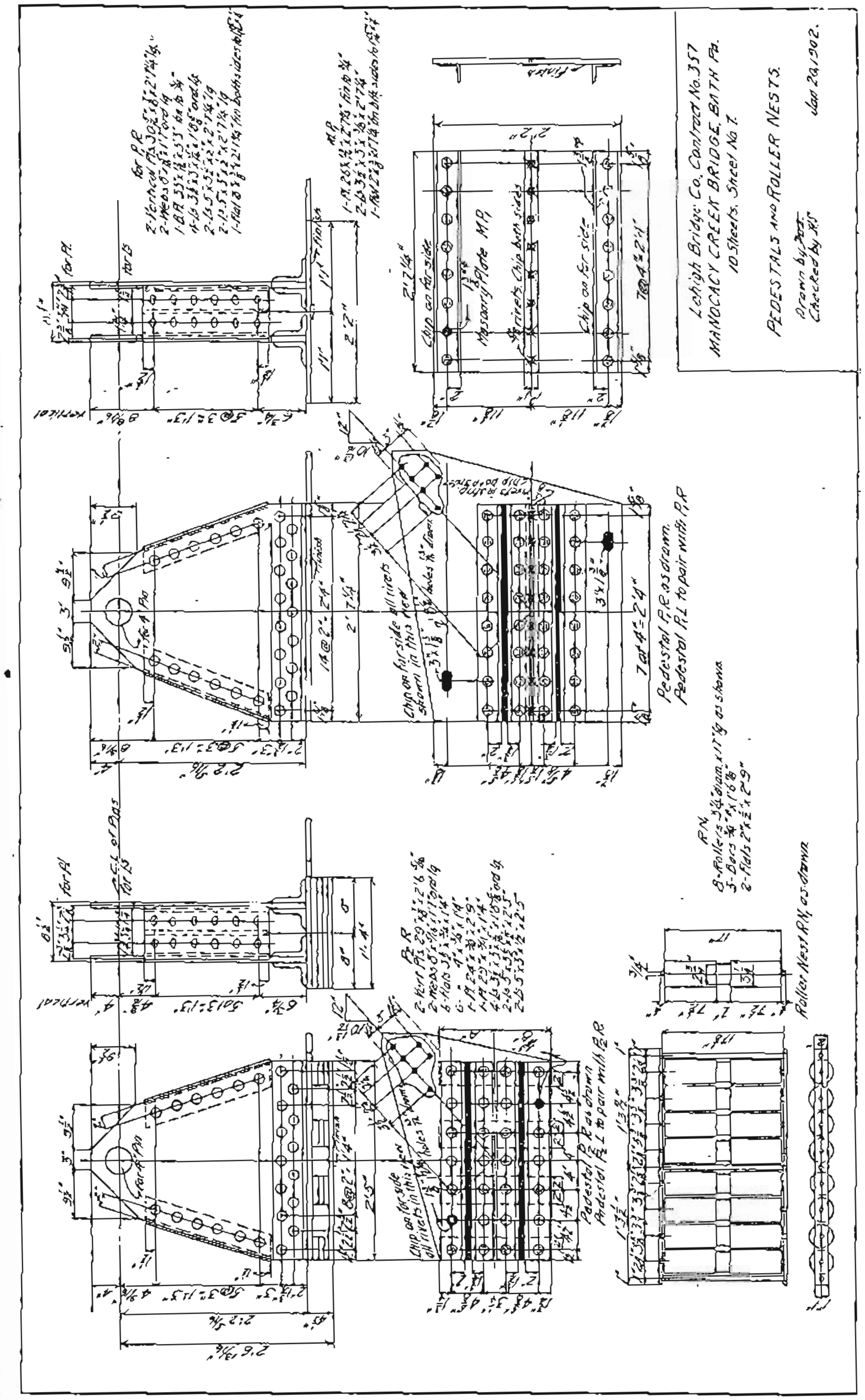


Fig. 142.

to allow for longitudinal motion. Their length is equal to the diameter of the anchor bolt plus the movement due to temperature.

SHEETS 8, 9, AND 10. TOP LATERALS, LOWER LATERALS, TRANSVERSE BRACING. — The detailing of these presents no new features over that of the portal strut. In all cases, the connection plates being $\frac{5}{16}$ inch thick, the joints will be weak in bearing in a $\frac{5}{16}$-inch plate. The stresses in the members are small, and rivets computed from these stresses do not give a sufficient number for good stiff details. Enough rivets to take up the entire strength of angles in tension should be used in all cases; when possible $\frac{7}{8}$-inch rivets should be used. Where two angles cross with backs in same plane and the legs on same side of the plane, as in lateral systems, one angle should continue, the other should be cut and a connecting plate used. This plate must be of sufficient cross-section to develop the full strength of angle.

ART. 119. ESTIMATE OF WEIGHT.

In practice the estimate of weight and cost is made long before the detailing is finished, in fact before the contract is let, and the exact weight is determined by weighing each member separately as it leaves the shop for shipment. For the student, the two corresponding operations are called approximate weight and computed weight. The former is made by formula or by comparing with some structure of like span and design, the latter by computing the weight, from the tables in handbooks, of each member, after it has been detailed, and, after multiplying by the number required, adding the results together, the sum being the computed weight of bridge. This sum is liable to an error of about one percent either way, due to inaccuracies of rolls in the mills. Each pair of $\frac{7}{8}$-inch rivet heads weighs about 0.45 pound, and each pair of $\frac{3}{4}$-inch rivet heads about 0.28 pound. The

following is an example of a convenient method of arranging the estimate of weight. It is for the intermediate posts of Sheet 3 (Fig. 140).

8 INTERMEDIATE POSTS. SHEET 3.

Number.	Shape.	Size in inches.	Length.	Weight per linear foot in pounds.	Total weight, pounds.
8	Channels	10 inch × 20 pound	28 ft. $8\frac{3}{4}$ ins.	20.00	4 600
8	Channels	10 inch × 20 pound	27 ft. $2\frac{3}{4}$ ins.	20.00	4 373
16	Plates	$7 \times \frac{5}{16}$	1 ft. 8 ins.	7.44	199
32	Plates	$7 \times \frac{5}{16}$	1 ft. 0 ins.	7.44	238
8	Angles	$4 \times 3 \times \frac{3}{8}$	0 ft. 8 ins.	8.50	45
1408	Lattice Bars	$2 \times \frac{5}{16}$	0 ft. 8 ins.	2.65	2 486
				Total weight in pounds =	11 941

A similar table made out for the four hangers of Sheet 3 determines their weight to be 772 pounds. The rivet heads should now be counted and added to the above, thus giving the computed weight of all the intermediate posts and hangers. In like manner, the members and details on each of the sheets may be tabulated, and thus the entire weight of the bridge can be determined with a precision closely equal to that of actual weighing on scales.

