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II

CONS

THE DESIGNING

OF ORDINARY

IRON HIGHWAY BRIDGES.

BY

J. A. L. WADDELL, C. E., B. A. Sc., Ma. E.,

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FIFTH EDITION.

SECOND THOUSAND.

NEW YORK:
JOHN WILEY & SONS,
53 East Tenth Street.
1894.

PREFACE.

THIS work is principally a compilation of the results of investigations made by the author during the last three years, and presented in a number of papers to the various American engineering societies. Several portions of the book. including many of the tables, are new, as this is the first systematic treatment, by the author, of bridges for cities and manufacturing districts; the previous papers having dealt especially with those for country roads. In making this compilation, the author has been governed by no blind adherence to what he has already written, but has made changes wherever they have appeared to be advisable.

One of the chief objects of this work is to reduce the labor of iron highway-bridge designing to a minimum, for which purpose every thing that could be so arranged has been tabulated. Not only are the exact sizes of hip verticals, joists, floor beams, beam hangers, lateral rods and struts, portal rods and struts, vibration rods, intermediate struts, lattice bars, stay plates, etc., given for all practical cases, but also the most economic dimensions of panels and trusses, and dead loads, so exact that by their use all necessity for a second trial is avoided. These tables, it is hoped, will prove useful to those in the actual practice of bridge designing, enabling them to greatly reduce the time required to make diagrams of stresses and sections and estimates of cost. The other tables, although they do not give final results, should also be of service.

The value of the book may appear to some readers to be limited, in that it treats of only the Pratt and Whipple systems; but it must be remembered that at least ninety per cent of all American iron highway-bridges are built on these systems. This fact alone ought to prove conclusively that they are the best type of bridge. Moreover, the author has demonstrated, in a paper entitled "Economy in Struts and Ties," published last year in the "Canadian Magazine of Science," and copied in the "American Engineer," by a method entirely practical, that for economy the web compression members of trusses should be vertical, or nearly so; thus showing, that, of all the ordinary types of truss, the Pratt or Whipple is the best.

Through bridges and pony trusses, both having inclined end posts, have alone been treated at length; for highway deck bridges are uncommon, and inclined end posts not only are more economical than vertical ones, but are also superior to them because they produce tensile stresses in the end panels of the bottom chords, thus adding to the rigidity of the structure.

The work is written for engineers, students, and, to a certain extent, county commissioners. It is not intended, though, to be used by itself as a text-book on bridges, dealing, as it does, with only one general style of truss, but to supplement the books generally used by classes in engineering schools.

It is essentially a treatise upon *bridge designing*, and not one upon *stresses*: nevertheless, it has been found necessary to discuss the latter subject in order to make the work complete. The author would refer those who wish to study concerning stresses to Burr's "Stresses in Bridge and Roof Trusses," Bovey's "Applied Mechanics," and Du Bois' "Strains in Framed Structures."

For county commissioners, Chapters IV., XIV., and XVII., Tables I.-V., XV.-XXV., XXX.-XXXIII., and XXXVIII., and parts of Chapter II., will be found very useful; containing, as they do, directions and data for making estimates of cost, and means of proving whether either designs or finished structures have or have not in many particulars sufficient strength.

Those portions of the "General Specifications" in Chapter II., relating to quality and tests of materials, workmanship, painting, etc., have been taken from standard specifications too numerous to permit of their authority being here quoted: nevertheless, the author must acknowledge that he has received considerable assistance from a paper by Mr. P. F. Brendlinger published in No. 4, vol. iii., of the "Proceedings of the Engineers' Club of Philadelphia," treating of some railroad-bridge specifications prepared by Theodore Cooper, C.E. He wishes to acknowledge also, with many thanks, the valuable aid rendered him by his assistants, Messrs. Y. Nakajima and T. Fukuda, in preparing drawings, and checking tables.

J. A. L. W.

TOKIO, JAPAN, February, 1884.

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PREFACE TO SECOND EDITION.

ALTHOUGH the first edition of this work was issued only three or four months ago, it has had the opportunity for receiving a thorough overhauling by a class of half a dozen students, who were requested to take special pains to point out errors. A few were found; but they are of small importance, being principally typographical or numerical. It is thought that all have been discovered and corrected, but it is possible that others may exist: so, if any reader find any, he will confer a favor upon the author by informing him of the same.

The correctness of the weights of iron in Tables I., II., and III., has received additional confirmation from the same class of students, each student having made a complete design for a bridge. The greatest variation found was less than one-half of one per cent. As the bridges varied in class, span, and width of roadway, one being on a skew, and two having sidewalks, it is fair to conclude that the tables may be relied upon as correct for all cases.

A slight change has been made in Chapter V., near the end, in respect to a statement concerning stresses in the posts of deck bridges; but, for reasons there stated, the formula has not been altered.

In a review of this treatise, by "*The American Engineer*," there was mentioned the fact that double beam hangers are not a satisfactory detail, because of the unequal distribution of the floor-beam load thereon. In the addenda is described a detail which will remove the objection. There are given also in this part of the work, and on Plate VIII., several other details that will be found to be improvements upon those previously described.

About July 1, there will be issued by the University of Tokio, in the form of the usual "Memoir," a treatise of the author upon "A System of Iron Railroad Bridges for Japan," which will be found to contain a number of important matters respecting the designing of railroad-bridges, that have not hitherto received proper attention. The book is not for sale; but there will be about two hundred copies distributed in America among engineers, colleges, public libraries, etc.

The author wishes to express his thanks to the profession for the favorable reception given to his first edition.

J. A. L. W.

TOKIO, JAPAN, May 6, 1885.

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PREFACE TO THIRD EDITION.

It is with considerable regret that the author allows this edition to go to press without making a single change or addition, more especially because his practice in bridge designing has lately been modified. Time will not permit of his re-writing the work for a year or two at least, so it will have to remain as it is for the present. The changes which he would like to introduce in the text are not in principles, but in methods and details of design; American practice having changed somewhat since the book was first written. These points will all be covered by some general specifications, which the author expects to issue in a few months. They will probably be for sale, and advertised in "*Engineering News*."

The principal changes in his methods are the following: —

1st, For long spans it is better to use the Pratt truss, with halved panels, instead of the Whipple truss. In spans exceeding two hundred feet, it is economical to make portions of the top chords inclined.

For short spans it is, in general, well to use rolled beams up to twenty feet, plate girders from twenty to forty feet, triangular riveted girders from forty to sixty-five feet, pin-connected pony trusses from sixty-five to ninety feet, and pin-connected through or deck truss bridges from ninety feet upward.

2d, The batter braces (or, as they are now being generally termed, the inclined end posts) are hinged at hip and pedestal.

3d, Floor beams are riveted to posts as near the lower chords as possible, and the lateral rods are attached to the lower part of the beams. The wooden shims over beams are retained for the purpose of spiking the joists thereto.

4th, Filling-plates under floor beam stiffeners are no longer necessary, as it is now very easy to bend the ends of the angles to fit around the flanges of the beams. It is permissible to stagger intermediate stiffeners. End stiffeners should be figured for the total shear, using an intensity of three tons for bridges of Class A, and three and three-quarter tons for bridges of Classes B and C.

5th, Upper lateral and portal struts are made of four angle-irons, with a single system of lacing-bars. These struts are rigidly connected to top chords or inclined end posts by riveting.

6th, End lower lateral struts can, in general, be made of single angle-iron of large size. It is preferable to use one of these struts at each end of every span. They can be riveted to the pedestals by means of a connecting-plate at each end.

7th, Lacing is used everywhere instead of latticing.

8th, Rivets are figured for shear and bearing, not for bending.

9th, In many cases flattened heads may be used to advantage, instead of countersinking the rivets, the thickness of the heads being three-eighths of an inch.

10th, Lateral rods are attached at each end to top chord by means of three short pieces of angle-iron, through one of which the rod passes. The adjustment is made by a nut at each end, which bears against the last mentioned angle-iron.

11th, In top-chord splicing, reliance is placed upon the abutting ends of the chord sections. The splice is made about fifteen or eighteen inches from the panel point on the side towards the nearer pier, and there are only two vertical rows of rivets in the splice plate on each side of the joint The shop practice of first-class bridge companies has improved so much during the past five years, that it is now legitimate to rely upon abutting ends for this detail. Nevertheless, the author would prefer to hinge the top chord at each panel point, for in this case there can be no doubt as to how the stresses travel. The same cannot be said for any other style of connection.

12th, When a bridge is sufficiently heavy, it is well to build the top chords and inclined end posts of plates and angles, using small, light angles above, and large, heavy angles below, so as to bring the centre of gravity of the section to the middle of the vertical plate.

13th, For hip verticals it is preferable to use a strut of two channels similar to the posts, but not so efficiently laced, as the member acts only in tension. Channels provide greater rigidity than do eye-bars.

14th, In proportioning compression members, Thacher's formula is used instead of that of the late C. Shaler Smith. It is the following: —

$$p = \begin{cases} 9{,}000 - 30\dfrac{l}{r} \text{ for } \square \ \square \\[2ex] 9{,}000 - 35\dfrac{l}{r} \text{ for } \square \ \bigcirc \\[2ex] 9{,}000 - 40\dfrac{l}{r} \text{ for } \bigcirc \ \bigcirc \end{cases}$$

where p is the intensity of working stress in pounds, l the unsupported length of strut in inches, and r the radius of gyration of section in inches. The above formula is for bridges of Class A. To use it for Classes B and C, multiply p by $\frac{5}{4}$; and to use it for lateral struts, multiply p by $\frac{3}{2}$. In the latter case, however, considerations of rigidity generally necessitate the use of greater sectional area than the stress would call for.

15th, In order to be in accordance with modern practice, floor beams should be proportioned by neglecting the resistance of the web to bending, and using an intensity of five tons for finding the *net* area of the bottom flange in bridges of Class A, and six tons for bridges of Classes B and C, then making the upper flange of the same section as the lower, taking care, however, to have the ratio of unsupported length of beam to width of flange not greater than thirty.

In the author's opinion this method is not so rational as the one which he used formerly, but it is more easily applied and gives about the same results.

Strictly speaking, the web of a built beam does aid the flanges to resist bending; but, in truth, the designing of built beams and girders pertains less to science than to rule of thumb.

16th, Bent eyes on rods are no longer allowed.

17th, In every thing relating to quality of material, workmanship, inspection, and tests, the Manufacturers' Standard Specifications are to be followed.

The author trusts that the preceding remarks, together with his new specifications, will keep his book from becoming antiquated until he can find time for re-writing the whole treatise.

J. A. L. W.

KANSAS CITY, MO., July 16, 1887.

PREFACE TO FOURTH EDITION.

Again with great regret does the author permit this work to reach another edition without receiving a thorough revision, amounting, in fact, to a complete re-writing of the book. The past year has been such a busy one, that it was impossible for him to spare the necessary time.

The new specifications promised in the last preface have been issued, and are now on sale by Mr. A. C. Stites, Walworth Building, Kansas City, Mo., the price being twenty-five cents per copy.

The author would advise that these be used in connection with this book, and that, where they conflict, the new specifications be followed. It will be seen that the latter contain considerably more than mere specifications, being a systematic attack upon the present methods of designing, letting, and building highway bridges. The pamphlet is being well advertised and distributed among county commissioners; and the author is sparing neither time, trouble, nor expense to accomplish the reform which he deems so necessary.

The re-writing of this treatise has already been begun; but there is no telling when it will be completed, for the author's spare time is very limited. He hopes, however, that it will be be finished before a fifth edition becomes necessary.

J. A. L. W.

Kansas City, Mo., April 11, 1888.

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THE DESIGNING

OF

ORDINARY IRON HIGHWAY-BRIDGES.

CHAPTER I.

INTRODUCTION.

THAT many bridge designers will have fault to find with the contents of this book goes without saying, but it will be found that the principal objections will come from those who design the lightest and poorest structures. The weights of iron in the bridges here treated are probably from twenty to fifty per cent greater than those in the bridges ordinarily built; but it is to be remembered that most American iron highway-bridges are not what they ought to be, and that the author has endeavored to design structures first-class in every respect.

The principal differences between these bridges and those ordinarily built are the stiffening of the end panels wherever necessary; the use of C. Shaler Smith's formula, involving, as it generally does, an increase of sectional area; the peculiar lower lateral system, which avoids using the floor beams as lateral struts; the large lateral rods employed; the allowance for initial tension in all adjustable rods; the variation of the intensity of working-stress for main diagonals; the assumption that all stresses at joints in top chords and at upper ends of posts are carried by the connecting plates and their rivets, no reliance being placed upon the abutting ends of channels; the limiting sizes of the sections of iron employed; and the unusual strength of the upper lateral and portal struts, especially when no vertical sway bracing is used.

On the other hand, those who wish to proportion bridges by the latest theories may object to the employment of C. Shaler Smith's formula (which fails to take into account the radius of gyration of the section) and to the non-employment of the results of Wöhler's and Weyrauch's recent investigations concerning intensities of working-stresses. To the first objection, the author would reply, that designers of ordinary highway-bridges cannot afford the time to spend at least fifteen minutes in obtaining the theoretically best intensity of working-stress for each strut, but must have for this purpose tables which will give the intensities without calculation: besides, the best theoretical intensity is merely approximate. To the second objection, he would reply, that the results of the investigations referred to have not yet been generally adopted, and that the variation of intensity of working-stress for the main diagonals used in this treatise is, in his opinion, for ordinary highway-bridge designing, a sufficient concession to the general correctness of the theory of those writers.

The units used throughout this work are as follows: the American ton (two thousand pounds) for the units of weight and stress, the foot for the unit of length, and the square inch for the unit of area.

It is presupposed that the reader, if he intend to design, or even study the designing of, iron bridges, has procured a copy of Carnegie's "Pocket-Companion," the most useful little book of its kind for bridge builders that has ever been printed: so the tables therein are here referred to, instead of being reproduced.

The sections of iron employed are those rolled at the Union Iron Mills, for the reason that not only is there more iron rolled in these mills than anywhere else in America, but the properties of the sections are tabulated in a much more convenient form than are those of any other mills. The author intended to prepare a table of channels rolled by the New-Jersey Steel and Iron Company of Trenton, N.J., similar to Table XXVIII.; but the information in that company's pocket-book is not complete enough to enable him to tabulate the thicknesses of the webs.

Any one intending to design bridges according to the method herein proposed should thoroughly acquaint himself with the numbers and uses of the different tables, so as to turn to the one required without delay. He should also become posted on the contents of Chapters VII.–XIII., so as not to have to refer to that part of the book, turning to Chapter II. to refresh his memory concerning intensities of working-stresses, limitations, etc., while designing each main member and detail. After a little practice, one will become acquainted with the method, when it will be necessary to refer to the tables only.

If a designer be in doubt about how to proportion any main member or detail, he can at once find out the method by looking in the Index, under the head "Proportioning," where he will see the numbers of the pages on which the proportioning of this member or detail is treated.

Any intelligent man who is not an engineer can make an approximate estimate of what a first-class iron highway-bridge ought to cost, by finding the required weight of iron from one of Tables I., II., or III., modifying it, if necessary, in the manner explained in Chapter IV.; and the required amount of lumber, from one of Tables XV., XVI., XVII., or XVIII., ascertaining the prices of iron per pound, and lumber per thousand feet, delivered at the nearest railway-station or seaport, and filling out the form for an estimate of cost given on p. 116. By referring to the Index, under the heading "Cost," he can ascertain where approximate data for all bridge-building expenses can be found.

No special treatment is here given for skew bridges; for none is needed, the methods for designing them being precisely the same as those for other bridges. On account of the obliquity, the working drawings for the lateral bracing and sway bracing are a little more complicated. Whenever it is convenient to do so, the panel lengths should be arranged so that the shoe of one truss comes opposite to the first panel point of the other truss, in order that the floor beams may be at right angles to the planes of the trusses, both for economical reasons, and to avoid using single beam hangers. This arrangement can often be made by shortening the panel length a little, and, if it be allowable, slightly changing the angle of the skew. Even if it be impracticable to make this arrangement, it is usually better, in skew bridges, to advance the ends of the floor-beams at one side of the bridge, by one or even two panel lengths, if, by so doing, the floor beams be shortened.

CHAPTER II.

GENERAL SPECIFICATIONS.

THIS chapter, at first thought, may appear out of place; for it is really a *résumé* of the whole subject of iron highway-bridge designing. It is placed here in order to be of easy reference.

Students, and those unacquainted with bridge designing, are advised to omit these specifications, and to return to them after having read through Chapter XIII.

Classification. — Highway-bridges may be divided into three classes; viz., Class A, which includes those for cities and their suburbs that are subjected to the *continued* application of heavy loads; Class B, which includes those for cities and their suburbs, or manufacturing districts, that are subjected to the *occasional* application of heavy loads; and Class C, which includes those for country roads, where the traffic is lighter.

Live Load. — The live loads for bridges of the different classes are to be taken from the following table: —

SPAN IN FEET.	MOVING LOAD PER SQUARE FOOT OF FLOOR.	
	Classes A and B.	Class C.
0 to 50	100 pounds	80 pounds
50 to 150	90 pounds	80 pounds
150 to 200	80 pounds	70 pounds
200 to 300	70 pounds	60 pounds
300 to 400	60 pounds	50 pounds

The live loads for joists, floor beams, beam hangers, and hip verticals, are to be one hundred (100) pounds per square foot of floor for bridges of Classes A and B, and eighty (80) pounds per square foot for bridges of Class C, irrespective of the length of bridge.

Dead Load. — The dead load is to include the weight of all the iron and wood in the structure, excepting that of those portions resting directly on the abutments, whose weights do not affect the stresses in the trusses; also, if necessary, an allowance for snow, mud, paving, or any other unusual fixed load that may ever be placed on the bridge.

Pine-lumber is assumed to weigh two and a half ($2\frac{1}{2}$) pounds per foot, board measure; and oak-lumber, four and a third ($4\frac{1}{3}$) pounds per foot, board measure.

Should, in any bridge of or below one hundred (100) feet span, the calculated dead load differ more than eight (8) per cent from that assumed, or in any bridge from one hundred (100) to two hundred (200) feet span, more than six (6) per cent, or in any bridge exceeding two hundred (200) feet span, more than four (4) per cent, the calculations of stresses, etc., are to be made over with a new assumed dead load.

Wind Pressure. — The wind pressure per square foot is to be assumed as forty (40) pounds for spans of one hundred (100) feet and under, thirty-five (35) pounds for spans over one hundred (100) and not greater than one hundred and fifty (150) feet, and thirty (30) pounds for all greater spans.

For bridges in unusually exposed situations, these pressures are to be increased by ten (10) pounds per square foot.

The total area opposed to the wind is to be determined by adding together the area of the vertical projection of the floor and joists, and twice the area of the vertical projection of the windward truss, hand rail, hub plank, guard rail, and ends of floor-beams.

Length of Span. — The length of span is to be understood as the distance between centres of end pins for trusses, and between centres of bearing-plates for all beams and girders.

Limiting Lengths of Span for Different Clear Roadways. — The greatest lengths of span for the different clear roadways are to be one hundred and forty (140) feet for twelve (12) foot roadways, one hundred and ninety (190) feet for fourteen (14) foot roadways, two hundred and sixty (260) feet for sixteen

(16) foot roadways, and three hundred and fifty (350) feet for eighteen (18) foot roadways. By clear roadway is meant the distance between the innermost portions of the opposite trusses, measured in a direction perpendicular to their planes.

Limit of Clear Headway. — The least allowable clear headway is to be fourteen (14) feet, unless some local consideration cause it to be increased. By clear headway is meant the vertical distance between the upper face of the flooring and the lowest part of the portal or overhead bracing.

Styles of Bridges for Different Spans. — Spans of and below twenty (20) feet are to consist of rolled beams; spans from twenty (20) to forty (40) feet, of riveted plate girders or trussed beams; spans from forty (40) to sixty-five (65) feet, of stiffened pony trusses or stiffened deck bridges, unless the weight of bridge be great enough to admit of the use of eye bars for the bottom chords; and spans above sixty-five (65) feet, of pin-connected through or deck truss bridges.

Limiting Depth of Pony Trusses. — The greatest allowable depth, measured from centre to centre of chords, for pony trusses without exterior side bracing, is to be six (6) feet; and that for pony trusses with exterior side bracing, nine (9) feet. For bridges with sidewalks, in which it is inconvenient to use exterior side braces, the depth may be increased to eight (8) feet, provided that the width of the top chord plate be double the depth of the top chord channels, and that the channels composing the posts and hip verticals be splayed outwardly so as to be separated in the clear at their feet by at least twenty-four (24) inches.

Limiting Slope for Batter Braces of Pony Trusses. — The least allowable slope for batter braces of pony trusses is to be two and a quarter ($2\frac{1}{4}$) horizontal to one (1) vertical.

Side Braces. — The least allowable batter for side braces in pony-truss bridges is to be five (5) inches to the foot, and all side braces are to be made to resist both tension and compression. In no case is a side brace to have less strength than that of a $2\frac{1}{2}'' \times 2\frac{1}{2}''$ 5-lb. angle-iron.

Limiting Length of Span for Double Intersection Bridges. — The least allowable length of span for double intersection bridges is to be taken from the following table: —

CLEAR ROADWAY.	Limiting Lengths.		
	Class A.	Class B.	Class C.
14′	165′	175′	180′
16′	155′	165′	170′
18′	150′	160′	165′
20′	145′	155′	160′
22′	140′	150′	155′
24′	140′	145′	150′

Limiting Sizes of Sections. — No rods less than three-quarters ($\frac{3}{4}$) of an inch in diameter are to be used in a bridge. No channels less than five (5) inches in depth are to be used for upper chords, batter braces, or posts, or less than four (4) inches in depth for other members. No flat bars less than one-half ($\frac{1}{2}$) inch thick, or one and a half ($1\frac{1}{2}$) inches wide, are to be used for diagonals or chord bars; nor any iron less than one-quarter ($\frac{1}{4}$) of an inch thick anywhere in a bridge, excepting for filling plates.

Expansion. — All spans are to be provided with some means of expanding and contracting longitudinally, with a variation in temperature of one hundred and fifty (150) degrees Fahrenheit. Spans of over seventy-five (75) feet are to have at one end nests of turned wrought-iron friction rollers, running between planed surfaces.

Anchorage. — At least one end of every bridge must be anchored to the foundations. If the overturning moment of the greatest assumed wind pressure be more than half the resisting moment of the weight of the bridge, the latter must be anchored at the roller end also, but in such a manner as not to interfere with the expansion.

Sliding. — At the roller end of a bridge, if the frictional resistance to the sliding of the shoe in the direction of the axes of the rollers be not more than double the tendency to slide produced by the wind pressure, a resistance equal to the difference between this tendency and the frictional resistance with a factor of safety of four (4) must be provided.

Continuous Spans. — Except in the case of swing bridges or cantilevers, consecutive spans are not to be made continuous over the points of support.

Camber. — The cambers for bridges of the different spans are to be taken from the following table: —

Span in Feet.	Camber in Inches.	Span in Feet.	Camber in Inches.
40–60	1.0	180–220	3.5
60–80	1.5	220–250	4.0
80–100	2.0	250–280	4.5
100–140	2.5	280–300	5.0
140–180	3.0		

Vertical Sway Bracing. — In all deck bridges, and in all through bridges where the depth from centre to centre of chords is twenty-four (24) feet or over, vertical sway bracing is to be used.

Portal and Lateral Struts. — Portal and lateral struts are to be proportioned to resist the compression produced by the wind pressure and the initial tensions in all the rods meeting at the end of the strut. If the strut be also subjected to bending, then to the area necessary to resist compression must be added sufficient area to resist the bending; the intensity of working bending-stress being taken equal to six (6) tons.

Effect of Wind on Posts and Batter Braces. — But the effect of the wind on the posts and batter braces is not to be considered to occur when the bridge is fully loaded: so, unless the stresses produced thereby exceed the product of the live load stresses by the ratio of seven and a half (7.5) to the intensity of working tensile stress for the bottom chord, the effect of the wind on these members may be neglected.

Effect of Wind Pressure on Bottom Chord Tension. — For the same reason, the sectional area of the bottom chord need not be increased to resist the tension caused by the wind, unless the latter exceed the product of the live load stress by the ratio of seven and a half (7.5) to the intensity of working tensile stress for chord bars.

Initial Tension. — To allow for the stresses caused in adjustable members by the screwing up of the turn buckles or sleeve nuts, the stress in each such member is to be increased by the amount given in the following table: —

Diameter of Rod.	Initial Tension.	Diameter of Rod.	Initial Tension.
$\frac{3}{4}''$	0.50 ton	$1\frac{5}{8}''$	2.25 tons
$\frac{7}{8}''$	0.75 ton	$1\frac{3}{4}''$	2.50 tons
$1''$	1.00 ton	$1\frac{7}{8}''$	2.75 tons
$1\frac{1}{8}''$	1.25 ton	$2''$	3.00 tons
$1\frac{1}{4}''$	1.50 ton	$2\frac{1}{8}''$	3.25 tons
$1\frac{3}{8}''$	1.75 ton	$2\frac{1}{4}''$	3.50 tons
$1\frac{1}{2}''$	2.00 tons	$2\frac{3}{8}''$	3.75 tons

Square or flat bars are to receive the allowance for round rods of equal sectional area.

Connection for Lateral Systems. — Whenever it be possible, the lateral rods of both upper and lower systems are to be connected directly to the chord pins. But, if the rods exceed one and three-quarter ($1\frac{3}{4}$) inches in diameter, bent eyes are not to be employed.

Lower lateral rods are not to be attached to the floor beams. To make them clear the joists, wooden lateral struts resting on the floor beams, and having wrought-iron jaws at their ends attached to the chord pins, are to be employed for the joists to rest upon.

These wooden struts are to be bolted about every two feet through the upper flange of the floor beam by five-eighth ($\frac{5}{8}$) inch bolts, care being taken to stagger the bolt holes, and to avoid placing a bolt at the middle of the beam.

Should the sizes of the lateral rods be such as to prevent the use of bent eyes, pins dropped vertically through the jaws are to be employed.

Stresses in End Lower Lateral Struts. — In figuring the stress in a lower lateral strut at the roller end of a bridge, the stress caused by the wind pressure is to be added to the transverse component of the initial tension in the end lateral rod, and from the sum is to be subtracted the product of the pressure on the windward shoe, when the bridge is empty and subjected to the greatest wind pressure, by the co-efficient of iron upon iron, which is about 0.25 for this case.

Stiffened End Panels. — If, in the end panel of a bridge, the longitudinal component of the greatest allowable working-stress (including initial tension) in the lower lateral rod exceed the tension in the lower chord of that panel, caused by the dead load alone, when the bridge is subjected to the greatest wind pressure, the bottom chord of that panel must be made to resist both tension and the excess of compression. Where two channels are employed for the lower chord section, the effective area of the webs alone must be counted on to resist tension. Where the stiffening is obtained by trussing the inner chord bars, the intensities of working tensile stress to be employed for the net section of those bars are four (4) tons for bridges of Class A, and five (5) tons for those of Classes B and C.

Top-Chord and Batter-Brace Sections. — The top chords and batter braces shall consist of two channels, with a plate above, and latticing or lacing below. The top plate must be of the same section, and the chord channels of the same depth, from end to end of span; the increase in chord section towards the middle being obtained by thickening the webs of the channels.

Post Sections. — Posts are to consist of two channels, with latticing or lacing on each side. The upper ends may be either rigidly attached to the chords, or may be hinged on the pins; preference being given to the latter method.

Portal and Upper Lateral Strut Sections. — Portal struts and upper lateral struts are to be formed of two channels, latticed or laced, and rigidly attached at their ends to the batter braces or chords.

Working Tensile Stresses. — Except for the case of trussed bars, mentioned under the divisions "Stiffened End Panels" and "Stiffened Hip Verticals," the intensities of working-stresses for iron in tension in the various members are to be as given in the following table: —

MEMBERS.	Intensities of Working-Stress.	
	Class A.	Classes B and C.
Lower chord bars and end main diagonals	5.00 tons	6.25 tons
Middle panel diagonals, counters, and hip verticals	4.00 tons	5.00 tons
Flanges of rolled beams	5.00 tons	6.00 tons
Flanges of built beams (net section)	4.00 tons	5.00 tons
Lateral and vibration rods	7.50 tons	7.50 tons
Beam-hangers	3.00 tons	4.00 tons

The intensities of working-stress for main diagonals intermediate between the counters or middle panel diagonals and the end diagonals lie between four (4) and five (5) tons for bridges of Class A, and between five (5) and six and a quarter ($6\frac{1}{4}$) tons for those of Classes B and C; the amounts being directly interpolated according to the position of the panel.

Working Compressive Stresses. — For struts composed of two channels with plates, or lacing, or latticing, the following formulas are to be used in finding the intensities of working compressive stresses.

For chords, batter braces, and posts in bridges of Class A,

$$p = \frac{\dfrac{f}{1 + \dfrac{H^2}{C}}}{4 + \dfrac{H}{20}}$$

and for all other cases,

$$p = \frac{\dfrac{f}{1 + \dfrac{H^2}{C}}}{4 + \dfrac{H}{30}},$$

p being the intensity of working-stress, and

$$H = \frac{\text{length of strut}}{\text{least diameter of strut}},$$

$$f = \begin{cases} 19.25 \text{ for two fixed ends} \\ 19.25 \text{ for one fixed end and one hinged end} \\ 18.90 \text{ for two hinged ends,} \end{cases}$$

and

$$C = \begin{cases} 5820 \text{ for two fixed ends.} \\ 3000 \text{ for one fixed end and one hinged end} \\ 1900 \text{ for two hinged ends.} \end{cases}$$

Where I-beams are employed for intermediate struts or end lower lateral struts, the working-stresses are to be taken from Table XI. For the flanges of rolled beams, the intensities of working compressive stress are to be taken equal to five (5) tons for bridges of Class A, and six (6) tons for bridges of Classes B and C. For the flanges of built beams, the corresponding intensities are to be four (4) and five (5) tons respectively on the gross section.

Working Bending-Stresses. — The intensities of working bending-stress on pins are to be seven and a half ($7\frac{1}{2}$) tons for bridges of Class A, and nine and three-eighths ($9\frac{3}{8}$) tons for those of Classes B and C. For pins belonging wholly to the lateral systems of bridges of either class, the intensity of working bending-stress may be taken equal to eleven and a quarter ($11\frac{1}{4}$) tons. The intensities of working bending-stress for rivets are to be seven and a half ($7\frac{1}{2}$) tons for bridges of Class A, and nine and three-eighths ($9\frac{3}{8}$) tons for those of Classes B and C. The latter intensity is also to be used for the lateral systems of bridges of Class A.

Where steel pins are employed, the intensity of working bending-stress must not be taken greater than twelve (12) tons for bridges of Class A, or fifteen (15) tons for those of Classes B and C, unless special experiments on the steel used show an ultimate bending resistance greater than sixty (60) tons per square inch; in which case a factor of five (5) may be used for bridges of Class A, and a factor of four (4) for those of Classes B and C. As before stated, the intensity of working bending-stress for channels in portal and lateral struts is to be six (6) tons.

Working Bearing-Stresses. — The intensities of working bearing-stress for pins and rivets, measured upon the projection of the semi-intrados upon a diametral plane, are to be six (6) tons for bridges of Class A, and seven and a half ($7\frac{1}{2}$) tons for those of Classes B and C. For pins and rivets belonging wholly to the lateral system of a bridge of any class, the intensity is to be taken equal to seven and a half ($7\frac{1}{2}$) tons.

Sizes of Upper Lateral Rods. — In bridges of less than two hundred (200) feet span, the stresses in the upper lateral system in through bridges, or the lower lateral system in deck bridges, usually call for sections of rods which are practically too small: therefore the inferior limits of the diameters of these rods in such cases are to be taken from Table XXV.

Stiffened Hip Verticals. — Hip verticals in three or four panel pony trusses are to be stiffened so as to resist compression. If the section employed consists of two channels, the net section of the webs alone is to be relied on to resist tension. If it consists of two flat bars trussed, the intensities of working tensile stress on the net section are to be reduced to three (3) tons for bridges of Class A, and to four (4) tons for those of Classes B and C.

Trussing. — Trussing is to be used only in the posts of pony trusses, where there is a great excess of strength, in the hip verticals of pony trusses, and in stiffening lower chord bars.

Upset Ends. — Middle panel diagonals, counters, lateral rods, vibration rods, and all other adjustable rods, excepting beam hangers that have an excess of section, are to have their ends enlarged for the screw threads, so that the diameter at the bottom of the thread shall be one-sixteenth ($\frac{1}{16}$) of an inch greater than that of the body of the rod, square or flat bars being figured as if of equivalent round section.

Threads. — All threads, except those on the ends of pins, must be of the United-States standard.

Minimum Dimensions of Chord and Batter-Brace Plates. — The minimum dimensions for the top plate in top chords and batter braces are to be taken from the following table. For five (5) and six (6) inch channels, the thickness does not increase with the width. For seven (7) inch channels, the thickness should be increased to five-sixteenths ($\frac{5}{16}$) of an inch, should the width exceed fifteen (15) inches. For the other channels, should the width of plate exceed that given in the table by from forty (40) to seventy (70) per cent, the thickness must be increased by one-sixteenth ($\frac{1}{16}$) of an inch, while, if it exceed by more than seventy (70) per cent, the thickness must be increased by one-eighth ($\frac{1}{8}$) of an inch.

DEPTH OF CHANNEL.	MINIMUM THICKNESS.	MINIMUM WIDTH.	DEPTH OF CHANNEL.	MINIMUM THICKNESS.	MINIMUM WIDTH.
5″	$\frac{1}{4}$″	7″	9″	$\frac{5}{16}$″	11$\frac{1}{2}$″
6″	$\frac{1}{4}$″	8″	10″	$\frac{5}{16}$″	12$\frac{1}{2}$″
7″	$\frac{1}{4}$″	9″	12″	$\frac{3}{8}$″	15″
8″	$\frac{1}{4}$″	10″	15″	$\frac{3}{8}$″	19″

Stay Plates. — Sizes of stay plates are to be taken from Tables XXXII. and XXXIII. Stay plates on latticed or laced compression members are to be placed as near the pin holes as possible. Latticing or lacing must never be used without stay plates at the ends.

Latticing and Lacing Bars. — The sizes for lattice bars and lacing bars are to be taken from Tables XXX. and XXXI. The distance from the back of an end rivet hole to the end of the bar must not be less than one-half the width of the bar. The ends of the bars are to be semicircular, except when there are two rivets at each end, in which case they should be cut parallel to the channels.

Inclination of Latticing and Lacing Bars. — Lattice bars shall make with each other, as nearly as circumstances will permit, angles of ninety (90) degrees; and lacing bars, angles of sixty (60) degrees.

Diameters of Rivets for Different Channels. — For attaching plates and lattice or lacing bars to the flanges of channels, the least diameters of the rivets to be used are to be taken from the following table; and the greatest diameters must not exceed those there given by more than one-eighth ($\frac{1}{8}$) of an inch.

Depth of channel . .	4″	5″	6″	7″	8″	9″	10″	12″	15″
Diameter of rivets . .	$\frac{1}{2}$″	$\frac{1}{2}$″	$\frac{1}{2}$″	$\frac{9}{16}$″	$\frac{5}{8}$″	$\frac{11}{16}$″	$\frac{11}{16}$″	$\frac{3}{4}$″	$\frac{13}{16}$″

Splice Plates. — The length of a splice plate is to be determined by the number of rivets necessary to transfer the stress from one main member to the other. The sum of the working bearing resistances of all the rivets on either side of the joint must not be less than the stress in the main member upon that side. The rivets must also be figured for bending. When practicable, a splice plate must be placed on each side of every member where a joint occurs.

The transmission of compressive stresses shall be considered as entirely through the medium of the rivets and connecting plates, and these must be proportioned accordingly; so that the area of the two splice plates connecting two channel bars must be at least equal to that of the larger channel bar.

Re-enforcing Plates. — Simple re-enforcing plates, or plates riveted to webs at pin holes in order to compensate for strength lost there, or to provide additional bearing for the pins, must have as many rivets to attach them to the webs as will give bearing and bending resistances for the same, equivalent to at least the greatest stresses that can come upon the re-enforcing plates.

Cover Plates. — Cover plates for top chords or batter braces are to have the same section as the chord or batter-brace plate, the joints in which they cover, and enough rivets on each side of the joint to take up the greatest stress that could ever come upon the said chord or batter-brace plate.