CHAPTER XI.

RAILROAD RIVETED BRIDGES.

ART. 120. FORMS OF TRUSSES.

As stated in Art. 70, the lower limit of span for riveted trusses ranges from 75 to 100 feet, and the upper limit from 120 to 150 feet, or even to 200 feet, according to different specifications and standards.

The types of trusses in most general use for riveted bridges are the Warren with sub-verticals, the Pratt, and the Baltimore. For the shorter spans where deck trusses cannot be built on account of local conditions, pony or half-through bridges are used whose floor system and transverse bracing of the trusses is similar to that of through plate-girder bridges. In larger spans, under the same conditions, the trusses are connected by lateral, portal, and sway bracing as in pin-connected bridges. Partial detail drawings of a pony truss whose span is 120 feet are given in Art. 121, those of a 170-foot through Pratt truss are shown on Plate VI, Art. 122, while the standard details of through Baltimore trusses for spans from 100 to 200 feet are shown on Plate VII, Art. 123. Illustrations of a Warren truss with sub-verticals for a span of 114 feet $3\frac{1}{4}$ inches may be seen on the inset of Engineering News, July 9, 1896.

The double intersection Warren truss is also used to some extent, sub-verticals being added where the panels are subdivided in order to secure a shallow floor. The use of more than a single system of web members is not regarded with favor in the

best practice because the stresses are not statically determinate. In some instances the stresses in double intersection Warren trusses are made indeterminate to a still higher degree by the insertion of long verticals connecting the two systems. This is contrary to the line of progress described in Chap. I.

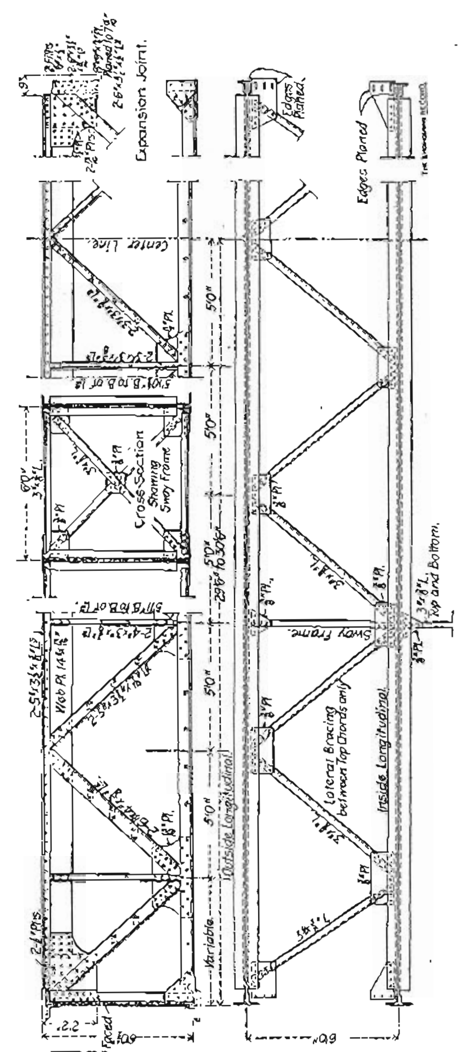


Fig. 143.

On elevated railroads, girders whose spans range from 40 to 65 feet are often required to be built with open webs in order to admit more light beneath the structure than the solid webs of plate girders. This requirement applies to most locations except those where the elevated structures occupy the middle of a very wide street. Fig. 143 shows the plan, elevation, and cross-section of a half span of the Boston Elevated Railroad. The deck trusses, whose depth is very nearly six feet, are of the Warren type with sub-verticals. The upper chord is subject to combined compression and flexure, and is composed of a pair of angles and a web plate, while the lower chord and all of the web members consist only of pairs of angles, separated by washers whose thickness equals that of the connecting plates. The lateral and sway bracing is the same as that of a deck plate-girder bridge.

ART. 121. DETAILS OF A LATTICE GIRDER.

Figs. 144, 145, and 146 show most of the details of a pony truss bridge whose span is 120 feet, taken from a standard plan of the Northern Pacific Railway. On the standard plan it is designated as a through lattice girder, and this term is very generally employed in practice, although strictly the term lattice girder applies to one with two or more systems of webbing.

Fig. 144 shows that the intermediate floor beams are connected to the sub-verticals above the lower chord. The end

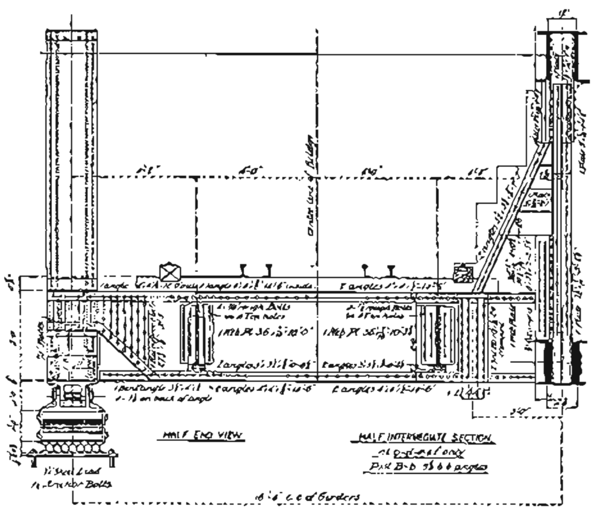


Fig. 144.

web plates do not extend to the top of the inclined stiffening angles, but only as high as the end connecting angles. The web splice plates are not limited to the clear space between the

flange angles, but are crimped over the vertical legs of these angles. The end floor beam is riveted by means of angles to the large plates connecting the corresponding sides of the end post and the lower chord. The lower flange angles continue straight to the lower end of the web plate, instead of being bent upward and extended over the end post, as on Plate IV. The lower flanges of all floor beams are riveted to the connecting plates of the lateral system. The trusses are spaced 16′ 4″ center to center, and the clear distance between the upper chords is 14′ 6″. Each stringer has a web plate $24'' \times \frac{7}{16}''$, and flanges composed of two angles $5'' \times 3'' \times \frac{3}{8}''$. The stringers have no separate lateral system, but each one is connected to both of the laterals of the bridge by means of short angles. Brackets in line with the stringers project 15 inches beyond the end floor beams, and support one cross-tie at each end of the span.

The sub-verticals are composed of two pairs of angles united by four web or tie plates, two of which are extended inward to connect with the inclined stiffening angles, as shown in Fig. 144. The diagonals Bc and cD (Fig. 145) consist of two pairs of unequal-legged angles united by a continuous web plate, while in the diagonals De and eF the angles are connected by four tie plates, the intermediate ones being much smaller than the end ones (Fig. 146). The upper chord is composed of two web plates, a cover plate, and four angles, the lower angles being larger than the upper ones. The increased chord section from D to F is provided by means of two additional web plates. The lower angles of the chord are united by tie plates and single lacing. The composition of the end post is the same as that of the chord BD. In the lower chord the angles have the same size throughout. From a to c the two web plates are only $\frac{3}{8}$ inch thick, from c to f there are four web plates $\frac{11}{16}$ inch thick, and from e to f two side plates are added in the clear space between the flange angles. The chord is laced on both top and bottom.

The joint or connecting plates are $\frac{3}{4}$ inch thick and are riveted to the inside of the webs of the chords and to the backs of the angles of the web members. Fillers are inserted at the

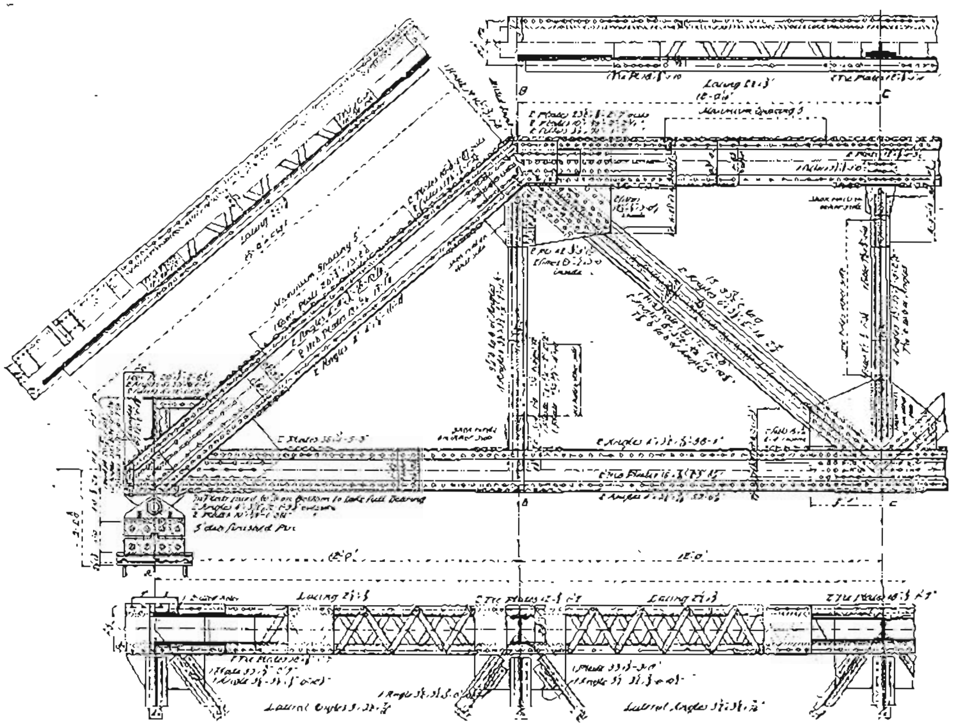


Fig 145

upper joints on account of the difference in the inside clear spacing of the two chords.

The truss is shipped in two parts and spliced in the field as indicated in Fig. 146. The connecting plates act as splice plates, and in addition two side plates and one cover plate are provided for the splice. At *B* two side plates, extending over

the vertical legs of the flange angles, as well as two fillers, are added as splice plates. The joints at both *B* and *F* are milled. The details of the end bearings are similar to those for plate girders. See the standard plan, Plate II.

This girder was designed for the same standard loading as

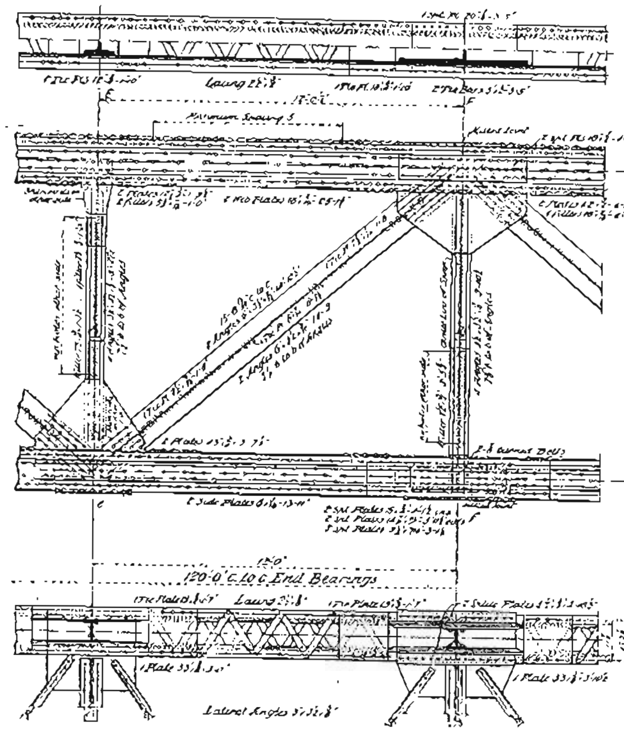


Fig. 145.

that described in Art. 102, and the weight of a complete span of the bridge is about 215 000 pounds. A riveted through span with overhead bracing was also designed for a span of 110 feet, and its weight was found to be 19 000 pounds less than that of the lattice girder of the same span; but since the former required more field-riveting than the latter, and as the absence of overhead bracing was regarded as a desirable feature, the girder span was adopted as the standard. See Journal Western Society of Engineers, vol. 6, page 53, Feb. 1901.

Joints and connections in riveted work, whether in tension or compression, are designed to develop the full strength of the members, proper provision being made for field riveting. The connecting plates must have a thickness proportioned to the amount of stress to be transferred, and must properly distribute the web stresses to the plates and shapes which compose the chords. The 1901 specifications of the Baltimore and Ohio Railroad limit the number of rivets in any connection of a $\frac{3}{8}$-inch plate to ten.

When but few lines of rivets are used in any connection and the lines are long, the elongation of the member within the limits of the connection makes a very unequal distribution of the stress to the rivets. This consideration will often determine the question whether to employ angles with legs wide enough to permit two rows of rivets instead of one.

Where a number of rows of rivets are inserted in tension members, it is important to make a sufficient deduction for rivet holes. See Experiments on Iron and Steel Joints Riveted on Angle by B. B. FLINT in Transactions of American Society of Civil Engineers, vol. 27, page 406, Oct., 1892. See also Net Section in Riveted Work by THEODORE COOPER in Railroad Gazette, vol. 22, page 583, Aug. 22, 1890.