Extension Plates. — Extension plates on the end of a strut, for the purpose of hinging the latter, are to have at least twice the sectional area of the strut from the pin-hole to the nearest edge of the stay plate; and the thickness must be great enough to give sufficient bearing upon the pin. The length of the extension plates is to be such as to allow of the use of a sufficient number of rivets to provide proper bearing and bending resistances for the same.

Shoe Plates, Roller Plates, and Bed Plates. — No shoe plate is to have a less thickness than three-quarters ($\frac{3}{4}$) of an inch, and no roller plate or bed plate a less thickness than seven-eighths ($\frac{7}{8}$) of an inch. When nine (9) or ten (10) inch channels are used for the batter braces, the thickness of the shoe plates is to be seven-eighths ($\frac{7}{8}$) of an inch. When twelve (12) inch channels are used for the batter braces, the thickness of the shoe plates, roller plates, and bed plates, is to be one (1) inch; and, when fifteen (15) inch channels are used, it is to be one and an eighth ($1\frac{1}{8}$) inches.

Bed plates must be of such dimensions that the greatest pressure on the masonry shall not exceed two hundred (200) pounds per square inch.

Every bearing upon masonry must be provided with either a bed plate or a roller plate, well fastened to the masonry by bolts not less than one (1) inch in diameter; but, if the shoe plate be sufficiently large, it may act as a bed plate at the fixed end of the span.

Beam-Hanger Plates. — Beam-hanger plates are never to be made less than three-quarters ($\frac{3}{4}$) of an inch thick, and their areas are to be such that the hanger nuts will always have a full bearing thereon. The necessary thickness for a beam-hanger plate is to be determined by considering it as a beam uniformly loaded by the whole weight that comes on the hangers; the length of said beam being the distance between the centres of the holes through which pass the ends of one hanger, and its width being the extreme dimension of the plate, measured parallel to the floor beam. The intensity of working-stress for bending in the plate is to be taken equal to that used in proportioning the floor beam.

Riveting. — In riveted work, all joints are to be squarely and truly dressed, and the rivet holes must be accurately spaced.

No rivets with crooked heads, or heads not formed accurately on the shank, or rivets which are loose either in the rivet holes or under the shoulders, will be allowed in a bridge.

Rivet holes in top-chord plates and batter-brace plates shall be spaced as nearly as practicable two and a half ($2\frac{1}{2}$) inches centre to centre near the panel points, and four (4) inches centre to centre elsewhere.

No rivet-hole centre shall be less than one and a half ($1\frac{1}{2}$) diameters from the edge of a plate: whenever practicable, this distance is to be increased to two (2) diameters.

The diameter of a hole shall never exceed that of the rivet by more than one-sixteenth ($\frac{1}{16}$) of an inch.

When two or more thicknesses of plate are riveted together in compression members, the outer row of rivets shall not be more than three (3) diameters from the side edge of the plate.

Rivet holes must never be spaced less than two and a half ($2\frac{1}{2}$) diameters from centre to centre: it is preferable that this distance be increased to three (3) diameters when so doing will not cause inconvenience in designing.

All the rivet holes of the respective parts of any structure must be made to exactly coincide, either by drilling the holes full size through the connecting portions, after being put together, or by sub-punching the pieces separately, and afterwards reaming the combined rivet holes to proper size. In all cases the burrs must be removed by slightly countersinking the edges of the holes.

All rivets in splice or tension joints are to be systematically arranged, so that each half of a tension member or splice plate will have the same uncut area on each side of its centre line.

No rivet, excepting those in shoe plates, is to have a less diameter than the thickness of the thickest plate through which it passes, nor in any case less than half ($\frac{1}{2}$) an inch.

Built Members. — The several pieces forming one built member must fit closely together, and when riveted shall be free from twists, bends, or open joints.

Use of Bolts. — The use of bolts instead of rivets is to be avoided whenever practicable.

Lateral Bracing for Plate Girders. — When a span consisting of plate girders is over fifteen (15) feet in length, the adjacent girders are to be braced to each other by diagonal angle irons attached to or near the lower flanges. There are to be, also, light bracing-frames at each end between adjacent girders, so placed as not to interfere with the expansion. When the span is over twenty-five (25) feet in length, the upper flanges of adjacent girders are also to be connected by diagonal angle-iron braces.

In every case the joists are to be dapped at least an inch onto the girders, and every third joist resting on any girder is to be bolted through the upper flange.

*Formula for Built Floor Beams and Plate Girders.** — The tension flanges of built floor beams and plate girders are to be proportioned by the formula

$$A = \frac{WL}{8DT} - \frac{A'}{6} + A'',$$

where A is the area of the flange, A' that of the web, A'' that lost from the flange by a rivet hole, W the uniformly distributed load in tons, L the length of the beam in feet between centres of supports, D the depth in feet between centres of gravity of flanges, and T the intensity of working tensile stress in tons. The same formula will apply to the compression flanges by making A'' equal to zero.

Stiffeners. — Built floor beams and plate girders must be stiffened by four (4) angle irons at each support, and by two (2) angle-irons at several intermediate points; the distance apart of the stiffeners being made no greater than twice the depth of the beam when the ratio of thickness of web to depth of same is not less than one-eightieth ($\frac{1}{80}$), and no greater than one and a half ($1\frac{1}{2}$) times the depth when this ratio is one over one hundred and twenty ($\frac{1}{120}$). Distances for intermediate ratios are to be interpolated.

Tee-irons are not to be used as stiffeners.

Stiffening angles, which must always be in pairs (one angle on each side of the web), must extend from the upper leg of the upper flange angle to the lower leg of the lower flange angle, being made flush with the other legs of the flanges by filling plates.

Web Splices in Floor Beams and Plate Girders. — Webs of floor beams and plate girders must be well spliced at all joints by a splice plate on each side of the web; and joints must be located where the shear is not great.

Limiting Depths of Floor Beams and Plate Girders. — The depth of the web of a floor beam or plate girder must never exceed one hundred and twenty (120) times its thickness.

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* For proof of this formula, see Appendix II.

Rivets in Flanges of Floor Beams and Plate Girders. — In spacing the rivets in the flanges of floor beams and plate girders, the flanges are to be divided into portions of about two (2) feet in length, the stresses in the flanges are to be found at each point of division, and there must be enough rivets between any consecutive points of division to take up the difference between the stresses at the points, providing that the rivets be not spaced more closely than two and a half ($2\frac{1}{2}$) diameters, nor more than six (6) inches apart.

Eyes. — In welded heads, the length of metal behind the pin-hole must be at least equal to the depth of the bar or diameter of pin, whichever be the greater, unless the head be correspondingly thickened; while in hammered heads the amount is to be the same as that above or below the pin hole.

The least amount of metal in the heads across the pin holes is to be as given in the following table: —

WIDTH OF BAR	DIAMETER OF PIN	METAL IN HEAD ACROSS PIN.	
		Welded.	Hammered.
1	0.80	1.50	1.50
1	1.04	1.50	1.50
1	1.12	1.50	1.53
1	1.20	1.50	1.56
1	1.28	1.50	1.60
1	1.36	1.55	1.72
1	1.43	1.60	1.76
1	1.50	1.67	1.85
1	1.64	1.67	1.95
1	1.77	1.70	2.05
1	1.90	1.76	2.21

In loop eyes the distance of the inner point of the loop from the centre of the pin must not be less than three (3) times the diameter of the pin. The loop must fit closely to the pin throughout its semi-circumference.

Pin holes in eye bars shall be bored to an exact size and distance, and to a true perpendicular to the line of stress. No error in the length of bar exceeding one sixty-fourth ($\frac{1}{64}$) of an inch will be allowed, nor any variation of more than one thirty-second ($\frac{1}{32}$) of an inch between the centre of the eye and the centre line of the bar.

Pins. — Pins are to be proportioned to resist the greatest bending produced in them by the bars or struts which they connect.

Steel pins are also to be proportioned for shearing.

No pin is to have a diameter less than eight-tenths ($\frac{8}{10}$) of the depth of the deepest bar coupled thereon; nor shall it vary from that of the eyes of the bars coupled thereto by more than one-fiftieth ($\frac{1}{50}$) of an inch.

The least allowable diameters for chord pins are two (2) inches for bridges of Class A, and one and a half ($1\frac{1}{2}$) inches for those of Classes B and C. The least allowable diameter for pins belonging wholly to the lateral systems of bridges of any class is one and a quarter ($1\frac{1}{4}$) inches.

Pin Bearings. — All pin holes through webs shall be re-enforced by additional material, so that the permissible pressure upon the bearings shall not be exceeded. Where a pin bears against a re-enforced channel bar, the web of the latter is not to be assumed to take any bearing-stress, unless the re-enforcing plates be riveted to it before the pin-hole be bored.

Chord Packing. — The lower chords are to be packed as closely as possible, and in such a manner as to produce the least bending-moments upon the pins. The various members attached to any pin must be packed as closely as possible, and all interior vacant spaces must be filled with wrought-iron fillers.

Expansion Rollers. — Expansion rollers for bridges of Class A are to be proportioned by the formula

$$p = 0.25\sqrt{d},$$

and those for bridges of Classes B and C by the formula

$$p = 0.3125\sqrt{d},$$

where p is the working-load in tons per lineal inch of roller, and d is the diameter of the roller in inches.

The least allowable diameters for rollers are one and three-quarters ($1\frac{3}{4}$) inches for bridges of Class A, and one and a half ($1\frac{1}{2}$) inches for bridges of Classes B and C. The spaces between rollers must never exceed three-quarters ($\frac{3}{4}$) of their diameter.

Turn Buckles and Sleeve Nuts. — All turn buckles and sleeve nuts must be made so strong, that they will be able to resist without rupture the ultimate pull of the bars which they connect, and without distortion, the greatest twisting-force to which they could ever be subjected. U-nuts are not to be used in any part of a bridge.

Sizes of Nuts. — The dimensions of all square and hexagonal nuts for the various diameters of rods are to be taken from Carnegie's "Pocket-Companion" (pp. 130, 131), excepting those nuts on the ends of pins which are subject to but a slight tendency to shear the thread: in this case, these dimensions may be diminished, in direct proportion to this tendency, until the thickness reaches the limit of one-half ($\frac{1}{2}$) of an inch.

Washers and Nuts. — Washers and nuts must have a uniform bearing. Cast-iron washers must be used under the heads and nuts of all timber bolts, when the bearing is on the wood.

Beam Hangers. — Whenever possible, four (4) beam hangers per beam are to be used. The screw ends are to be provided with check nuts.

Jaws. — Great care must be taken, in designing jaws for the ends of any strut, that they be so strong in every respect, that, when the strut is subjected to its ultimate load, it will fail near the middle rather than at the ends.

Brackets. — Brackets, or knees, must be used to connect each overhead strut to the posts or batter braces. They should be of straight tee, angle, or channel iron: if made curved, no dependence is to be placed upon them, either for strength or stiffness. When there is no vertical sway bracing, each intermediate bracket must be proportioned to resist the compression induced in it by the wind pressure concentrated at the windward and leeward points of that panel of the top chords to which the bracket belongs; and each portal bracket, to resist the compression induced in it by one-half of the total wind pressure concentrated at the panel points of the top chords.

Cutting off the Flanges of Channels. — The flanges at the ends of channel bars must not be cut away, if it be practicable to avoid doing so: if not, there must be sufficient re-enforcing used to make the strut as strong as it would have been with the flanges uncut.

Sizes of Flooring and Joists. — Pine flooring is to be at least three (3) inches thick, and oak flooring at least two and a half ($2\frac{1}{2}$) inches thick. It is to be laid with close joints, and well spiked to each alternate joist with two (2) seven (7) inch cut spikes. Consecutive boards should not be spiked to the same joists.

Joists are to be proportioned by the formula,

$$W = \frac{bd^3}{cl^2}$$

where W is the safe uniformly distributed load in tons, b the breadth of the joist in inches, d the depth of same in inches, l the length in feet, and $c = 16$ for pine and 11.5 for oak.

Where the greatest load is a concentrated one, it is to be considered as supported equally between the joists directly under the wheels and those contiguous to the same; i.e., the wheels on one side of a wagon are supposed to be placed directly over a joist, which joist is assumed to take half their load, the remaining half being equally divided between the two adjacent joists. All concentrated loads must be reduced to equivalent uniformly distributed loads in respect to deflection by multiplying them by one and six-tenths (1.6) before applying the formula.

Wooden Hand Rails, etc. — Wooden hand railing is to be made of pine, the posts being $4'' \times 6'' \times 4'$, with two runs of $2'' \times 6''$ timbers — one on its flat, and the other below on edge to support the first for a hand rail — and one run of $2'' \times 12''$ hub plank. The latter and the lower run of $2'' \times 6''$ timber are to be let into the posts to their full depth, and spiked to the same with five (5) inch cut spikes; and the posts are to be halved on the outer joists, to which each one is to be bolted by two (2) three-quarter ($\frac{3}{4}$) inch bolts.

Guard rails are to be of $6'' \times 6''$ pine, bolted to each hand-rail post, and to the floor once in, at most, every five (5) feet, by three-quarter ($\frac{3}{4}$) inch bolts.

When there are no wooden hand-rail posts, the guard rails must be bolted through joists placed symmetrically below them, care being taken that there be no bolt within two (2) feet of the middle point of any joist.

The joints in the guard rails are to be lap joints, six inches long, and are to have a bolt passing through the middle of each lap.

About every eighth or tenth plank, there is to be a crack left between flooring boards to let the water run through: this crack should be less than a quarter ($\frac{1}{4}$) of an inch in width.

Should it be considered desirable to elevate the guard rails in order to let the air and water pass beneath, it is to be accomplished by inserting hard-wood shims — one (1) foot long by two (2) inches deep, and six (6) inches wide — beneath the guard rails at each bolt hole; the bolt passing through the middle of the shim, and each shim being fastened to the floor by four (4) five (5) inch cut spikes.

Iron Hand Railing. — Bridges with sidewalks will not require the wooden hand railing inside the trusses, but are to be provided with a neat, substantial iron hand rail on the exterior of each sidewalk. It is to be rigidly attached to the floor beams and exterior sidewalk joists.

Details not previously Mentioned. — Finally, as regards the proportioning of any structure, if cases should occur which are not covered by the preceding specifications, the following rule is in all such cases to be adhered to: "Details must always be proportioned so as to resist every direct and indirect stress that may ever come upon them under any probable circumstances, without subjecting any portion of their material to a stress greater than the legitimate corresponding working-stress."

Cast-Iron. — No cast-iron is to be used except for washers for timber bolts, and for ornamental work and name plates.

Name Plates. — The names of the designer and the manufacturer of the bridge must be attached thereto in a prominent position and in a durable manner.

Field Riveting. — Field riveting must be done with the button sett. The heads of the rivets must be hemispherical, and no rough edges must be left.

Painting. — All finished work, before leaving the shop, shall be thoroughly cleaned from all loose scale and rust, and covered with one good coat of pure boiled linseed-oil well worked into all joints and open spaces. In riveted work all surfaces coming into contact shall be painted before being riveted together.

Bed plates, and all parts of the work which will not be accessible for painting after erection, shall have two coats of paint.

Pins, bored pin holes, and turned friction rollers, shall be coated with white lead and tallow before being shipped from the shop.

After the structure is erected, the iron-work shall be cleansed from mud, grease, or any other objectionable material that may be found thereon, then thoroughly and evenly painted with two coats of paint mixed with pure linseed-oil, of a color pleasing to the eye; the tension members being generally of a lighter shade than the compression members.

Wherever it be possible to so design it, the iron-work must be made accessible to the paint-brush.

Timber. — All timber is to be of the best quality, free from wind-shakes, large knots, decayed wood, sap, or any defect that would impair its strength or durability.

Quality of Workmanship. — All workmanship is to be first-class; abutting joints are to be truly planed and dressed, so as to insure a perfect bearing; the pin holes in chords, batter braces, and posts, are to be bored as truly as is specified for the eye bars; and there are no rough edges or corners to be left on the iron-work.

Bars which are to be placed side by side, or in similar positions in the structure, shall be bored at the same temperature, and of such equal length, that, upon being piled on one another, the pins shall pass through the holes at both ends without driving. Whenever necessary for the protection of the thread, provision shall be made for the use of pilot nuts in erection.

Quality and Tests of Materials. — All wrought-iron must be tough, fibrous, and uniform in character. It shall have a limit of elasticity of not less than twenty-six thousand (26,000) pounds per square inch.

Finished bars must be thoroughly welded during the rolling, and free from injurious seams, blisters, buckles, cinder spots, and imperfect edges.

For all tension members the muck bars shall be rolled into flats, and again cut, piled, and rolled into finished sizes.

They shall stand the following tests. Full-sized pieces of flat, round, or square iron, not over four and a half ($4\frac{1}{2}$) square inches in sectional area, are to have an ultimate strength of fifty thousand (50,000) pounds per square inch, and are to stretch twelve and a half ($12\frac{1}{2}$) per cent of their whole length.

Bars of a larger sectional area than four and a half ($4\frac{1}{2}$) square inches are to be allowed a reduction of one thousand (1,000) pounds per square inch for each additional square inch of section, down to a minimum of forty-six thousand (46,000) pounds per square inch.

Specimens of a uniform section of at least one (1) square inch, taken from bars of four and a half ($4\frac{1}{2}$) square inches section, and under, are to have an ultimate tensile strength of fifty-two thousand (52,000) pounds per square inch, and are to stretch eighteen (18) per cent in eight (8) inches.

Similar specimens from bars of a larger section than four and a half ($4\frac{1}{2}$) square inches are to be allowed a reduction of five hundred (500) pounds per square inch for each additional square inch of section, down to a minimum of fifty thousand (50,000) pounds per square inch.

Similar sections from angle and other shaped iron are to have an ultimate strength of fifty thousand (50,000) pounds per square inch, and are to stretch fifteen (15) per cent in eight (8) inches.

All iron for webs of plate girders is to have an ultimate strength of not less than forty-six thousand (46,000) pounds per square inch of area of test-piece, and is to have a minimum elongation of ten (10) per cent in eight (8) inches.

Rivets are to be of the best quality of double refined iron.

The cast-iron must be of the best quality of soft gray iron.