ART. 122. DETAILS OF A PRATT TRUSS.

Plate VI contains parts of a general drawing of a riveted Pratt truss whose span is 170 feet. It was taken from one of a series of standard plans of riveted bridges having a considerable range of span, and which were designed for class W of WADDELL'S compromise standard live loads (Art. 32). All

the material is medium steel except the rivets and anchor bolts, which are of soft steel. The trusses are spaced 17 feet apart between centers, while the stringers are spaced 7 feet apart.

The stringers (not shown on Plate VI) have web plates $39\frac{3}{4}'' \times \frac{3}{8}''$, and flanges of two angles $6'' \times 4'' \times \frac{1}{2}''$, the long legs being horizontal. There are seven intermediate pairs of stiffeners crimped over the flange angles. The end connecting angles have fillers under them twice as wide as the angles. The lateral system of the stringers in each panel consists of four diagonals, each composed of one $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle. The stringers are connected by means of two short angles to the lower laterals of the bridge in a very effective manner in accordance with the specification given in Art. 94. The connection of one of the transverse braces between the lower flanges of the stringers is shown on the plate.

The floor beams have intermediate stiffeners. Their connection to the posts and suspender is very simple, since these members have the same width as the stiff lower chord. The web of the end floor beam is reënforced at each end by a plate 25 inches wide inside of the end connections. A diaphragm like that in the verticals is placed between the large connecting plates at the panel point L_0.

The posts and suspender have sections like those for pin-connected trusses. The diagonal U_1L_2 consists of two plates and four angles united by a single line of lacing, while the diagonal U_2L_3 has two channels with the flanges turned inward and united by two lines of lacing. In the middle panel there are two stiff diagonals, each composed of two pairs of angles connected by a single line of lacing. Both diagonals are cut at their intersection and riveted to a pair of connecting plates.

The upper chord and end post consist of a cover plate and two channels connected below by tie plates and single lacing.

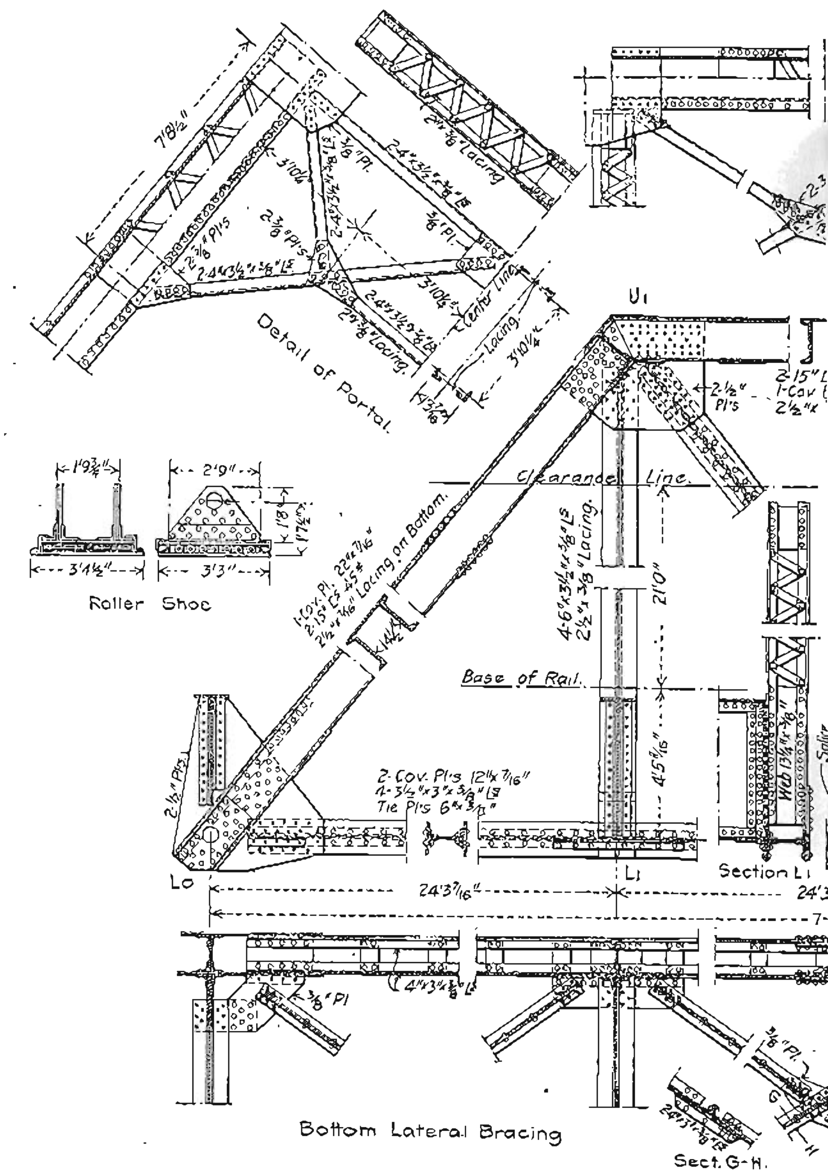
The splices of the upper chord are similar to those of a pin-connected truss. From L_0 to L_2 the lower chord section is composed of two web plates and four angles united by a series of narrow tie or batten plates, as shown on the drawing. From L_2 to L_3 two plates of the same section are added and the angles increased to $4'' \times 3'' \times \frac{7}{16}''$, while in the middle panel the chord consists of two plates $12'' \times \frac{9}{16}''$, two plates $12'' \times \frac{5}{8}''$, and four angles $4'' \times 3'' \times \frac{7}{16}''$. The entire chord is spliced on the left of L_2, the composition of the splice being given on the plate. There is a similar splice also at the left of L_3.

The upper laterals are made up of two angles $4'' \times 3'' \times \frac{3}{8}''$, laced together so as to form a stiff member as deep as the chord, and attached by connecting plates to the top and bottom of the lateral struts as well as to the chords. Each of the lower laterals consists of two angles $4'' \times 3'' \times \frac{3}{8}''$, placed with the $4''$ leg vertically and riveted together every foot. The splice at the intersection of the laterals has two angles of the same size in addition to the $12'' \times \frac{3}{8}''$ plate in order to give stiffness as well as strength to the splice. The end connecting plates are riveted to the bottom flanges of the floor beams and to the shelf angles attached to the side of the chords.

The portal bracing consists of two small trusses, one of them connected to the upper and the other to the lower side of the end posts. All the corresponding members of these trusses are laced together in pairs, thus making a portal of considerable stiffness in all directions. The construction of the intermediate sway bracing is fully shown on the drawing.

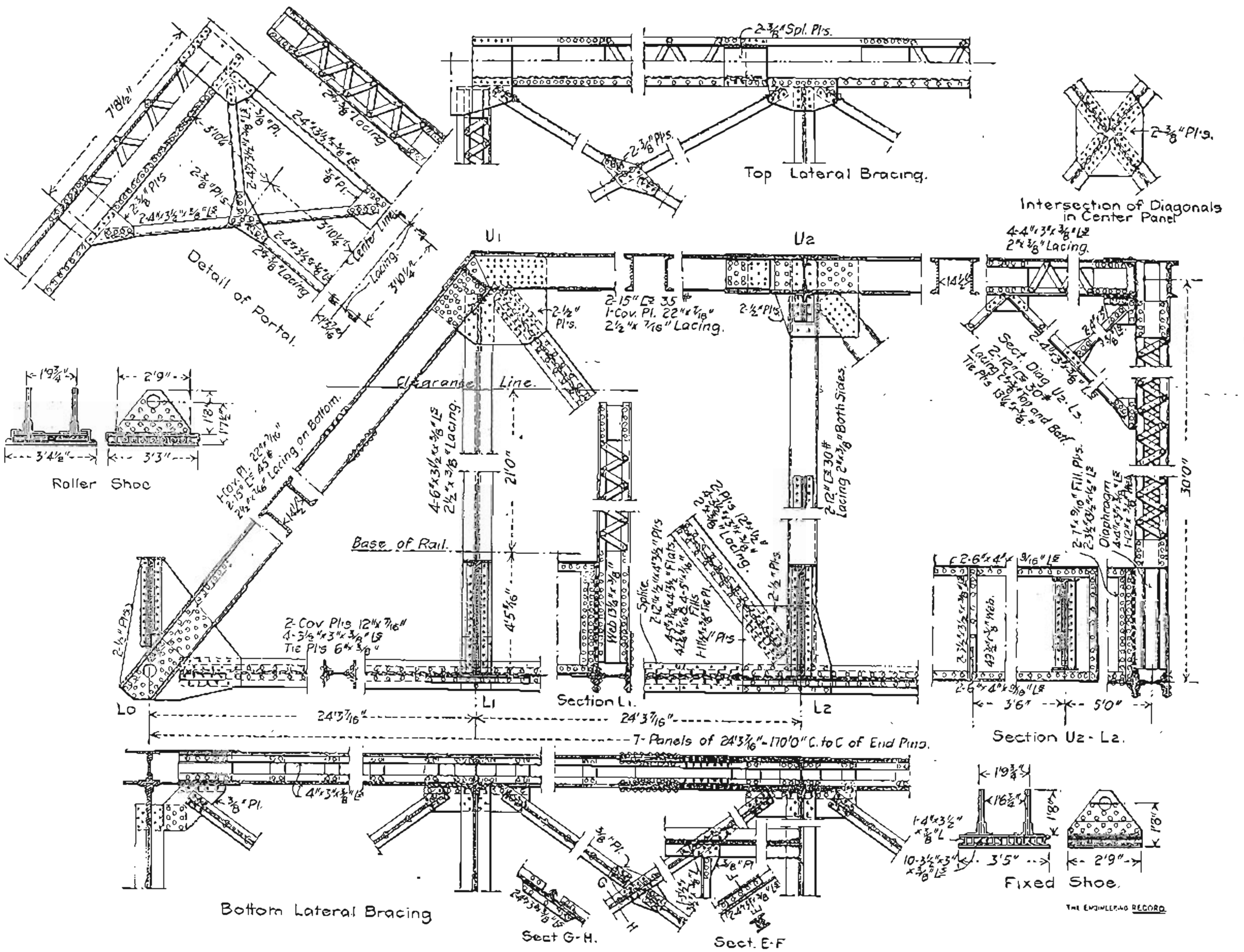
The connecting plates at the different panel points are all $\frac{1}{2}$ inch in thickness. They are riveted to the inside of the upper chord and end posts, and to the outside of all the other members. The reaction of the panel point L_0 is transferred from the end post to the pedestal by a 6-inch pin, the necessary bearing

Single-track Through Riveted Tr



PARTIAL DETAIL DRAWING OF ONE OF THE RIVETED BRI
FOR THE VERA CRUZ AND PAC

Single-track Through Riveted Truss Bridge. Span, 170 Feet.



PARTIAL DETAIL DRAWING OF ONE OF THE RIVETED BRIDGES DESIGNED BY WADDELL AND HEDRICK IN 1899 FOR THE VERA CRUZ AND PACIFIC RAILWAY IN MEXICO.

CHAP. XI.

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of the end post being secured by means of $\frac{5}{8}$-inch pin plates in addition to the large connecting plates. On account of the eccentric location of the pin, two angles are inserted to reduce the effect of this eccentricity on the pin plates and to aid in transferring its share of the stress into the cover plate.

The general construction of the pedestals or shoes at both ends is indicated on the drawing. The rollers are 3 inches in diameter. In the first shoe the $3\frac{1}{2}$-inch vertical legs of the angles are planed to 3 inches. The web or pin plates of the pedestals as well as the bearing and bed plates are $\frac{3}{4}$ inch thick, while the connecting angles are $6'' \times 6'' \times \frac{3}{4}''$. The anchor bolts are of soft steel $1\frac{1}{4}$ inches in diameter and 2 feet long, having cold-pressed threads and foxed ends.

The ends of stringers and floor beams are faced as well as those of abutting compression members in order to secure perfect contact. All rivet holes are punched $\frac{1}{8}$ inch less and reamed to $\frac{1}{16}$ inch greater diameter than that of the rivet. All truss members are assembled in the shop and the field-rivet holes reamed to a perfect fit.

In the shorter spans the diagonals U_1L_2 are made of two channels laced together instead of two pairs of angles similarly connected, and the sections of the end post are balanced by riveting flats to the lower flanges of the channels.

A riveted truss bridge of the same type whose span is 150 feet is described and illustrated in Engineering News, vol. 40, page 114, Aug. 25, 1898. It was designed by the same engineers for the Kansas City, Pittsburg and Gulf Railroad. Two of the principal differences in the details consist in the upper chord being laced on both the upper and lower sides, and in the portal bracing being a simple lattice girder, combined with knee braces, attached to the upper side of the end post. See also two communications on the comparative economy for short spans of riveted trusses and the "A" type of pin-connected trusses, on page 346 of the same volume.

ART. 123. DETAILS OF A BALTIMORE TRUSS.