Test of Structure. — On the completion of the entire structure, any bridge, after being in constant use for one day, may be tested by a load equal to that for which it was designed remaining upon it for at least one hour, then removed, and placed upon it again. The first removal of the load may show a little permanent set; but, when the second load is applied, the total deflection must not be any greater than it was for the first loading, and upon its removal the bridge must return to the exact position which it occupied after the removal of the first loading.

CHAPTER III.

LIST OF MEMBERS.

In the following list of members will be found the names of all the parts, both of wood and iron, in the bridges with which this treatise deals. Its use will be explained in Chapter XIV. It is inserted here so that those unacquainted with bridge designing may inform themselves as to the names of all the parts of a bridge before proceeding farther. This they can do by consulting the Glossary, and referring to the plates there indicated. The author would advise students, before proceeding to Chapter IV., to study closely Plates I.–IV., so as to obtain a general idea of how bridges are constructed.

LIST OF MEMBERS IN A WROUGHT-IRON HIGHWAY-BRIDGE.

MAIN PORTIONS.

Channel Bars. — Top chords, batter braces, bottom chords, posts, upper lateral struts, end lower lateral struts, upper portal struts, lower portal struts, hip verticals in pony trusses.

Plates. — Top chords, batter braces.

I-*Beams.* — Floor beams, intermediate struts, end lower lateral struts, bottom chords.

Bars and Rods. — Main diagonals, counters, hip verticals, upper lateral rods, lower lateral rods, vibration rods at portals, vibration rods on posts, chord bars.

Angle Iron. — Side bracing, end lower lateral struts.

Iron hand railing.

Floor beams.

DETAILS.

Stay Plates. — Top chords, batter braces, ends of posts, middle of posts, bottom-chord channels, upper lateral struts, end lower lateral struts, upper portal struts, lower portal struts.

Re-enforcing and Connecting or Splice Plates. — Hip inside, hip outside, top-chord intermediate panel points inside, top-chord intermediate panel points outside, bottom-chord struts at shoe, bottom-chord struts at first panel points, shoe inside, shoe outside, lower ends of posts inside, lower ends of posts outside, middle of posts inside, middle of posts outside, lateral struts to top chords, upper portal struts to batter braces, lower portal struts to batter braces, portal struts to brackets and name plates, intermediate struts to posts, side bracing to floor beams, end lower lateral struts to pedestals.

Cover Plates. — Hip joints, joints at intermediate panel points of top chords.

Filling Plates. — Hips, intermediate panel points of top chord, over end floor beams, between pedestals and lateral struts.

Extension plates at upper ends of posts, shoe plates, roller plates, bed plates, beam-hanger plates, name plates.

Lacing or Latticing. — Top-chord channels, batter-brace channels, bottom-chord channels, post channels, upper lateral strut channels, end lower lateral strut channels, upper portal struts, lower portal struts, hip verticals in pony trusses.

Trussing. — Hip verticals in pony trusses, lower-chord bars.

Pins. — Top chords, bottom chords, middle of posts, lower lateral rod connection to jaws, vibration-rod connection to upper portal and lateral struts, vibration-rod connection to lower portal struts, between floor beams and beam-hanger plates.

Bolts. — Brackets to portal struts and lateral struts, brackets to batter braces and posts, name plates to portal struts, vibration rods to lateral struts, vibration rods to intermediate struts, bed plates to piers (anchor bolts), shoes to bed plates, expansion pedestal connection to bed plates, portal struts to batter braces, hand-rail posts to joists, lower lateral struts to floor beams, lower lateral struts to jaws, felly planks to floor and hand-rail posts.

Brackets or Knee Braces. — Portal struts to batter braces, upper lateral struts to posts, intermediate struts to posts.

Ornamental work at portals, beam hangers, expansion rollers, roller frames, fillers for pins, turn buckles, sleeve nuts, connecting-chord heads for bottom-chord channels, jaws for lateral and portal struts.

Angle Iron. — Batter braces to shoe plates, sides of roller plates, ends of roller plates.

Pieces of Channels. — Bearing for bent eyes of upper lateral rods, bearing for bent eyes of lower lateral rods, batter-brace connection to shoe plates.

Rivet Heads. — Plate to chord and batter-brace channels, latticing or lacing to channels, latticing to latticing, the various stay plates to channels, the various connecting and re-enforcing plates to channels, connecting plates to shoe plates, connecting plates to intermediate struts, connecting plates to side bracing and floor beams, connecting plates at pedestals to pedestals and lateral struts, cover plates to chords, extension plates to posts, trussing to hip verticals and posts, trussing to chord bars, ornamental work to portals, brackets to posts and batter braces, brackets to portal, lateral, and intermediate struts, components of jaws to each other, angle irons to shoe plates, angle irons to roller plates, the various pieces of channels to the parts which they connect.

Spikes. — Flooring to joists, hand rails to posts, hub planks to posts, felly planks to flooring, joists to lower lateral struts, jaws to lower lateral struts.

Washers. — Hand-rail post bolts, bolts connecting lateral struts to floor beams, felly-plank bolts.

Nuts. — On pins, on bolts, on beam hangers, lock nuts on beam hangers, pilot nuts.

Details for a Built Floor Beam. — Web, upper-flange angles, lower-flange angles, top plate, bottom plate, stiffening angles, filling plates, re-enforcing plates at beam-hanger holes, rivet heads.

Details for a Trussed Beam. — Rolled I-beam for upper chord, lower-chord bars, end diagonals, counters, I-beam posts, connecting plates for posts to beam, re-enforcing plates at feet of posts, pin plates for end diagonal connection to beam, stiffeners at supports, pins and nuts on same, fillers, turn buckles or sleeve nuts, rivet heads.

Lumber. — Joists, flooring, hand-rail pieces, hand-rail posts, hub planks, felly planks or guard rails, lower lateral struts.

DETAILS FOR A SPAN COMPOSED OF PLATE GIRDERS.

Webs, upper-flange angles, lower-flange angles, top plates, bottom plates, stiffening angles, filling plates, angles for lateral braces, connecting plates for lateral braces, shoe plates, bed plates, anchor bolts and nuts, rivet heads.

CHAPTER IV.

LIVE AND DEAD LOADS.—WIND PRESSURE.

As stated in Chapter II., highway-bridges are divided into three classes, A, B, and C, which are respectively for locations where the loads are heavy and of frequent occurrence, locations where the loads are occasionally heavy, and locations where the loads are light.

After deciding upon the length of span, width of roadway, and class of bridge for any location, the live load per square foot of floor is to be taken from the table on p. 5. The reason why long spans may be proportioned for lighter loads than short ones is the very small probability of a long span ever being covered by the maximum load, while there is a chance of such an event taking place in case of a short span.

It can easily be seen, then, that, in all bridges of *any length* of span, each panel should be proportioned to sustain the maximum load; for it is possible to load one panel heavily without loading any of the others.

This panel excess will affect only the sizes of the joists, floor beams, beam hangers, and hip verticals. Sometimes the panel excess is supposed to exist when the bridge is partially or wholly covered by the moving load, thus affecting all the main members of the trusses; but this is too much refinement for highway-bridge designing.

The dead load per lineal foot is to be taken from one of Tables I., II., and III., if there be no special loading such as that due to snow, if the style of hand railing, guard rail, etc., for the bridge to be designed, correspond with that adopted in this work, if the width of roadway correspond with one of those in the table, and if the length of the span be exactly divisible by ten. If either or both of the last two conditions be not fulfilled, the dead load is to be directly interpolated; then if there be any difference in the lumber, or any special loading, the effect of the change or changes is to be calculated by the method to be explained presently.

The weights of iron and lumber given in Tables I., II., and III., are the results of calculations for the bills of materials of sixty bridges, so chosen in respect to length of span, and width of roadway, as to indicate the laws by which the weights vary with these dimensions. All the bridges are of the most economic depth and panel length, corresponding in these respects with the dimensions given in Table IV. The figures in the five columns under each roadway give respectively the weights of iron per lineal foot in the trusses, lateral systems, and floor system; the weight of lumber per lineal foot, not including waste; and the dead load per lineal foot, corrected for the small amount resting on the piers.

To find the dead load for any span where there is to be an increased weight of flooring, or an additional load, find the new value for the weight of lumber (proportioning the joists by the method given in Chapter IX., remembering that pine lumber weighs two pounds and a half per foot, board measure, and oak lumber four pounds and a third), then that of the floor system by the formula

$$F' = \frac{F}{2}(1 + r),$$

where F' is the weight per foot of the floor system required, F that of the floor system given in the table, and r (greater than unity) the ratio of total loads on floor beams in the two cases considered.

Next assume T', the new value for the weight T in the truss column, and find the ratio r' (greater than unity) of total loads per foot on trusses in the two cases; then

$$T' = \frac{T}{5}(1 + 4r').$$

If the value of T' thus found agrees with that assumed, all right; if not, it will be necessary to try again until it does agree.

Next add together the differences between T' and T, between F' and F, and between the weights of lumber per lineal foot. To the sum of these differences add the weight of snow or other special loading per lineal foot of bridge, and the dead load taken from the table. The final sum will be the dead load required.

To find the dead load for a bridge with sidewalks, look in the table of the class to which the bridge belongs, and find the weights for a bridge of the same span and roadway without sidewalks, then estimate the increments of these weights as follows : —

First, the weight of lumber per lineal foot on the sidewalks is to be calculated ; and from it is to be subtracted twenty-four pounds, which is the weight per foot of the wooden hand rails, hub planks, and hand-rail posts (lumber not required when there are sidewalks). The difference will be the increment for the "lumber" column.

The increment for the "lateral system" column will be zero : that for the "floor system" column can be found approximately by the formula

$$I = \frac{2bF}{3B},$$

where I is the increment required, F the weight per foot of floor system taken from the table, b the sum of the widths of the sidewalks, and B the clear width of main roadway.

The increment of the "truss" column is found by assuming the dead load required, and calling it W_b.

Let

W_a = the dead load given in the table,

P_a = the live load per lineal foot on the main roadway,

P_b = that on the sidewalks ;

then

$$\frac{P_a + P_b + W_b}{P_a + W_a} = \text{the ratio of total loads.}$$

Let

T_a = the truss weight from the table,

T_b = the new truss weight ;

then

$$T_b = \frac{T_a}{5}\left(1 + \frac{4(P_a + P_b + W_b)}{P_a + W_a}\right),$$

and the increment will be $T_b - T_a$.

Next add W_a, the three increments found, and the weight per lineal foot of the iron hand rails: the sum will be the dead load required. If it agrees with the assumed load W_b, all right; if not, another trial for the new truss weight is to be made.

There is one case in which this method would give too great a result: it is that of a pony truss with side braces, of which the only representative in the tables is the sixty-foot span. To apply the method to this case, it will be sufficiently exact to use the weight of floor system of the fifty-foot span, because the floor beams of the sixty-foot span project beyond the trusses. This change being made, the method can be otherwise followed exactly.

The full double horizontal lines in Tables I., II., and III., divide the single from the double intersection trusses.

All the bridges in Table III. lying to the left of the double vertical line which separates the twenty-two-foot and twenty-four-foot roadways, have stiffened end panels. The corresponding lines of division in Tables I. and II. separate the twenty-foot and twenty-two-foot roadways.

The weights of iron in Tables I., II., and III., *do not include the weight of the spikes.*

It is seldom necessary to make an allowance for snow load in bridges of Class C, but it may be advisable to do so in bridges of Classes A and B; for, after a heavy snow-storm, the travel on country roads would be light, which would not necessarily be the case in a city or its suburbs. The proper allowance for snow load should be from ten (10) to thirty (30) pounds per square foot of floor; according to climate, locality, probability of greatest live load occurring simultaneously with the snow load, etc.

As stated in Chapter II., the wind pressure assumed is forty (40) pounds per square foot for spans of one hundred (100) feet and under, thirty-five (35) pounds for spans between one hundred (100) and one hundred and fifty (150) feet, including the latter, and thirty (30) pounds for all greater spans. It is true that actual wind pressures do occasionally exceed these amounts; but in view of the fact that the chance of any one bridge ever being subjected to such pressure throughout its whole length is extremely small, and that it could receive once in a while a far greater pressure without suffering material injury if the bridge be properly designed, it seems legitimate to adopt the pressures assumed.

Moreover, when a highway-bridge is blown down, the actual loss is seldom much greater than the cost of a new bridge. Travellers can cross the stream at the nearest bridge above or below, until the structure be replaced. And the fall of the bridge need involve no loss of life: for, in the first place, no human being would be likely to be upon it in such a storm; and, in the second, if there were, he could not escape being dashed to pieces or blown off, even if the bridge were sufficiently rigid to withstand the pressure.

With railroad-bridges, of course, it is a very different matter. The delay caused by the loss of such a bridge may be much more expensive than the replacing of the structure. Besides, railroad-bridges are subjected to the greatest wind pressure when covered by a train; so that the fall usually involves the loss of human life.

If the lateral systems of highway-bridges were to be made as strong as those of railroad-bridges, unstiffened eye-bars could be very seldom employed for the bottom chords; because the compression there due to the wind pressure would be far in excess of the tension due to dead load (*vide* Appendix I.).

Even with the pressures assumed, it is necessary to rely upon the stiffness of the joists to prevent buckling the bottom chords of at least two-thirds of the iron and combination highway-bridges in the United States.

It is not necessary to add any area to the section of the bottom chords to resist the tension due to wind pressure, unless this tension exceeds that due to the live load multiplied by the ratio of the intensity of working tensile stress in lateral systems to that of working tensile stress in chords. Should it so exceed, the chords should be proportioned to resist the wind stress plus the dead-load stress multiplied by the aforesaid ratio, using an intensity of seven and a half ($7\frac{1}{2}$) tons, or, in case of stiffened eye-bars, six and a half ($6\frac{1}{2}$) tons. This excess should be looked for in narrow bridges in unusually exposed situations, in the design for which the wind pressure has been increased ten (10) pounds per square foot.

For this same reason, of there being no probability of a live load remaining upon a highway-bridge during a heavy storm, the effect of the wind upon posts and batter braces may be neglected, unless the stresses produced thereby exceed those due to the live load multiplied by the ratio before mentioned; in which case the wind stresses should be multiplied by the reciprocal of this ratio, and to the product should be added the dead-load stresses, in order to find the greatest stresses for which to proportion these members. As before, this excess is to be looked for when the assumed wind pressure has been increased by ten (10) pounds per square foot: it will probably be only the lighter posts that will be so affected.

But the bending effect of the wind pressure upon portal and lateral struts, when no vertical sway bracing except brackets is used, should always be provided for.

The author believes, that, instead of designing highway-bridges in ordinarily exposed situations to resist the greatest recorded wind pressure in the district, it is better to run a little risk of losing a structure than to make all the bridges so much more expensive. Nevertheless, he wishes it to be distinctly understood, that, in advocating the adoption of comparatively low wind pressures, he does not countenance the building of such miserable apologies for lateral systems as one finds in the majority of highway-bridges.

CHAPTER V.

STRESSES IN TRUSSES.

THE length of span having been decided by considerations of both necessity and economy (*vide* Chapter XV.), and the width of roadway by the requirements of travel, there remain to be determined, before making out the diagram of stresses, only the style of intersection, panel length, and depth of trusses. These matters are fully treated in Chapter XV. Meanwhile, the style of intersection may be settled by remembering that the single is more expensive than the double, and that the inferior limits of the latter for the different classes of bridges and different clear roadways are given in the table on p. 8. The most economic panel lengths and depths of truss for locations where long timber is expensive, and, in fact, for nearly all locations, are to be taken from Table IV. For locations where long timber is very cheap, there can be made a little saving in the iron-work by using Table V. instead of Table IV.

As is customary in figuring stresses, uniformly distributed loads are to be considered as concentrated at the panel points; and the half-panel load at each end of the truss is not supposed to produce any stress in any member of the truss.

The first step in making a diagram of stresses is to fill out one of the following tables of data: —

SINGLE INTERSECTION.	DOUBLE INTERSECTION. Even Number of Panels.	DOUBLE INTERSECTION. Odd Number of Panels.
$n =$	$n =$	$n =$
$l =$	$l =$	$l =$
$d =$	$d =$	$d =$
diag. =	short diag. =	short diag. =
sec $\theta =$	long diag. =	long diag. =

SINGLE INTERSECTION.	DOUBLE INTERSECTION. Even Number of Panels.	DOUBLE INTERSECTION. Odd Number of Panels
$\tan\theta =$	$\sec\alpha =$	$\sec\alpha =$
$w =$	$\tan\alpha =$	$\tan\alpha =$
$W_1 =$	$\sec\beta =$	$\sec\beta =$
$W'' =$	$w =$	$w =$
$W' =$	$W_1 =$	$W_1 =$
$\frac{1}{n}w =$	$W'' =$	$W'' =$
$\frac{1}{n}w\sec\theta =$	$W' =$	$W' =$
$W_1\sec\theta =$	$\frac{1}{n}w =$	$\frac{1}{n}w =$
$\frac{1}{2}W_1\sec\theta =$	$\frac{1}{n}w\sec\alpha =$	$\frac{1}{n}W_1 =$
$W''\tan\theta =$	$W_1\sec\alpha =$	$\frac{1}{n}w\sec\alpha =$
$\frac{1}{2}W''\tan\theta =$	$\frac{1}{2}W_1\sec\alpha =$	$\frac{1}{n}W_1\sec\alpha =$
	$\frac{1}{n}w\sec\beta =$	$\frac{1}{n}w\sec\beta =$
	$W_1\sec\beta =$	$\frac{1}{n}W_1\sec\beta =$
	$\frac{1}{2}W_1\sec\beta =$	$\frac{1}{n}W''\tan\alpha =$
	$W''\tan\alpha =$	
	$\frac{1}{2}W''\tan\alpha =$	

where n is the number of panels in the span, l the length of each panel, d the depth from centre to centre of chords, θ the inclination of the diagonal ties in the single-intersection truss to the vertical, w the live panel load in tons on one truss (i.e., one-half the product of the live load per foot by the panel length, or one-half the product of the clear roadway by the panel length by the live load per square foot in pounds, all divided by two thousand), W_1 the panel dead load on one truss, $W'' = w + W_1$, W' the portion of W_1 concentrated at the upper panel point, α the inclination of the short diagonal ties in double-intersection trusses to the vertical, and β the inclination of the long diagonal ties in same to the vertical. The value of W' is usually between one-fourth and one-third of W_1: by taking it always equal to one-third of W_1 a small error on the side of safety will be made in designing short spans.