Plate VII shows the standard details for riveted bridges adopted by the New York Central and Hudson River Railroad. One of the reasons for adopting the Baltimore truss for riveted bridges was the necessity, in a number of instances, of shallow, solid trough floors requiring short panels. A large number of these trusses have been built during the recent extensive renewals on the principal lines of this railroad, involving in the aggregate, up to 1902, about 70 000 tons of steel bridges.

The plate shows the details of the connection of rectangular trough floors to the lower chord whose depth is greater than that of the upper chord, since it is subject to combined flexure and tension. When the floor system consists of stringers and floor beams, the bottom flanges of the floor beams are almost even with the bottom of the lower chord, and in order to secure adequate connections to the verticals of the trusses the web is spliced near each end, and the end web plates extended upward as a gusset plate or knee brace, in the same manner as for through plate girders (see Fig. 56). The splice plates extend beyond the ends of the outer stringers, thus serving also as filler plates, and these, together with the web plate, are slotted over the upper flange angles of the chord.

The short suspenders and the short diagonals, as well as the long sub-verticals in the smaller spans, are made up of two pairs of angles laced together. The long suspenders either have two plates in addition to the four angles, or they consist of two channels laced on both sides. The long diagonals and the lower chord consist of built-up channels with additional

web plates on the inside, or filler plates between the angles on the outside, in order to enlarge the section when needed. The sections of the upper chord and end post are similar to those which are built up of plates and angles for pin-connected trusses of short spans.

Both the portal and intermediate sway bracing consist of lattice girders with multiple systems of webbing, the flanges being composed of two angles and a plate which is wide enough for the connections of the web bracing. Brackets are also used, those of the portal having a solid web while the others have only a single pair of angles to stiffen the main knee-brace angles. The upper laterals are composed of two pairs of angles laced together so as to form a member as deep as the chords to which they are connected. The lower laterals consist of two angles riveted back to back, and are spliced to a common connecting plate which is riveted to the bottom flange at the middle of alternate floor beams. By means of angle clips they are also connected to the lower flanges of the stringers. The end connecting plates are attached to the bottom angles of the lower chords.

The lower ends of the trusses are connected by end floor beams, and are supported by pedestals whose pin bearings are located below the lower chord. The web or pin plates of the pedestals are stiffened by outside angles and by a connecting diaphragm, the pin taking bearing throughout its length. Alternative details are given for nests of cylindrical and segmental rollers respectively.

Figs. 147–149 give three views of one of the fixed spans of the New York Central and Hudson River Railroad passenger bridge at Albany, N. Y. The span is $181\frac{1}{2}$ feet long. The trusses are $30\frac{1}{3}$ feet deep and are spaced 29 feet center to center. The composition of the principal members together with a small-scale illustration showing the character of the details, which conform to the standards described above, are given in Engineering Record, vol. 40, page 498, Oct. 28, 1899. The railroad improvements are also briefly described in Railroad Gazette, vol. 31, page 774, Nov. 10, 1899, and several views are given of the old bridge trusses as well as of the new ones which replaced them. The new structure was completed in 1900.

The views show clearly the connections at the joints, and the relation of the members meeting at the different joints, as well

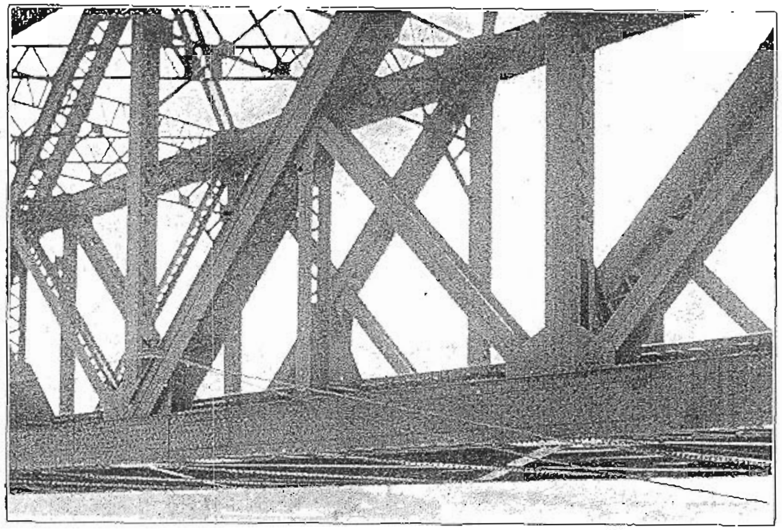


Fig. 147

as the forms of the members and their details. The bridge was designed for very heavy passenger service, and especial attention was given to securing stiffness as well as adequate strength in the truss members and in each span as a whole. These three illustrations, Figs. 147–149, are from photographs taken in January, 1902, and kindly furnished for publication here by W. J. WILGUS, Chief Engineer of the New York Central and Hudson River Railroad.

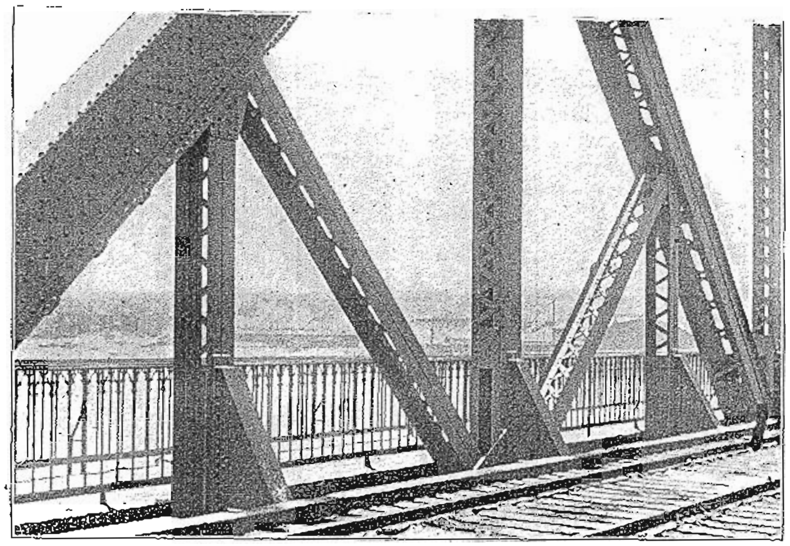


Fig. 148

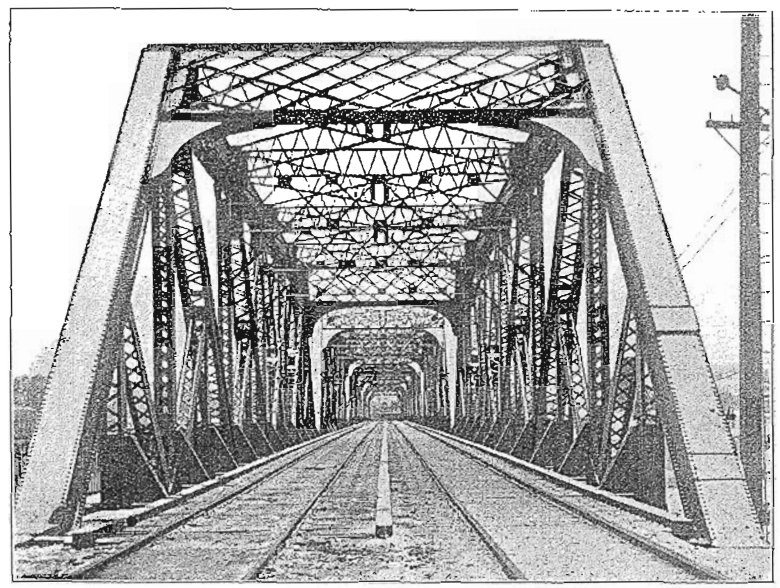
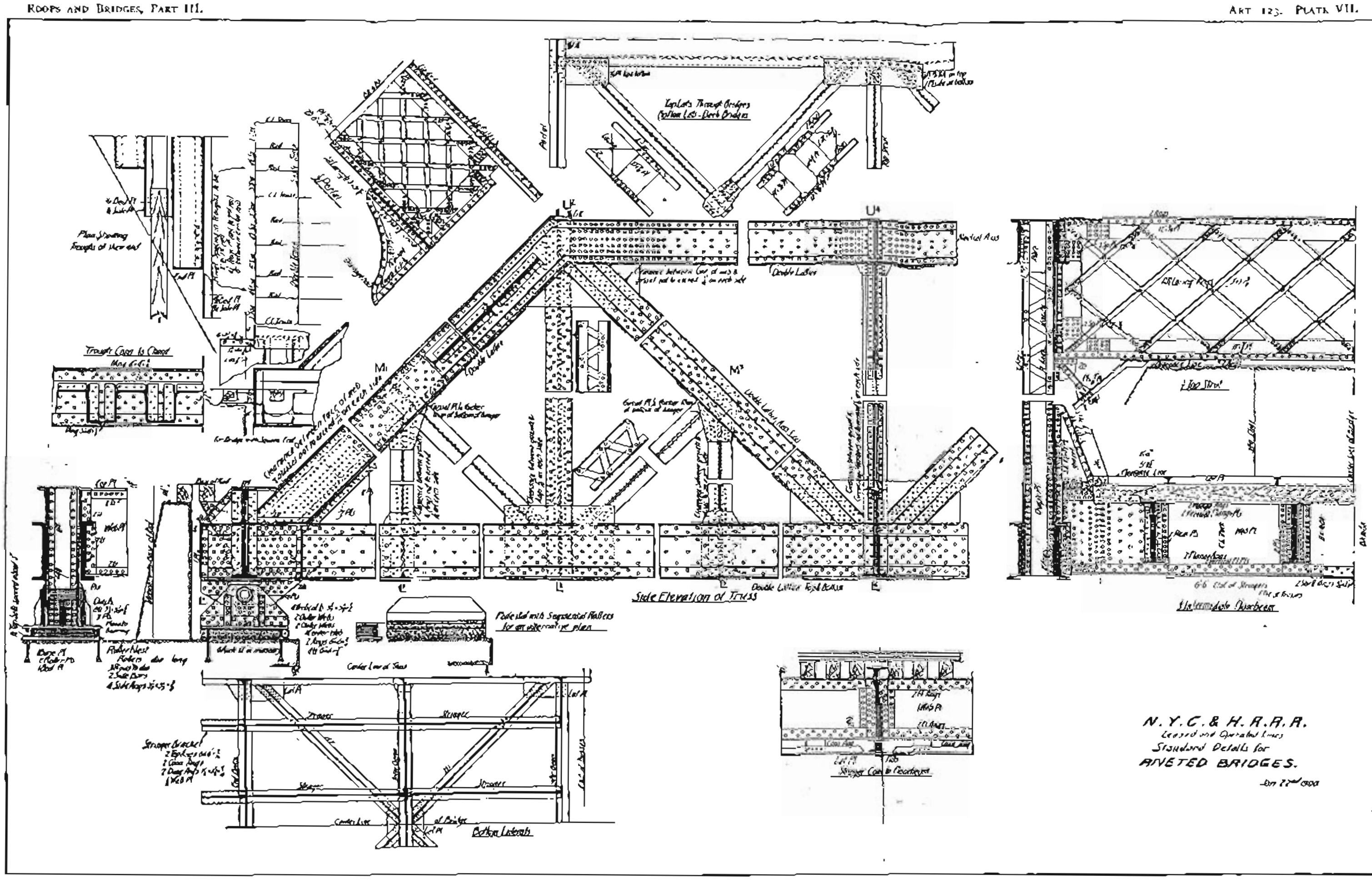


Fig. 149 Portal of Passenger Bridge, Albany, N. Y.


Side Elevation of Truss
Intermediate Floorbeam
Bottom Laterals
Top Strut
Double Lattice
N. Y. C. & H. R. R. R.
Leased and Operated Lines
Standard Details for
RIVETED BRIDGES.

James Garvin

From: "James Garvin" <jgarvin@nhdhr.state.nh.us>
To: "Preservation" <preservation@nhdhr.state.nh.us>
Cc: <ncshpo-listserv@lists.ncshpo.org>; <Mark@arkansasheritage.org>; <Robert.Scoggin@ahtd.state.ar.us>
Sent: Tuesday, July 29, 2003 11:29 AM
Subject: Re: [ncshpo-listserv] FW: Questions on Pratt Thru trusses

I am not aware of standard plans for Pratt through truss spans that had a national circulation, although it is possible that the federal Bureau of Public Roads (BPR) or the American Association of State Highway Officials (AASHO) issued such standard plans for adoption on a nationwide scale. Both the BPR and the AASHO had issued standard bridge specifications by the 1920s and 1930s.

In any case, standard plans for Pratt through truss bridges were available on a more limited scale. There were three main sources of such plans: 1) state highway departments; 2) consulting engineers; and 3) bridge fabricating companies.

First, many state highway departments developed standard Pratt truss plans for statewide adoption during the early 1900s. One such fabrication plan is illustrated in "Spans in Time: A History of Nebraska Bridges" (Nebraska State Historical Society, 1999). According to this book, the 1913 Nebraska legislature passed a law requiring all counties to adopt state standard bridge plans. By this time, the office of the Nebraska state highway engineer had prepared standard designs for some 250 bridge configurations (only some of these would have been Pratt through trusses).