Having filled out the table of data, the next step is to draw a skeleton diagram large enough to contain all the stresses and sections. It is not necessary that the diagram be drawn to scale; but the ratio of panel length to depth of truss on the diagram, for the sake of appearance, should not vary too greatly from the ratio of the actual values of these dimensions. A panel length of an inch and a half, and a depth of two inches and a half, are about as small dimensions as will be found convenient.

At each lower panel point write lightly in pencil, so that it can be afterwards erased, the number of the panel point, beginning with zero at the right-hand end of the span.

It is well known, and will be accepted here without proof, that the greatest stresses in the chords and batter braces occur when the bridge is entirely covered by the moving load; that the greatest stress in any diagonal exists when the live load extends to its foot from that end of the bridge towards which the diagonal points in a *downward* direction; that the greatest stress in any post occurs when the main diagonal (or, if there be none, when the heaviest counter) attached to its upper end receives its greatest stress; and that the two diagonals of a panel cannot at the same time be subjected to the same kind of stress, excepting, of course, the initial tension.

It is apparent that when the greatest stresses in all the diagonals sloping upward in one direction, and in all the posts and chord panels on one side of the central plane, are found, the greatest stresses in the diagonals sloping in the opposite direction, and in the posts and chord panels on the other side of the central plane, can be immediately written. This fact is so well known, that, in making a diagram of stresses, it is usual to write the stresses on only one-half of the members of the truss.

First let us take a single-intersection through-bridge.

The greatest stress in any diagonal sloping upward from right to left can be found by the formula

$$T = \frac{n'}{2}(n' + 1)\frac{w}{n}\sec\theta + \left(n' - \frac{n-1}{2}\right)W_1 \sec\theta,$$

where n' is the number of the panel point at the foot of the diagonal. This formula is applicable to counters as well as to main diagonals. If the stress should come out negative, it shows that no counter is needed in the panel considered. It is also applicable to the batter brace by putting $(n - 1)$ for n'.

The stress in any post can be found by the formula

$$C = \frac{n'(n'-1)}{2} \cdot \frac{w}{n} + \left(n' - \frac{n+1}{2}\right) W_1 + W',$$

where n' $\left(\text{not less than } \frac{n}{2}\right)$ is the number at the foot of the post.

The stress in any panel of the top chord is given by the formula

$$C = \frac{n'(n-n')}{2} W'' \tan\theta,$$

where n' $\left(\text{not greater than } \frac{n}{2}\right)$ is the number at the end of the panel nearest to the centre of the bridge.

The stress in any panel of the bottom chord, except the one at the end of the span, is given by the formula

$$T = \frac{(n'-1)(n-n'+1)}{2} W'' \tan\theta,$$

n' having the same value as in the last formula. For the end panel, the stress is the same as for the second panel.

As the values of $\frac{w}{n}$, $\frac{w}{n}\sec\theta$, and $W_1 \sec\theta$, are given in the table of data, the substitution in these formulas is a very simple matter.

The stress in the hip vertical is $w + \frac{1}{2}$ (weight of floor beam plus a panel weight of lumber), neglecting the weight of the beam hangers, end lower chord bars, etc., which is not worth considering. It is not necessary to calculate this stress; for the section required. or the size of the square bars, if that shape be employed, can be taken immediately from one of Tables VI., VII., or VIII.

Some engineers may object to using formulas for figuring stresses: if so, the following method will give the same results for single-intersection bridges.

Pass a vertical plane through the middle point of the bottom chord: all the dead loads to the right of this plane may be considered to go to the right-hand pier, and all to the left of the plane to the left-hand pier. Should there be a post at the middle of the bridge, the weight at the foot is to be considered as halved, one-half going to each pier. Then the stress in any main diagonal of the left-hand half of the bridge is to be found by commencing at the right-hand end, and adding the numbers at the panel points until the foot of the diagonal considered is reached, multiplying the sum by $\frac{1}{n} w \sec\theta$, and to the product adding the number of panel dead loads between the central plane and the panel point at the foot of the diagonal considered (including the one at this point) multiplied by $W_1 \sec\theta$.

For instance, in a ten-panel bridge, the stress in the end main diagonal, the number at its foot being eight, will be

$$(1 + 2 + 3 + \text{etc.} \ldots + 8)\frac{w \sec\theta}{10} + (\tfrac{1}{2} + 1 + 1 + 1) W_1 \sec\theta.$$

The stress in a counter on the right-hand half of the bridge will be found by adding the numbers at the panel points until the foot of the counter considered is reached, multiplying the sum by $\frac{1}{n} w \sec\theta$, and from the product subtracting the dead-load stress of the main diagonal which crosses the counter. Thus, in the ten-panel bridge, the stress in the second counter from the centre in the right-hand half of the span, or the one at the foot of the third panel point, is

$$(1 + 2 + 3)\frac{w \sec\theta}{10} - (\tfrac{1}{2} + 1) W_1 \sec\theta.$$

The greatest stress in any post is found by adding W' to the vertical component of the greatest stress in the main diagonal attached to its upper end; thus, in the same bridge, the stress in the first post from the left-hand end, or the one at the eighth panel point, is

$$(1 + 2 + 3 + \text{etc.} \ldots + 7)\frac{w}{n} + (\tfrac{1}{2} + 1 + 1) W_1 + W'.$$

For the case of a middle post, the stress in one of the counters at the upper end must be substituted for that of the main diagonal; thus, in the same bridge, the stress in the middle post is

$$(1 + 2 + 3 + 4)\frac{w}{n} - \tfrac{1}{2}W_1 + W'.$$

The stresses in the chords are to be found by the following method:—

Pass a plane through the foot of the post at or nearest to the middle of the truss, and take the centre of moments at this foot. From the moment of the re-action at the nearest end of the bridge subtract the sum of the moments of the panel loads (W''') lying between the centre of moments and this end, and divide the difference by the depth of the truss. The result will be the stress in the panel of the top chord nearest the centre of the bridge: it will be some multiple of W''' tan θ.

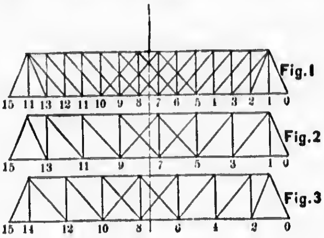
The stress in the panel of the bottom chord immediately below will be equal to the one found, less the horizontal component of the *main diagonal* of the panel, when the bridge is covered by the moving load. This horizontal component will be zero for a truss with an odd number of panels, and $\frac{1}{2}W'''$ tan θ for a truss with an even number of panels.

The stress in the next panel of the bottom chord towards the nearest end of the bridge is found by subtracting from the one already determined the horizontal component of the stress in the *main diagonal* at the panel point between the two panels considered; the bridge, as before, being fully loaded. This component is a multiple of W''' tan θ. In this way can be found all the stresses in the panels of the bottom chord, the correctness of the work being checked by seeing if the stress in the end panel be equal to the re-action multiplied by tan θ. If so, the remaining upper-chord stresses may be at once written by inspection; for the stress in the nth panel of the top chord, counting from the nearest pier or abutment, and supplying the missing panel at the end, is numerically equal to that in the $(n + 1)$th panel of the bottom chord.

It seems almost unnecessary to state, that the stresses in the top chords, batter braces, and posts, are compressive, and those in bottom chords, main diagonals, counters, and hip verticals tensile.

Next let us consider the double-intersection truss.

The formulas for this case are so complicated that it is better not to employ them. The simplest method is to draw a skeleton diagram, and number the panel points, as in the single-intersection truss. The double-intersection truss really consists of two trusses, as may be seen in the accompanying diagram.



Such a division is necessary in order to calculate the chord stresses when the truss contains an odd number of panels. This is accomplished by finding, by the method of moments already explained, the chord stresses in each of the trusses shown in Figs. 2 and 3, and then combining them. Thus the stress in panel 9–10 of the lower chord in Fig. 1 is equal to that in panel 9–11 of Fig. 2, plus that of panel 8–10 of Fig. 3.

The live-load stress in any diagonal sloping upward from right to left is found by noting whether the number at its foot be odd or even, then taking the sum of the odd or even numbers, from one or two up to the number at the foot of the diagonal, and multiplying the sum by $\frac{w}{n}$ sec α, or $\frac{w}{n}$ sec β, as the case may be.

The stress due to the dead load is found by taking the sum of the same numbers, and from it subtracting the sum of the odd or even numbers from one or two up to $n - (n' + 2)$, where n is the number of panels in the span, and n' is the number at the foot of the diagonal considered. Whether the odd or even numbers should be taken can be ascertained by following out towards the left the system to which the diagonal belongs: if the system contain the short diagonal at that end, then the even numbers are to be taken, otherwise the odd ones.

The difference thus found, multiplied by $\frac{W_1 \sec \alpha}{n}$, or $\frac{W_1 \sec \beta}{n}$, as the case may be, will give the dead-load stress in the diagonal. Thus, in the diagram, the dead-load stress in the main diagonal at the panel point 10 is

$$[(2 + 4 + \text{etc.} + 10) - (1 + 3)] \frac{W_1 \sec \beta}{n}.$$

As in the case of the single intersection, the stress in a main diagonal is equal to the sum of the live and dead load stresses; that in a counter, to the difference between its live-load stress and the dead-load stress of the main diagonal crossing it at the middle of its length; that in a post, by the sum of W'' and the vertical component of the greatest stress in the main diagonal (or, if there be none, that in the principal counter) attached to its upper end. As the batter braces belong to both systems of triangulation, their stresses are the sum of the stresses found by each system, or by the formula

$$C = [1 + 2 + 3 + \text{etc.} \ldots + (n - 1)] \frac{(w + W_1) \sec \alpha}{n}.$$

If the number of panels be even, the calculation for the dead-load stresses may be much simplified by counting the number of panel points on the system considered lying between the central plane and the panel point at the foot of the diagonal, including the latter, remembering that the load at the middle panel is halved, and multiplying the result by $W_1 \sec \alpha$, or $W_1 \sec \beta$.

The finding of the chord stresses is also simplified when there is an even number of panels; for they can then be calculated by the method explained for the single-intersection truss.

In every double-intersection truss, there is necessarily a little ambiguity; for it is possible that the whole of the load concentrated at the first panel point does not travel by the system of odd numbers; but this ambiguity is a matter of small moment.

The only difference between the stresses in a deck bridge and those in a corresponding through bridge will be in the posts, the stresses for which are to be found by letting the live load extend from the farthest end of the bridge to the top of the post; so that the post will no longer take its greatest stress with the main diagonal attached to its top, but with the one attached to its foot.

The formula for post stresses in single-intersection deck bridges is, therefore,

$$C = \frac{n'(n'-1) + 2n}{2}\left(\frac{w}{n}\right) + \left(n' - \frac{n+1}{2}\right)W_1 + W'.*$$

To find the stress in a post of a double-intersection deck bridge, add w, W', and the vertical component of the greatest stress in the principal diagonal attached to its upper end.*

In designing bridges where there is an assumed snow load, the counter stresses, and the post stresses produced by the counters, should be figured *without* the snow load; because, the greater the dead load, the less the counter stresses.

In Carnegie's "Pocket-Companion," pp. 141–143, will be found tabulated the numerical co-efficients for the stresses in single-intersection trusses having from three to twelve panels, and in double-intersection trusses having from eleven to twenty panels. The panel dead loads are supposed to be concentrated on the

* This method of finding post stresses is not exact, but gives an error on the side of safety, varying from $\frac{w}{2}$ at the centre to zero at the ends of the span: it assumes the total panel load w to pass down the post before being divided into the portions which pass to right and left, when in fact the portion going to the farther end passes down the main diagonals as compression. The formula was originally obtained under the false assumption; but it has been retained for the following reasons: —

1st, There is a certain amount of shock accompanying the application of the panel live load on the post;

2d, The load w coming from the floor beam is applied to one side of the axis of the post, and consequently tends to produce a slight bending thereon;

3d, The distribution of the excess of stress is favorable, being greatest for the light posts near the middle of the span, and smallest for the heavy ones near the ends.

bottom chords in through bridges, which will cause an error on the side of danger in the post stresses: this fact is pointed out on p. 141. A slight difference will be found between the coefficients there given for the diagonal and chord stresses of double-intersection trusses having an odd number of panels, and those obtained by following the method indicated in this chapter. The latter will give stresses slightly in excess of those in the "Pocket-Companion;" but the difference is so small, that it is scarcely worth mentioning. Had the engineer who prepared the tables been a believer in the use of long panels, he would have commenced his double-intersection trusses with seven panels instead of eleven.

Tables XLI. and XLII. give the stresses for all bridges treated in this work.

CHAPTER VI.

STRESSES IN LATERAL SYSTEMS AND SWAY BRACING.

THE wind loads concentrated at the panel points are determined by imagining a horizontal plane passing through the middle of the truss, and supposing that the pressure on all the exposed surface of the bridge above this plane is concentrated at the upper-panel points, and all below this plane at the lower-panel points. This may be a correct assumption, or may not; but it is as likely to be correct as any other.

Where vertical sway bracing is used, the division of wind pressure becomes still more ambiguous; but, as before, the same assumption is as likely to be correct as any other.

In calculating the area opposed to the wind, the area of the vertical projection of one truss, hand railing, including hub plank, guard rail, and the rectangles described about the windward ends of the floor beams, is to be doubled, and to this is to be added the area of the vertical projection of the floor and joists.

As the windward hand rail would probably fail under high pressure, the total area thus found is somewhat in excess; but such a failure should not be depended upon when the wind is considered to strike the bridge suddenly. For spans of and under two hundred, or sometimes even two hundred and thirty feet, the sizes of the upper lateral rods are not to be determined by the effect of the wind pressure, as this method would make them smaller than experience would indicate to be necessary for rigidity. The sizes to be used can be found in Table XXV.

The wind stresses on the lateral systems are to be calculated for a moving load, instead of one upon the whole bridge; because this method causes the rods towards the centre of the span to be somewhat increased in diameter: besides, it is possible for a portion only of a structure to be subjected to wind pressure; the rest being protected by a hill, a building, or some other neighboring object.

Without making any appreciable error, the wind pressure, for the purpose of simplifying calculation, may be considered as equally distributed between the two sides of the bridge, although the windward side does receive the larger share.

The stress in any diagonal can be found by the formula

$$T = \frac{n'(n'+1)}{2} \cdot \frac{w \sec \theta}{n},$$

and that in any strut, except at the end of the lower lateral system, by the formula

$$C = \frac{n'(n'-1)+n}{2n} \cdot w,$$

where w is the sum of the pressures at a windward and leeward panel point, n the number of panels in the wind bracing, counting in the two lacking at the ends of the upper lateral bracing in through bridges, n' (not less than $\frac{n}{2}$) the number at the leeward end of the diagonal, or at either end of the strut, the panel points being marked as directed in the last chapter, and θ the angle that the diagonals make with the struts.

The stresses in the diagonals are to be increased for initial tension, or, what is the same thing, the working-stresses are to be taken from Table IX.

The effect of the initial tensions on the struts is also to be added to the stresses in those members.

The method of calculating the stresses in the vertical sway bracing is as follows. It is essentially that of Professor Burr, as given in his treatise on "Stresses in Bridge and Roof Trusses."

In Fig. 1, let P be the pressure supposed to be concentrated at the upper panel point on one side of the bridge. It is that which comes upon a panel length of top chord, one-half the area of the diagonals meeting at the panel point and the portion of the post above the plane AB.

Let P' be the pressure concentrated at one end of the intermediate strut JK. It is that which comes upon the portion of the post between the planes AB and CD, the latter passing halfway between the intermediate strut and the bottom chords. If the intermediate strut be at the middle of the post, and if the main diagonals and counters be coupled on a pin at this point, it would be necessary to divide the pressure upon the diagonals between the upper, middle, and lower points of the posts; the middle taking one-half, and the others one-quarter each.

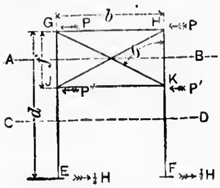


Fig. 1.

Let

$d =$ the depth of the truss,

$f =$ the vertical distance between the upper lateral and intermediate struts,

$b =$ the perpendicular distance between centres of trusses,

and

$\theta =$ the angle made by the vibration rods with the vertical.

The pressures concentrated at the lowest points of the posts do not affect the vertical sway bracing, so are not considered.

The total pressure, $2(P + P') = H$, is assumed to be equally resisted by the feet of the posts. It is possible that this assumption is incorrect, for one foot may resist more than the other; but, when it is remembered that perhaps the whole of the force $2P$ passes through the upper lateral system to the pedestals at the feet of the batter braces, it will be conceded that the assumption is not upon the side of danger.

If the whole of $2(P + P')$ were to be resisted by the feet of the posts, the functions of the upper lateral system would be rather limited, the whole of the wind pressure upon the structure being carried by the lower lateral system, which is highly improbable.

But, whether the wind pressure upon the upper part of the trusses be carried by the upper or by the lower lateral bracing, it is better, as far as the vertical sway bracing is concerned, to proportion the latter under the supposition that the pressures at the upper panel points are carried thereby to the feet of the posts.

Taking the centre of moments at E, the moment of the pressure is

$$2Pd + 2P'(d-f),$$

which can be resisted only by the moment of a released weight V upon the foot at F; thus,

$$2Pd + 2P'(d-f) = Vb,$$

and

$$V = \frac{2d(P+P') - 2P'f}{b}.$$

This release of weight V must pass up the vibration rod KG, causing a tension therein equal to

$$V \sec\theta = \frac{2d(P+P') - 2P'f}{b} \sec\theta.$$

To find the stress on the strut JK, pass a plane through the sway bracing, cutting GH, GK, and JK (HJ not being strained); take the centre of moments at G, and consider the forces acting on the left side of the truss; then the moment of the stress in JK will balance the moments of P' and $\frac{1}{2}H$, thus,

$$(JK) = \frac{\frac{1}{2}Hd - P'f}{f} = \frac{d}{f}(P+P') - P',$$

to which must be added the horizontal component of the initial tension in JH. (JK) represents the stress in JK.

The stress in the upper lateral strut GH is that due to the wind pressure, considering it as a portion of the upper lateral system plus the sum of the horizontal components of the initial tensions in the three rods meeting at one of its ends.