Other states followed the same model of adopting standardized designs, perhaps modeling these designs on federal standard specifications.

Second, consulting bridge engineers prepared standard designs for metal truss bridges, carrying out stress calculations for standard highway loadings and preparing drawings for trusses of varying lengths. The New Hampshire Division of Historical Resources (the SHPO) has copies of the office records of the New Hampshire bridge engineering firm of John W. Storrs, who began his practice in 1906. These include diagrammatic drawings of through Pratt truss bridges in length increments of five feet, calculated for varying live loads.

Third, large bridge fabricators employed their own engineering departments and under certain circumstances produced standard designs for trusses of all types. Predominant among such companies was the American Bridge Company, which had offices in New Jersey, New York, Philadelphia, and Pittsburgh. This company was incorporated in New Jersey in 1900 by J. P. Morgan and Company. The firm quickly purchased twenty-four independent bridge companies that collectively represented fifty percent of the nation's fabricating capacity. In 1901, United States Steel Corporation acquired most of the stock of American Bridge Company, and operated the company as a subsidiary of United States Steel. The company still exists as the largest structural fabricator in the United States.

When northern New England was devastated by floods in 1927, American Bridge Company sent engineers to work within the highway departments of Vermont and New Hampshire. The company helped the highway departments to overcome the catastrophic losses of the flood by developing standard sets of copyrighted plans for bridge trusses of various types. Because these trusses were of standard lengths, some of the replacement bridges were somewhat longer than the original spans, making it necessary to establish new bridge seats back from the toes of the old abutments that were fitted with new trusses.

Again, American Bridge Company adhered to standard federal specifications in this work. Thus, if the federal government did not issue actual fabrication drawings (and they may have), the standard federal specifications certainly resulted in considerable uniformity of all steel truss types, including through Pratt trusses.

For detailed information on pin-connected and riveted joints, the best sources would be any bridge textbook of 1900-1915. The classic text is J. A. L. Waddell's "Bridge Engineering" of 1916 (with later editions). This book is diffuse in its descriptions, so I will mail Mr. Scoggin a few more succinct pages from Merriman and Jacoby's "A Text-Book on Roofs and Bridges, Part III, Bridge Design" of 1902. At this period, the pin connection was still favored by most American engineers, but the text describes the riveted connection, then more favored in Europe.

Enclosed.

Sincerely,

James L. Garvin
Architectural Historian
New Hampshire Division of Historical Resources
P. O. Box 2043--19 Pillsbury Street
Concord, NH 03302-2043
(603) 271-6436

----- Original Message -----
From: "Preservation" <preservation@nhdhr.state.nh.us>
To: "Jim" <jgarvin@nhdhr.state.nh.us>
Sent: Thursday, July 24, 2003 12:16 PM
Subject: Fw: [ncshpo-listserv] FW: Questions on Pratt Thru trusses

> FYI
>
>
>
> ----- Original Message -----
> From: "Mark Christ" <Mark@arkansasheritage.org>
> To: "ncshpo-listserv" <ncshpo-listserv@lists.ncshpo.org>

Scoggin, Robert W.

From: Michael E. Garner [megarner@uark.edu]
Sent: Thursday, July 31, 2003 12:07 PM
To: Scoggin, Robert W.
Cc: 'Katrina Coleman'
Subject: RE: Presentation for AR GIS Conference

Robert,

It is not clear at this time if Katrina will be available to attend the conference. I am listed as a participant and will be available to stand in for Katrina if necessary. I will need a lcdisplay for the presentation.

Thanks,

mike

-----Original Message-----
From: Scoggin, Robert W. [mailto:Robert.Scoggin@ahtd.state.ar.us]
Sent: Thursday, July 31, 2003 11:46 AM
To: 'mgarner@uafortsmith.edu'
Cc: Pearson, Linda
Subject: Presentation for AR GIS Conference

(Katrina Coleman)Your paper "Analysis of an Arboretum using GIS/GPS/City Green" has been accepted for presentation at the upcoming AR GIS Conference
in Eureka Springs, AR. Your presentation is scheduled for Thursday, September 4th at 3:15 - 3:45pm. A short bio is required for use by the moderater of your presentation track. Please respond with equipment needs
for your presentation (laptop, projector, slide projector, overhead, etc.).
If you will be using a presentation constructed in PowerPoint on a Windows
XP Platform we request that you bring your own laptop. Please alert us to
this fact in a response email.

Robert W. Scoggin
Historic Resources Coordinator
Environmental Division
Arkansas State Highway & Transportation Department
10324 Interstate 30
P.O. Box 2261
Little Rock, Arkansas 72203-2261
Office: 501-569-2077 / Fax: 501-569-2009
email: robert.scoggin@ahtd.state.ar.us
www.arkansashighways.com

Scoggin, Robert W.

From: Mark Christ [Mark@arkansasheritage.org]
Sent: Tuesday, July 29, 2003 10:49 AM
To: Scoggin, Robert W.
Subject: FW: [ncshpo-listserv] FW: Questions on Pratt Thru trusses

Bob,

Just in case you didn't get this.

Mark

-----Original Message-----
From: James Garvin [mailto:jgarvin@nhdhr.state.nh.us]
Sent: Tuesday, July 29, 2003 10:30 AM
To: Preservation
Cc: ncshpo-listserv@lists.ncshpo.org; Mark Christ; Robert.Scoggin@ahtd.state.ar.us
Subject: Re: [ncshpo-listserv] FW: Questions on Pratt Thru trusses

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Sincerely,

James L. Garvin
Architectural Historian
New Hampshire Division of Historical Resources
P. O. Box 2043--19 Pillsbury Street
Concord, NH 03302-2043
(603) 271-6436

----- Original Message -----
From: "Preservation" <preservation@nhdhr.state.nh.us>
To: "Jim" <jgarvin@nhdhr.state.nh.us>
Sent: Thursday, July 24, 2003 12:16 PM
Subject: Fw: [ncshpo-listserv] FW: Questions on Pratt Thru trusses

> FYI
>
>
>
> ----- Original Message -----
> From: "Mark Christ" <Mark@arkansasheritage.org>
> To: "ncshpo-listserv" <ncshpo-listserv@lists.ncshpo.org>
> Cc: "Scoggin, Robert W." <Robert.Scoggin@ahtd.state.ar.us>
> Sent: Thursday, July 24, 2003 10:10 AM
> Subject: [ncshpo-listserv] FW: Questions on Pratt Thru trusses
>
>
> I am forwarding the message below on behalf of Bob Scoggin of the
> Arkansas Highway and Transportation Department in hopes that someone