If GH be considered as a portion of the vertical sway bracing, its stress may be found by passing a plane, as in the last case, and taking the centre of moments at K, considering the external forces acting on the left-hand half of the truss; then the moment of the stress in GH will balance the moments of the horizontal re-action at E and the pressure at G, the moment of the increased weight at E balancing the moment of the increased re-action; thus,

$$(GH) = \frac{\frac{1}{2}H(d-f) + Pf}{f} = \frac{d}{f}(P+P') - P',$$

or equal to the stress in JK.

At first thought, it might appear that the two stresses found for GH should be added together to obtain the total stress; but such is not the case, for the wind pressures cannot pass by both the vertical sway bracing and the upper lateral bracing: so the greater stress must be taken. In all practical cases, the greater stress will be found by considering GH as belonging to the upper lateral system.

The bending moment on the post is

$$\tfrac{1}{2}H(d-f) = (P+P')(d-f);$$

and, if m be the distance between centres of gravity of post channels, the stress on *one* channel produced by the bending will be

$$C = \frac{(P+P')(d-f)}{m}.$$

The released weight V, on the windward post, passes down the leeward post, producing a stress equal to $\frac{V}{2}$ on each channel, making the total wind stress on *one* channel

$$C + \frac{V}{2}.$$

According to the method given in Chapter IV., if twice this stress, or $2C+V$, exceed the live-load stress on the post, multiplied by seven and a half ($7\frac{1}{2}$), and divided by the intensity of working tensile stress for lower chords, the post must be proportioned for dead-load and wind stresses, instead of dead-load and live-load stresses.

All these formulas, except that for the stress in GH, may be made applicable to the portal bracing by putting for d the length of the batter brace, for f the perpendicular distance between centre lines of upper and lower portal struts, for P' the pressure on one-half of the batter brace, and for P one-fourth of the sum of all the pressures concentrated at windward and leeward panel points of the top chord.

If P_l be the pressure at the leeward hip, then the stress on the upper portal strut will be given by the formula

$$C = \frac{d}{f}(P+P') - P' + P - P_l.$$

The stresses on all vibration rods must be increased for initial tension, or the rods must be proportioned by using Table IX.; and the stress on each portal strut is to be increased by the sum of the components of the initial tensions in all the rods meeting at one of its ends, taken in the direction of its length.

When there is no vertical sway bracing, stiffness is obtained by the use of knee braces, or brackets (*AB*, *CD*, Fig. 2), making angles of forty-five degrees with the vertical. Let the notation be as shown in the figure; V being, as before, the release of weight at F. P is the sum of the pressures at H and G.

Taking the centre of moments at E gives

$$Vb = Pd \quad \text{and} \quad V = \frac{Pd}{b}.$$

Again: taking the centre of moments at A gives the value of the bending-moment M on the strut at that point; thus,

$$M = V(b - S) - \tfrac{1}{2}Pd = \frac{Pd}{2b}(b - 2S).$$

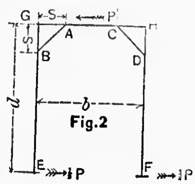


Fig.2

Let h equal the distance between the centres of gravity of the two channels of which the upper lateral strut is composed, then the bending-stress will be

$$C = \frac{M}{h} = \frac{Pd}{2bh}(b - 2S).$$

The intensity of the working bending-stress being six tons, the number of square inches to be added to the area of *each* channel, in order to resist bending, will be

$$A = \frac{C}{6} = \frac{Pd}{12bh}(b - 2S).$$

The stress in AB is found by taking the centre of moments at G, and making the moment of its stress R equal to the moment of the horizontal re-action at E; thus,

$$RS\sqrt{\tfrac{1}{2}} = \tfrac{1}{2}Pd,$$

and

$$R = \frac{Pd}{S}\sqrt{\tfrac{1}{2}} = 0.707\frac{Pd}{S}.$$

As before, to make these formulas applicable to a portal, make d equal to the length of the batter brace, and P equal to one-half the sum of the pressures concentrated at all the upper panel points of the bridge.

To find the effect of the wind on posts and batter braces, use the formula previously found, substituting in it S for f.

Finally, the stress in an end lower lateral strut, at the free end of the span, may be obtained by the formula

$$C_n = \frac{2n-1}{4} \cdot w + I \cos\theta + \frac{n-1}{4} \cdot w' - \tfrac{1}{4}\left(\frac{W}{4} - V\right),$$

where n is the number of panels in the bridge, w the sum of the windward and leeward panel wind loads for the lower system, w' the same for the upper system, I the initial tension in the end lower lateral rod, θ the angle between this rod and the strut, W the total weight of the unloaded bridge, and V the release of weight at a windward shoe.

Owing to the fact that the joists of the end panel rest on the masonry, this formula will give a result slightly on the side of safety.

One or two applications of this formula will convince the most sceptical, that the general idea that any section is strong enough for a strut between pedestals is a fallacy. Too great a reliance has hitherto been placed upon the friction of the shoe, the released weight there not having been considered; and the pressure which comes from the upper panel points seems to have been neglected.

CHAPTER VII.

REMARKS CONCERNING MAIN MEMBERS.

Top chords should nearly always be built of two channels, with a plate on top, and latticing or lacing below. It is never good practice to use a single **I**-beam for top chord or batter brace, because of the great variation in stiffness in its two principal rectangular planes and the difficulty in making neat details for the connections. When the span becomes so short that it appears to be economical to use such a section, it is short enough to employ plate girders which are far superior, both as regards strength and stiffness, to a bridge with **I**-beam chords.

The same objection applies to an **I**-beam post, a favorite design of inferior bridge companies. If one were to take the trouble, in passing over a few bridges where they are used, to cast his eye along the posts, he would generally see that they are bent to one side or the other, or to both; the latter being the case when there are employed what are termed out West "Giasticutus rods," or horizontal rods five-eighths or three-quarters of an inch in diameter, passing from the middle of one post to the middle of the next in the same truss. Such rods are a noticeable feature in arch bridges, a class of structure that ought to be universally condemned. The principal objections to these bridges are their lack of rigidity, and their inability to resist wind pressure, because of the absence of efficient lateral bracing. But another grave fault is, that, being as a rule built by companies of the lowest order, they are weak in section and detail, and the workmanship is poor. They are, without doubt, the cheapest kind of iron bridge that can be manufactured: hence their general adoption throughout the West, where short-sighted economy in building is the order of the day.

The I-beam is more often found in upper lateral struts, where its use is quite as objectionable. Even if strong enough, which it seldom is, it is by no means the best section for that place, owing to the difficulty in connecting to the top chord. Where it rests on the chord plate, and is riveted thereto, the lateral rods being attached to the chord pins, there is a great leverage afforded to the wind stresses to distort the chord; and, where connected to the pin by a jaw, the detail has to be either very clumsy or very weak. Another objection to I-beams for lateral struts is the little room which there is in the flanges for punching rivet holes. But the chief one is the small resistance that they offer to the bending effect of the wind pressure when there is no vertical sway bracing. What has been said of I-beams in lateral struts can be said with much more effect concerning I-beams in portal braces, for great stiffness and strength are there necessary in order to carry the wind pressure upon the upper half of the bridge to the foundations.

The proper function of an I-beam is to resist deflection in the plane of its web: consequently it should be used as a floor beam, in which place its depth should seldom be less than ten inches, never less than nine inches. When one is debating about using such small floor beams, he should figure them for a concentrated wheel load, as well as for a uniformly distributed load.

About the only places where a small I-beam can be legitimately employed are between the pedestals, as a lateral strut at the fixed end of a span, or at the free end if the bridge be narrow and the span very short, and in vertical sway bracing as an intermediate strut.

For upper lateral struts, iron gas-pipe was formerly often employed, and is so yet to a certain extent. Regarded as a section, nothing could be better or more economical; but the connections made with it are very weak. Then, again, there is the objection that it is a closed column, and consequently inaccessible to painting. Notwithstanding the fact that two of the leading bridge companies of the United States employ almost exclusively closed columns, such columns are not, by engineers in general, conceded to be so good as open ones, which are always accessible to the paint-brush.

Some other common forms of upper lateral struts are the following: two tee-irons trussed, the upper resting on the chords, and riveted thereto, the lower abutting against the same, and attached by bent plates; two channels trussed and attached to the chords in the same manner; a combination of a channel and a plate, with trussing between; and two tee-irons laced or latticed, with a jaw plate at each end wider than their flanges, screwed up to the chords by nuts on the ends of the chord pins. Owing to their lack of both strength and rigidity, all these are poor contrivances, two channels laced or latticed being the best form of strut that can be designed for the upper lateral system.

As stated in the "General Specifications," in no highway bridge should the channels in chords, posts, or batter braces, be less than five inches in depth, nor in any other part of the structure less than four inches. One does hear occasionally of such a thing as a three-inch channel top chord with two-inch pins, for a sixty or seventy foot span. But, fortunately for the public safety, such structures are few and far between. The author once heard the senior representative of one of the most flourishing highway-bridge companies in America contend that two three-inch channels trussed make a very good centre post for short through-spans, — strong enough, because the area called for by the stress is less than three square inches. He must either have forgotten, or been ignorant of, the fact that stiffness is as important a factor in a bridge as simple strength. In reality, strength is dependent upon stiffness; for where vibration can occur, the stresses are increased, not only in the members where stiffness is wanting, but in adjoining members of the structure.

Light sections for compression members are more economical than heavy ones, and it is generally preferable to use them. But, if the situation be one where the members will be exposed to excessive moisture, the webs should be thickened.

The top plate for chords and batter braces should generally be from one-quarter to three-eighths of an inch thick. Any thing below the inferior limit would be liable to distortion when roughly handled, and to rust through too readily; and any thing above the superior limit would usually be inconsistent with the best distribution of area in the section.

It used to be customary, and the practice is still followed to some extent, to make the top plate of varying thickness, or to vary the number of plates, increasing from the ends of the truss to the centre, making the channels of the same dimensions throughout. But this method is not advisable; for the proper place for the larger part of the material in a chord like the one under discussion is in the channels, and not in the plate. Similarly, in any channel, the proper place for the larger part of the material is in the flanges, and not in the web; the reason being, in both cases, that the moments of inertia of the section in respect to vertical and horizontal neutral axes are increased by removing a portion of the area away from these axes, and the strength of a strut increases with the moments of inertia of its section.

Star iron should never be employed in an iron bridge, and there is never any necessity for using tee-iron. Two of the latter sections, latticed by a triple or quadruple intersection of thin, narrow bars, are sometimes adopted for a portal brace; but it is evident how weak such a strut must be, and it is in the very place where a strong one is most needed.

Four angles with the legs turned in, and set at the corners of a square, laced on the four faces thus formed, make an economical strut, as far as the section is concerned; but it is probable that the extra weight of detail and the increased cost of shop-work will make it more expensive than another strut of larger section. Two channels latticed or laced are the best form of portal strut. Large, heavy cast-iron portals made in one or two pieces look very well, and might be made strong enough, but are not so neat and graceful as some other kinds of bracing, besides adding unnecessary dead load to the structure. Cast-iron is not to be depended upon, and should not be used in any part of an iron bridge to resist stress.

Channels in posts usually have their webs parallel to the direction of the plane of the truss, with their flanges turned outward: sometimes they are turned inward; and, where the floor beams are riveted to the posts, the webs are, or should be, placed at right angles to the plane of the truss, the flanges turning outward.

Theoretically it is more economical, as far as the area of the section is concerned, to turn the flanges in, for the moment of inertia is greater; but, on the other hand, the difficulty encountered in riveting in a confined space more than equalizes the advantage just mentioned.

Another advantage which can be claimed for channels turned in, viz., avoiding cutting them off before reaching the upper chord pin, is partially counterbalanced by the increased size of pin, due to the larger leverage thus given to the stresses in the diagonals. Notwithstanding the difficulty in riveting, it is often found necessary, in swing bridges, to turn in the flanges of the post channels in order to form a good connection with the channel bottom chords: otherwise, the channels of the bottom chords may be turned in, and the post channels be allowed to bestride them.

The objection to cutting away the flanges of channels at the feet of posts has been shown by some experiments made by the Chicago and Alton Railroad Company, as given in a paper read before the Western Society of Engineers by Mr. E. J. Ward, who shows that this cutting-away reduces the strength of the strut about ten per cent.

Main diagonals, as will be demonstrated in Chapter X., should have the proportion of width to depth of about one to four; and the chord bars, the proportion of from one to four to one to seven, according to the number of them in the panel.

It is preferable, for appearances, to make the counters of square or round instead of flat bars, because of the unsightly change that there would be in the diameter of the flat bars at the upset ends. It is immaterial, except for the effect upon the pins, whether the hip verticals be flat, square, or round; but the preference is usually given to square iron.

Built floor beams in ordinary bridges should be formed of solid plates and angles, and not made trussed; because, even if the latter method permit of a saving of material, it is more conducive to vibration. Where the panels are long and the roadway is very wide, it would be permissible to use trussed beams, provided that they be made very rigid in their details, and not too slight in their sections.

CHAPTER VIII.

PROPORTIONING OF MAIN MEMBERS OF TRUSSES, LATERAL SYSTEMS, AND SWAY BRACING.

HAVING found all the stresses in the main members of the truss and in those of the lateral systems and sway bracing, and having written them alongside the respective members in the diagrams, the next step is to calculate the sections required. The diagrams for the lateral systems and sway bracing may be roughly drawn in pencil; for they need not be preserved, as the sizes of the members are to be written on the truss diagram.

For the tension members of the trusses, the sections required can be found by dividing the stresses on the diagram by the proper intensities of working-stress, as given on p. 12; remembering that the intensities for main diagonals are to be interpolated. When found, the required areas for the sections should be written on the diagram, after the stresses, prefixing them with the letters S. R. (section required), as shown on Plate V. Then, by using Carnegie's "Pocket-Companion," pp. 94–105, or some equivalent tables, can be found the sizes necessary to give *at least* the section required, taking care that the sections be in good proportion.

The stresses in the counters are to be increased for initial tension by the amounts given on p. 10; or, what is the same thing, the size required can be found from Table IX. by looking down the column headed "Working-Stress = 4 tons per square inch," if the bridge belong to Class A, or down the one headed "Working-Stress = 5 tons per square inch," if it belong to Class B or Class C, until a stress is reached which is equal to or greater than one-half or the whole of the stress on the diagram, according to whether double or single counters be employed; then, by following the horizontal line which contains this stress, either to right or left, will be found the size of the counters or counter required.

As previously mentioned, the sections required for, and the sizes of, the hip verticals, can be found without calculation from one of Tables VI., VII., or VIII. Should the joists and flooring be of oak instead of pine, the section required for, and the size of, hip verticals, can still be found from the table by supposing an increase of one foot in the panel length.

The sizes of the lateral and vibration rods can be found from Table IX. by looking in the column headed "Working-Stress = 7.5 tons per square inch," in the same manner as explained for counters. If the panel length correspond with the one given in Table IV., or if it do not differ greatly therefrom, there need be no calculations made for stresses in the lateral systems and sway bracing; for the dimensions of all the struts and rods for these systems are given in Table XXV. In that table the dimensions in the column marked "Pan. 1" are the sections respectively of the upper portal struts, the portal vibration rods (if any), the lower portal struts (if any), and the end lower lateral rods. Those in the other columns are the sections respectively of the upper lateral struts, the upper lateral rods, the post vibration rods (if any), the intermediate struts (if any), and the lower lateral rods. The portal struts are thus assumed to belong to the first panel; the first upper lateral strut, with its sway bracing, to the second panel, etc.; so that, when the bridge has an odd number of panels, there is no lateral strut or vertical sway bracing given for the middle panel. The forty-foot, fifty-foot, and sixty-foot spans, being pony trusses, have only lower lateral rods. Spans above one hundred and fifty feet in length have vertical sway bracing.

If the counter stresses be large, it is preferable to use double counters: sometimes both single and double counters are employed in the same truss. Where there is an odd number of panels, the centre diagonals should be made double and adjustable. The number of main diagonals per panel is generally two; but, if the sections become so great as to necessitate excessively large chord pins, it is better to employ four; placing two inside, and two outside, of the top chord and posts. The widths of the main diagonals should, for the sake of appearance, increase from the centre of the bridge to the ends. For the same reason, it is well to have all the chord bars of the same, or nearly the same, depth; the correct area of section being obtained for each panel by varying the thickness and the number per panel. In large bridges it is permissible to reduce the depth of the chord bars towards the ends of the span in order to economize on the pins. It is also permissible, when there are several chord bars in the same panel, to employ depths varying by a quarter of an inch, provided that the bars of smaller depth be placed on the inside.

As stated in the "General Specifications," where chord bars are trussed to resist the buckling effect of the wind pressure, the intensities of working-stress for the trussed bars on the *net section* should be reduced to four tons for bridges of Class A, and to five tons for those of Classes B and C.

"Chord packing" is a term applied to the arrangement of the chord bars, diagonals, posts, and beam hangers upon the bottom chord pins. It is a matter of great importance, but is very often neglected. The three principal considerations to be kept in mind while arranging the packing are, that the bending-moments on the pins are to be made as small as possible, that the packing is to be made as close as circumstances will permit, and that there be sufficient clearance to avoid all chance of finding the space between the post channels too narrow when the bridge is being erected.

The width of the packing is dependent, not only upon the number and thickness of the bars, but also upon the width of the top chord plate. The latter is often, in its turn, dependent upon the chord packing.

The usual arrangement is to pack the main diagonals, counters, and beam hangers inside of the posts, and the chord bars outside; bringing the latter, however, within the batter braces at the shoes, unless the end panel contain four bars per truss, when two should go outside, and two inside. It is not absolutely necessary that the chord bars pull in the exact line of the trusses; an inch or two of deflection in twenty feet being scarcely noticeable, and making no appreciable difference in the length of the bar: nevertheless, it is better to make the bars as nearly as possible parallel to the planes of the trusses. The main diagonals should be placed next to the post, then the beam hangers, and inside of all, the counters with a filler between them long enough to permit of the screwing-up of the turn buckles, or sleeve nuts.

The arrangement of the chord bars will be treated in Chapter X.

The sections of the top chords and batter braces are to consist of two channels, with a plate on top, and latticing or lacing below. The same depth of channel, and the same width and thickness of plate, are to be employed from one end of the chord to the other; the difference in area being obtained by thickening the webs of the channels. On this account, there is often an excess of section in the end panels of the top chord, and, in long bridges, even in the next panels.

It is customary and better, but not necessary, to make the depth of the channels in the batter braces the same as that of the channels in the chord. The top plate for the batter brace should be of the same size as that for the chord.

The width of the top plate is dependent upon the depth of the channels; as the transverse distance between the centre lines of the rivets which attach the channels to the plate should be never less, and not (unless there be good reason) much greater, than the depth of the channels. The least dimensions for such plates for different channels are given on p. 15. The chord channels are sometimes spread apart in pony trusses, so as to increase the lateral stiffness; and in any bridge it may be necessary to spread them a little to admit of a certain manner of packing below: but, the more narrow the chord plate, the more economy of material will there usually be.

To proportion the top chord or batter brace for a given stress, assume the depth of the channels, and divide the length of the panel or batter brace by it, both dimensions being expressed in the same unit. Referring to Table X. or XI., according to the class of bridge to be designed, look down the column marked "Ratio of *L* to *D*," until the ratio just found is reached: the number to the right, in the first of the three columns, is the intensity of working-stress to be used. The three columns are for the three cases, — both ends fixed, one end fixed and one end hinged, and both ends hinged, marked ◻◻, ◻◯, and ◯◯ respectively. The tables were calculated by the formula of C. Shaler Smith, C.E.; to whom the author is indebted for its use, and for other valuable information in connection with bridge work. Then, to find the area of the top chord or batter brace, divide the stress given on the diagram by the intensity of working-stress taken from the table; from the quotient subtract the area of the top plate, and divide the remainder by two: the final quotient will be the area of each channel. This calculation should be made with both the stress in the panel nearest the middle of the span and that in the end one, or, in long spans, that in the one next to the end. If, then, with the depth of channel assumed, it be found that there is, in the table of channel sections employed, a light channel that will not be much too heavy for the end, and a heavier one suitable for the middle of the chord, all right: if not, another trial must be made, with a channel of a different depth. The greater the depth of channel, the less the ratio of length of strut to diameter, and consequently the greater the intensity of working-stress, and the less the sectional area required: so, generally speaking, it is well to use the lightest and deepest channels possible, unless the saving in section be small, when it will be more economical, for other reasons, to use the next smaller depth. These reasons will be given in Chapter XV. The dimensions of the channels and plate should be written on the diagram of stresses as shown on Plate V.

The sizes of the post channels are to be found in a similar manner to the one just described, with these two exceptions, — that the column for two hinged ends is to be used, and that there is no plate. Some engineers prefer fixing the upper ends of the posts by attaching them, through the medium of plates, to the chord, thus saving a little in the section; but, as will be seen farther on, there is no true economy in so doing.

In high double-intersection bridges, where the diagonals are halved, and connected by pins passing through the middle of the post channels, as shown in Fig. 15, Plate II., the post may be proportioned for half-length with both ends hinged; but in this case the counters must extend to the ends of the span, although there be no stress in some of them, for the purpose of preventing the posts from moving laterally at the middle.

The upper lateral struts and portal struts are to be proportioned by using Table XI. for both ends fixed, and adding, if necessary, to the section thus found, enough area to resist the bending as determined in Chapter VI.

The ultimate strength of the intermediate struts, which are I-beams, can generally be found from experiments made by the manufacturers; a factor of safety of four being sufficient. In default of such experiments, the approximate working-stresses (*not intensities*) for I-beams used as pillars may be taken from Table XL., which has been compiled from an old edition of Carnegie's "Pocket-Companion." When the I-beam strut is supposed to bend in a vertical plane, its length should be taken equal to the distance between the points of attachment of the brackets; but, when it is assumed to bend in a horizontal plane, its length must be taken equal to the distance between opposite posts of the trusses.

Brackets should extend inward and downward, from about four feet in narrow bridges, to about six feet in wide ones. The sway bracing given in Table XXV. was proportioned for brackets of these dimensions. Brackets beneath intermediate struts not only serve to stiffen the struts, but add to the appearance of the bridge.

The intermediate lower lateral struts being of wood, it will not be necessary to calculate their sections, which practical considerations will always cause to be greater than the stresses would demand.

The lower lateral struts between expansion pedestals should consist of two channels laced or latticed, and attached to the end chord pins by jaws. The size for the channels is to be found by using Table XI. for one fixed and one hinged end, — not that either end is really fixed or hinged, but because the strength of a strut so attached is intermediate between that of one with fixed ends and that of one with hinged ends. It is not positively necessary to use a lateral strut at the fixed end of a span; but it is much better to do so, especially in long spans, not only to distribute the horizontal re-actions, but also to keep the chords in line, for there is necessarily a little play in the anchor bolt holes.

It is not unusual to make the struts between pedestals of the same dimensions at both ends of the span, although the one at the fixed end need not be so strong as the one at the free end.

Appendix I., the substance of which appeared as an editorial in the "American Engineer" of July 20, 1883, shows the necessity for stiffening, at least the end panels of many bottom chords. This can be accomplished in several ways; one by inserting a strut between the inner chord bars; another by using channel bars, laced or latticed, instead of eye bars, in which case the net section of the *webs alone* should be relied on to resist tension; and another by trussing the inner chord bars. The second of these methods is the most satisfactory, but at the same time the most expensive. When stiffening the end panel, it is well, though not perhaps essential, to stiffen also the second panel, where the stress is the same as in the one at the end. Such a practice is certainly conducive to the prevention of vibration of light bridges under rapidly moving loads.

Hip verticals in three or four panel pony trusses are to be made to resist the compression which might be produced in them by over-screwing the turn buckles of the counters. The section to be employed is either that of two channels laced or latticed, or two flat bars trussed: in the latter case, as previously stated in the "General Specifications," the intensities of working tensile stress on the net section are to be three tons for bridges of Class A, and four tons for bridges of Classes B and C. If two channels be used, the net area of the webs alone is to be relied on to resist tension.

CHAPTER IX.

PROPORTIONING OF FLOOR SYSTEM.

THE wooden portions of the floor system are the joists, flooring, hand railing, hub planks, and guard rails, or felly planks. Of these, only the joists require calculation for strength. Pine flooring is generally three inches thick, and oak flooring two and a half inches. The hand railing, when of wood, should consist of $4'' \times 6'' \times 4'$ posts, not more than ten feet apart, $2'' \times 6''$ rails, and $2'' \times 12''$ hub plank, all of pine, and built as shown in Plate II., Fig. 13, and as specified on p. 23.

The guard rails should be of $6'' \times 6''$ pine, connected as specified in the same place.

To proportion the joists, first assume their number per panel and their dimensions, in order to determine the total weight of lumber per panel; to this add the total maximum panel live load, or the product of the panel length by the clear roadway by the live load per square foot, given on p. 5, the sum being expressed in tons; then, referring to Table XIII. or Table XIV., find, with the given panel length and the assumed depth of joists, the safe load for a joist one inch wide, and divide this number into the total load just found: the quotient will be the total width of joists per panel, when laid side by side. Divide this total width by the assumed width of one joist: the quotient will be the number of joists per panel. If it agree approximately with the number assumed, and if the distance between centres of joists, when in place, will be between eighteen and twenty-four inches, all right; if not, another trial must be made, with a different depth of joist, and a new assumed panel weight of lumber. It may be well, in any case, to try two depths of joists, in order to see which is the more economical. The minimum size of pine joists should be $3'' \times 10''$: the maximum size that it is advisable to figure on is $4'' \times 14''$, because deeper joists cannot always be readily purchased. It is to be remembered that pine lumber can be found in the market in only certain sizes, usually even inches in depth, and *always* even feet in length; i.e., timbers $3'' \times 8''$, $3'' \times 10''$, or $3'' \times 12''$ are readily procured, while timbers $3'' \times 9''$ or $3'' \times 11''$ are not; also, if one require joists eighteen feet six inches long, it will be necessary for him to buy lengths of twenty feet, and cut off a foot and a half. Timbers over eighteen feet in length cost more per thousand than those of that and shorter lengths.

Tables XV., XVI., XVII., and XVIII. give not only the sizes of joists, and number per panel, but also the total number of feet, board measure, of pine and oak per panel, including, whenever there is any, waste material.

The total load for a floor beam consists of the live load, the weight of lumber which it supports, and the weight of the beam itself. The latter must of course be assumed: this can always be done with sufficient exactness to determine the floor-beam load. The latter is assumed to be uniformly distributed between centres of bearings.

In calculating the dimensions of a floor beam to sustain a given load, the section of the web is to be assumed; and the beam is to be proportioned according to the formula given on p. 19, and to the principles there enunciated. It may be necessary to make two or three designs, in order to determine the most economic depth; but it will be often found that a variation of several inches in the depth will not affect the weight per foot.

The lower lateral strut, which is to be well bolted to the floor beam, will add considerably to the strength and stiffness of the latter. The joists should be dapped on to the strut at their bearings, so as to offer a resistance to the lateral deflection of both strut and beam.

The lower flange plate, if there be one, need not extend over more than the middle half of the length of the beam. The rivets attaching the plate to the lower flange angles should be staggered, and should be spaced about four inches apart; and the areas lost from the plate and angles by these rivet holes should be deducted when figuring the net section.

In heavy beams, several plates are often used to vary the section gradually from the centre of the beam to the ends; but if one share Weyrauch's views upon rivet stresses, as expressed in his "Structures of Iron and Steel," he will avoid any such practice.

Many bridge companies reduce the depth of built beams at the ends, in order to save a little weight of iron. This method may be advantageous to the company which pays for finished bridges by the pound; but it is seldom so to the manufacturer, for the triangular pieces cut from the web are often wasted: besides, the extra work in cutting the web, bending the angles, and making square rests for the beam-hanger nuts on the inclined flanges, more than counterbalances any saving of material.

For a bridge with sidewalks, reducing the depth of the floor beams at the ends adds to the appearance of the structure, and need not interfere with the bearing of the hanger nuts.

Tables XIX., XX., and XXI. give the sizes of floor beams for all cases ordinarily met with.

To illustrate the method of proportioning an ordinary floor beam, let us take the case of a beam for a twenty-foot panel, fourteen feet clear roadway, and fifteen feet between centres of trusses, the bridge belonging to Class A.

The live load on the beam will be

$$14 \times 20 \times \tfrac{100}{2000} = 14 \text{ tons.}$$

The weight of the lumber, from Table XV., is

$$\frac{2085 \times 2.5}{2000} = 2.606 \text{ tons.}$$

Let us assume the weight per foot of the beam to be fifty-five pounds, the total weight of same will then be

$$\frac{55 \times 16}{2000} = 0.44 \text{ ton.}$$

The total load on the beam is, therefore,

$$14.000 + 2.606 + 0.440 = 17.046 \text{ tons.}$$

The most economic depth for the beam can be found by trial, or by consulting Table XIX., which gives $\frac{1}{4}'' \times 27''$ for the section of the web.

Let us assume these dimensions, and take the effective depth D equal to $26''$; then substituting in the formula given on p. 19, omitting A', and remembering that $T = 4$ tons for bridges of this class, gives

$$A = \frac{17.046 \times 15 \times 12}{8 \times 26 \times 4} - \frac{1}{6} \times \frac{1}{4} \times 27 = 2.56 \ \square'',$$

the half of which is $1.28 \ \square''$, corresponding to a weight per foot of 4.27 pounds, because a bar of wrought-iron one inch square and three feet long weighs just ten pounds. Referring to Carnegie's "Pocket-Companion," p. 68, we find that a $2\frac{1}{2}'' \times 3''$ 4.4# angle will be required. Let us see if a $2'' \times 3''$ 5# angle will do for the bottom flange. Assuming that the rivets are $\frac{5}{8}''$, and the holes $\frac{11}{16}''$, in diameter, the area lost by a rivet hole will be $2 \times \frac{5}{16}'' \times \frac{11}{16}'' = 0.43 \ \square''$, which, added to 2.56, gives $2.99 \ \square''$, corresponding to two angles, each weighing five pounds per foot. The assumed angles will therefore be exactly what are required. For stiffeners, let us use $2'' \times 2''$ 3.1# angles. Four of them at each end of the beam will be needed to take up the compression produced by the stress in the beam hangers, leaving a space between the inner angles equal to about fourteen feet. The ratio of thickness of web to depth of same is $\dfrac{1}{4 \times 27} = \dfrac{1}{108}$. Referring to p. 19, we find, by interpolating, that the distance between stiffeners should be 1.65 times the depth, or about $44\frac{1}{2}''$. The number of spaces between stiffeners in the fourteen feet will be $\dfrac{14 \times 12}{44.5} = 4$, requiring six stiffeners, three on each side of the web. The filling plates will have to be $\frac{5}{16}'' \times 2'' \times 22\frac{1}{2}''$.

The method of finding the number and distribution of the rivets in the flanges will be treated in Chapter XIII.: for the present, it will be sufficiently accurate to assume that the average spacing is two inches and a half.

We are now ready to pass to the bill of iron for the beam, the list of details for which is given on p. 30.

BILL OF IRON.*

Web	1	$\frac{1}{4}''$	$27''$	$16'$	360#
Upper flange . .	2	$2\frac{1}{2}'' \times 3''$	4.4# L	$16'$	141 "
Lower flange . . .	2	$2'' \times 3''$	5# L	$16'$	160 "
Stiffening angles . .	14	$2'' \times 2''$	3.1# L	$26\frac{1}{2}''$	96 "
Filling plates . . .	14	$\frac{5}{16}''$	$2''$	$22\frac{1}{2}''$	55 "
Rivet heads . . .	220	pairs	@	0.16#	35 "
Total weight of beam					847#

Dividing 847 by 16 gives 53 pounds as the weight per foot of the floor beam.

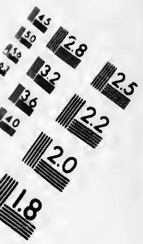
Referring to Table XIX., we find that the beam there given agrees with the one just designed in every respect, except that the weight is a pound and a half per foot greater. This is owing to the fact that the weights in the tables of floor beams were made large enough to cover a slight variation in the designing.

There is no need for proportioning rolled beams, because in Carnegie's "Pocket-Companion," pp. 33–44, are given the working-loads for all the beams rolled at the Union Iron Mills. These loads are directly applicable to bridges of Classes B and C. For bridges of Class A, multiply the calculated load upon the required beam by six (6), and divide by five (5), then search in the "Companion" for a beam to sustain the resulting load.

Plate girders for short spans are to be designed according to exactly the same principles as those laid down for the designing of floor beams. The details, too, are the same, except that there should be two inclined stiffening angles at each end of the beam, one on each side of the web, their lower ends resting over the edge of the bed plate nearest to the centre of the span, as shown in Plate II., Fig. 17.

The distance apart of plate girders should not exceed four-

* The method for preparing this table is explained in Chapter XIV.



teen (14) feet, on account of the difficulty in obtaining joists large enough to support the concentrated wagon-loads.

Trussed beams are sometimes made with one trussing-post, and sometimes with two. To determine the relative length of the part between the posts in the latter case, —

Let

l_1 = length of an end division,

l_2 = length of the central division,

and

$l = 2\,l_1 + l_2$ = length of beam between centres of supports.

The whole beam is now divided into three beams, two of which may be considered fixed at one end, and supported at the other, and the third fixed at both ends. If the moments of the loads do not balance each other over the posts, the rigidity of the connection there may be considered sufficient to insure fixedness.

The greatest moment for a beam fixed at one end, supported at the other, and subjected to a uniform load of w tons per lineal foot, is

$$\tfrac{1}{8}wl_1^2 \text{ at the fixed end.}$$

The greatest moment for a beam fixed at both ends is

$$\tfrac{1}{12}wl_2^2 \text{ at either end.}$$

Remembering the assumption of the fixedness of the beam over the posts, it is evident, that, in order to make the moments over these points a minimum, the two values found should be made equal to each other, so that

$$\tfrac{1}{8}wl_1^2 = \tfrac{1}{12}wl_2^2,$$

or

$$l_2^2 = \tfrac{3}{2}l_1^2,$$

and

$$l_2 = 1.224l_1.$$

Again:

$$l = 2l_1 + l_2 = 3.224l_1:$$

therefore

$$l_1 = 0.31l,$$

and

$$l_2 = l(1 - 2 \times 0.31) = 0.38l,$$

or the length of the central portion should be about four-tenths of that of the beam between supports.

To proportion the upper flange of a trussed beam having two trussing-posts, such as shown in Plate II., Fig. 16, —

Let

d = depth of beam proper,

D = depth between centre line of beam proper and centre line of bottom chord of trussing,

w = uniform load per foot on beam,

$P = \frac{1}{2}w(l_1 + l_2)$ = load concentrated over one post,

and let

l, l_1, and l_2 have the same values as before.

The area of the compression flange of the beam necessary to resist bending only is given by the formula

$$A = \frac{wl_2^2}{12dC} - \frac{1}{6}A',\text{*}$$

where C is the intensity of working-stress upon the flange, to be taken from the "General Specifications," p. 13, and A' is the area of the web.

The stress in either chord of the truss is

$$F = \frac{Pl_1}{D}.$$

Let A'' equal the area of one flange of the beam, supposing that the loads P were really concentrated over the posts, instead of being distributed, then

$$A' + 2A'' = \text{area of ideal beam,}$$

and

$$C'(A' + 2A'') = F = \frac{Pl_1}{D};$$

$$A'' = \frac{1}{2}\left(\frac{Pl_1}{C'D} - A'\right),$$

where C' is the intensity of working resistance to compression.

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* For proof of this formula, see Appendix II., which gives the demonstration for a similar case.

It should be about three (3) tons for bridges of Class A, and four (4) tons for those of Classes B and C. The total area of the lower flange of the beam should therefore be

$$A + A'' = \frac{wl_2^2}{12dC} - \tfrac{1}{6}A' + \tfrac{1}{2}\left(\frac{Pl_1}{C'D} - A'\right) = \frac{wl_2^2}{12dC} + \frac{Pl_1}{2C'D} - \tfrac{2}{3}A'.$$

If the beam be a rolled one, as it nearly always is, there is no need of figuring upon the size of the upper flange; while, if it were a built beam, it might be as well, for practical reasons, to make the flanges of the same size, although theoretically a slight reduction in the area of the upper one would be permissible.

For a beam with a single trussing-post, the bending-moment over the post is

$$\frac{wl_1^2}{8},$$

and the area of flange necessary to resist bending is, as before,

$$A = \frac{wl_1^2}{8dC} - \tfrac{1}{6}A'.$$

P, in this case, is equal to wl_1; making the re-action at each end of the beam, under the supposition of concentrated loading,

$$\tfrac{1}{2}wl_1.$$

The direct compression on the upper chord of the trussed beam is, therefore,

$$\tfrac{1}{2}\frac{wl_1^2}{D} = C'(A' + 2A''):$$

$$\therefore \qquad A'' = \tfrac{1}{2}\left(\frac{wl_1^2}{2C'D} - A'\right),$$

and the total area of the flange is

$$A + A'' = \frac{wl_1^2}{8dC} - \tfrac{1}{6}A' + \tfrac{1}{2}\left(\frac{wl_1^2}{2C'D} - A'\right)$$

$$= \frac{wl_1^2}{8dDCC'}(C'D + 2Cd) - \tfrac{2}{3}A'.$$

The author does not claim that these formulas are exact; but practically they will prove to be a great deal more useful than others theoretically more correct, but also much more complex.

At the end of Chapter XIII., there is given a complete design for a trussed floor beam with two posts. The reason why it is not inserted here is, that it is necessary to understand the contents of Chapters X.–XIII. inclusive, in order to properly proportion the details.

The weight supported by the four hangers that usually sustain a beam is that of a panel live load upon both trusses, that of the lumber in one panel, and that of the beam itself. The total load divided by eight times the intensity of working-stress will give the area of the section of a hanger.

Square sections lie more closely to the pins than round ones, and take up less room in the packing; but they must always be upset, which, in short hangers, makes them more expensive than round ones.

Single beam hangers are allowable in skew bridges, where, indeed, their use is often unavoidable, or in narrow bridges with short panels, where there is not much weight to be supported.

Tables XXII., XXIII., and XXIV. give the sizes of beam hangers for nearly all bridges without sidewalks.

The most simple manner of finding the size of single beam hangers for any roadway and panel length is to look in the table of hip verticals of the same class for the section required, and multiply it by one-half of the ratio of working-stresses for hip verticals and beam hangers: the result will be the area in square inches of the section of the hanger.

If the floor and joists be of oak, the tables of floor beams and beam hangers can still be employed by supposing an increase of one foot in the panel length.

CHAPTER X.

THEORY OF PIN PROPORTIONING.

THE subject of "bridge pins" is one deserving of more consideration than has been accorded it by engineers, and authors of technical works. Until 1873, when Mr. Charles Bender, C.E., presented his paper on "Proportions of Pins used in Bridges" to the American Society of Civil Engineers, very little was known concerning it; the usual custom among engineers when proportioning pins having been to allow one square inch of pin area for every eight or ten thousand pounds of shear in the section most subject to shearing-stress. As Mr. Bender states generally, and as will be shown farther on to be true for iron bridges, it is not the shear, but the bending-moment, which causes the greatest tendency to rupture; so that in any iron structure it will be sufficient, in finding the sizes of pins, to calculate the greatest moment induced in them by the various members coupled thereon, and to proportion accordingly, due regard being paid to the stresses in the eye-bar heads. Before making any investigations, it will be well to review and summarize the most important results of the investigations of others in this subject.

The principal conclusions arrived at by Mr. Bender are, that, for a well-fitting pin of large diameter, a pressure on the bearing-surface of six tons per square inch is not too large; that for simplicity it is well to assume that this pressure is uniformly distributed over the diameter of the pin; that wrought-iron, after millions of impacts, may break on the side where the stress is tensile, but never on the side where it is compressive, the ultimate resistance to crushing being about thirty tons per square inch; that the shearing-stress at the centre of a pin is one and three-eighths times the average shear on the whole section; that in iron and steel the ratio between the greatest allowable tensile and the greatest allowable shearing-stresses should be as 5 to 4, which would make the uniformly distributed shear 2.91 tons per square inch, to correspond with a tensile stress of 5 tons per square inch; and that, owing to various considerations, iron in pins may be strained much more than similar iron in tension members.

Mr. B. Baker, C.E., in "Beams, Columns, and Arches," treats of pins merely incidentally. He finds, that, for iron in solid circular beams, the average value of ϕ is $\frac{11}{16}f$, where f is the ultimate resistance per square inch to rupture by tension, and ϕ the difference between the apparent ultimate resistance per square inch to rupture by bending and f, according to the equation $F = f + \phi$, F being the apparent ultimate resistance per square inch of the extreme fibre which first gives way; and, that for steel, the value of ϕ varies between $1.7f$ and $1.9f$.

Professor Burr devotes five pages of his work on "Stresses in Bridge and Roof Trusses" to the subject of pins, and illustrates the particular case of a suspension-bridge cable pin, and a general case for ordinary truss-bridge pins.

Professor Du Bois, in "Strains in Framed Structures," also gives a mathematical discussion of how to find the maximum bending-moment.

Table XII. gives the working bending-moments on all the iron and steel pins, and the working-shear on all the steel pins, which will ever be required for highway-bridges. Having calculated the bending-moment, the requisite diameter for the pin can be found by looking down the column for the class of bridge considered, until a bending-moment at least equal to the one found is reached. The diameter will be found at either end of the horizontal row thus located. The use of the columns for shear will be made apparent presently.

The upper and lower horizontal lines in the tables of bearings (Tables XXVI. and XXVII.) give the diameters of the pins; the extreme vertical lines, the necessary widths of bearing-surface at each end of the pins, including both channel and reenforcing plates; and the other vertical lines, the permissible pressure, on the bearings. The method of using these tables is the following. The pressure which the pin is to carry is to be taken from the diagram of stresses. A trial diameter is then assumed. The vertical column in either Table XXVI. or Table XXVII., headed by this diameter, is to be followed down, until a number nearest the pressure to be carried is found. At either end of the horizontal row thus located will be found the proper width of bearing. Knowing the width of bearing, diameter and pressure, the moment to which the pin is subjected may be at once calculated. Turn, then, to Table XII, and see if this moment agree with the working-moment corresponding to the trial diameter. If it does, all right: if not, another trial is to be made, with a new assumed diameter. After a little experience, the first trial will be sufficient. A consideration of other details, such as widths and depths of eye bars, etc., will frequently aid very much in these trials.

To find the least value of the ratio of the diameter of pin to depth of eye bar in an iron bridge, by considering the tension in the bar, and the pressure between the pin and bar, —

Let

$w =$ width of bar,
$d_1 =$ depth of bar,
$d =$ diameter of pin,
$C =$ intensity of working compressive stress,
$T =$ intensity of working tensile stress;

then

$$wd_1T = \text{tension in bar,}$$

and

$$wdC = \text{compression on pin and eye.}$$

These, of course, are equal; and, as $C = 6$ tons when $T = 5$ tons, there results the equation,

$$d = \tfrac{5}{6}d_1 = 0.833d_1,$$

which shows that the diameter of the pin should never be less than eighty-three per cent of the depth of the bar. It is possible, though, that good iron of twenty-five tons tensile strength will resist more than thirty tons per square inch in compression: consequently d may be taken at $0.8d_1$ as a matter of convenience.

To find the proportion between width and depth of bars for the smallest allowable pin in an iron bridge,—

Let the notation be as before, and first let us suppose that there be but one pair of bars acting at each end of the pin, and that the total tension be a fixed quantity. The stress in one bar is wd_1T, and its moment is w^2d_1T. This must be equal to the resisting-moment of the pin, which is given by the well-known equation

$$M = \frac{RI}{D}.$$

Here $R = \frac{3}{2}T$, $I = \frac{1}{4}\pi r^4$, and $D = r = \frac{d}{2}$, substituting which gives

$$M = \tfrac{3}{64}\pi T d^3.$$

Equating the two values of the moments gives

$$w^2d_1T = \tfrac{3}{64}\pi T d^3,$$

or

$$w^2 = \frac{3\pi}{64} \cdot \frac{d^3}{d_1}.$$

Now, to make the diameter of the pin as small as possible, the moment of the stress must be made as small as possible; and, as the stress is constant, the lever-arm w must be made as small as possible. But the product of w and d_1 is a constant: so when w is smallest, d_1 must be greatest. But the greatest value of d_1 is $\frac{5}{4}d$; substituting which gives

$$w^2 = \frac{3\pi}{64} \times \frac{64}{125}d_1^2 = 0.754d_1^2,$$

and

$$w = 0.274d_1,$$

or about one-fourth of the depth of the bars.

If there be two pairs of similar bars acting at each end of the pin, instead of one pair, the equation of moments will be

$$2w^2d_1T = \tfrac{3}{64}\pi T d^3,$$

or

$$w^2 = \frac{3\pi}{128} \cdot \frac{d^3}{d_1}.$$

As before, to make d a minimum, w must be made a minimum, or d_1 a maximum: therefore $d = \frac{4}{5}d_1$, which, substituted, gives

$$w = 0.194d_1,$$

or about one-fifth of the depth of the bars.

For three pairs of similar bars at each end of the pin, the equation of moments will be

$$3w^2d_1T = \frac{3}{64}\pi Td^3,$$

substituting in which $\frac{4}{5}d_1$ for d gives

$$w = 0.159d_1,$$

or about one-sixth of the depth of the bars.

Finally, if there be four pairs of similar bars at each end of the pin, the equation of moments will be

$$4w^2d_1T = \frac{3}{64}\pi Td^3,$$

which gives

$$w = 0.137d_1,$$

or about one-seventh of the depth of the bars.

To find the greatest working shearing-stress (supposed to be uniformly distributed) in terms of the working resistance to tension, —

Let S = actual varying resistance to shearing, considered uniformly distributed. The greatest value of S will correspond to a value of w equal to $0.274d_1$; for suppose the moment to remain at its maximum value, and the dimensions of the bar to vary (consequently the stress therein also), the tension in the bar will be greatest for the value of w corresponding with the greatest value of d_1: therefore the shear will also be greatest for that value.

Equating the tension to the shear gives

$$wd_1T = \frac{\pi d^2 S}{4}.$$

Substituting $\frac{5}{4}d$ for d_1, and $0.274(\frac{5}{4}d)$ for w, gives

$$0.274(\tfrac{5}{4}d)^2T = \frac{\pi d^2 S}{4},$$

and

$$S = 0.545\,T;$$

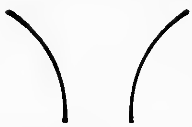
for $T = 5$ tons, $S = 2.725$ tons. But the greatest allowable value for S is, according to Bender, 2.91 tons. This proves, that, if an iron pin be properly proportioned for crushing and bending, it will be strong enough to resist shear, and in fact, that, before the pin could shear, it would either break by bending or crushing, or the eye of the bar would give way. A similar investigation for steel bridges, where $T = 8.35$ tons, $C = \frac{7}{4}T$, and R (the intensity of working bending-stress) $= 1.8T$, gives $d = 0.5714d_1$, $w = 0.1816d_1$, and $S = 5.912$ tons = the actual intensity of shearing-stress when the pin is strained up to the bending-limit, and the ratio $\frac{w}{d_1}$ for that condition of stress is at its minimum, and consequently the area of the bar, the tension therein, and the shear on the pin, at their maxima. But the greatest allowable shear is, according to Bender, $\frac{4}{5} \times \frac{8}{11} \times T = \frac{32}{55} \times 8.35 = 4.858$ tons; so that, for a pair of steel bars pulling on a steel pin in opposite directions, or a single steel bar against a steel bearing, the pin in certain cases will be liable to rupture by shearing, and will therefore have to be proportioned to resist that stress.

After making out the diagram of stresses, and proportioning the main members of a bridge, comes the determination of the sizes of the pins, — a matter that is liable to occupy more time than did all the previous work. Knowing the sizes of all the bars in the structure, the clear width between the inner faces of the top chord channels (and consequently that between post channels) can be found, after which the arrangement of all the bars in the bridge can be decided on. Care must be taken in performing the latter, that no two consecutive chord bars or ties coupled on the same pin pull in the same direction, unless this arrangement reduce the bending-moments, as it can sometimes be made to do; that the lighter set of bars be so placed as to reduce the bending-moments; and that the diagonal ties be placed close to the posts, and the beam hangers close to the ties. Especial care is needed at the panel point where the number of chord bars is different in the consecutive panels. It is possible to arrange the bars there, so that there will be an extremely large moment produced, or so that it will be smaller than at any other panel point of the bottom chord. The neglect of any of these precautions will cause an undue bending-moment on the pin.

The arrangement completed, the next questions to be decided are, first, under what condition of loading will each pin take its greatest bending-moment, and, second, at what point on the pin will this be found. In large bridges, and in many well-proportioned small ones, the bottom chord pins are subjected to their greatest bending-moments when the bridge is fully loaded. Under this condition, the stresses in the chord bars can be taken from the diagram of stresses; but those in the main diagonals must be calculated for the load covering the whole bridge, and their horizontal and vertical components be ascertained.

After having had some practice, one will very often be able by simple inspection to decide at what place the greatest moment of flexure will exist; but, if not, it will be necessary to calculate the values of both horizontal and vertical moments at different points, and find where their combined result is a maximum. As Professor Burr shows, the actual moment is represented by the diagonal of a rectangle whose sides represent the vertical and horizontal moments. It is usually more convenient to square the component moments, add the results, and extract the square root of the sum, than to make out a diagram.

The moments of the stresses can be easily recorded by drawing two curved lines, as shown in the accompanying diagram,



representing the directions in which the stresses tend to bend the pin, and writing each moment as calculated under one or other of them, according to whether it would produce positive or negative rotation. The difference between the sums of each column will give the actual horizontal or vertical moment, as the case may be. The condition that a load covering the whole bridge may not produce the greatest moment in the bottom chord pins is either when there is a single counter coupled at the centre of the pin, or a main diagonal coupled at a distance from the member that takes up its stress. As a rule, single counters and single beam hangers are to be avoided, on account of the unnecessarily large bending-moments they produce. The size of pin for the hip joint depends greatly upon the arrangement of the bars which it couples. In a double-intersection bridge, where there are two hip verticals, two long diagonals, and two short ones, the best arrangement is to put one pair of diagonals on the outside of the chord, and the other pair inside, close to the bearing; the verticals coming next, and being kept apart by a filler. Sometimes it is not advisable to couple outside of the chord, in which case the moment would become so great, that it would necessitate the employment of a pin whose diameter would make the heads of the eye bars too large for the space allotted them. In such a case, a steel pin can be used to advantage. Hinged ends at the hip joints require large pins, for the entire stresses in both chords and batter braces come upon them with great leverage, due to the necessarily large bearing-surface. Such a connection is not advantageous: it is better to allow the channels to abut. Such hinged ends are a great convenience in erection, but usually necessitate an increase in the sizes of the batter braces and the top chords at the end panels. A detail to obviate this necessity will be given in Chapter XIII.

It is not necessary to consider the bending-effect of the stresses in the lateral rods upon the chord pins, for the wind and the live load are not supposed to act simultaneously.

Lateral rods should always be so connected to the chord pins, that the effect of the stress in the outer one will be to diminish the horizontal component of the moment on the pin; i.e., if the tendency of the chord and web stresses is to bend the pin convex to the middle of the bridge, the outer lateral rod should point towards the middle; but, if it be to bend the pin concave to the middle of the bridge, the outer lateral rod should point towards the nearest end of the span.

The ends of pins have to be reduced in diameter, so that the nuts and pin pilots may be screwed thereon. Care must therefore be taken in proportioning small pins to see that sufficient area be left under the root of the thread to resist the tension on that section caused by the greatest transverse components of the stresses in the lateral rods. The principal objection to the use of large pins is not always the undue weight of the pins themselves, but the increased size of the chord and tie-bar heads, and the room that they take up.

On the other hand, it is not always desirable to use the smallest possible pin, as the width of the bearing is an inverse function of the diameter of the pin: so if, owing to the necessity of a large number of rivets, the re-enforcing plates be long, it might be economical to increase the diameter so as to reduce the width. Thickening the heads of eye bars has an injurious effect on the pins, although a beneficial one upon the heads, for the lever arms of the stresses are thereby increased.

Bridges with weak pins will not necessarily fail by the rupture of the pins. The reason for this is thus stated by Professor Burr: "The distortion of the pin beyond the elastic limit will relieve the outside eye bars of a large portion (in some cases, perhaps all) of the stress in them. This result will produce a redistribution of stress in the eye bars, by which some will be understrained, and the others correspondingly overstrained. Thus, although the pin may not wholly fail, the safety of the joint will be sacrificed by the overstrained metal in the eye bars."

CHAPTER XI.

PRACTICAL METHOD OF PIN PROPORTIONING.

THE ordinary method of pin proportioning is to figure the diameters of a few principal pins, and to make the others of the same sizes. Thus, by inspection, can be found which pin near the middle of the bottom chord is subjected to the greatest bending-moment. If there be an even number of panels in the span, it will be the middle pin; but, if there be an odd number, it may be the first or second pin from the middle, according to the number and arrangement of the chord bars. The vertical component of the bending-moment on any one of these pins is so small in comparison with the horizontal component, that it may be neglected. For bridges with an even number of panels, —

Let

T = tension in middle panels of lower chord,

and

w = the *average* thickness of chord bars in these panels;

then, approximately,

$$\frac{Tw}{2} = \text{bending-moment on middle pin.}$$

This formula may be applied, but perhaps with less accuracy, to a bridge having an odd number of panels; and, if the chord be properly packed, the error will be upon the side of safety.

With the exception of the chord pins at the shoes and at the first panel points from the ends of the span, all the lower chord pins may have a diameter corresponding to this maximum bending-moment.

To find the size of the lower chord pin at the first panel point, use the formula,

$$H = \frac{Tw}{2}$$

for the horizontal component of the moment, and the formula

$$V = \frac{tA(d + d')}{4}$$

for the vertical component; t being the intensity of working-stress for the hip verticals, A their area (S. R.), to be taken from one of Tables VI., VII., and VIII, d the diameter or thickness of a hip vertical, and d' that of a beam hanger.

The moment given by the formula

$$M = \sqrt{H^2 + V^2}$$

applied to Table XII. will determine the diameter required. This diameter is to be used also for the pin at the shoe.

To find the size of a hip pin, lay off the stresses in one hip vertical and one end main diagonal to any convenient scale, and find the value of their resultant by the parallelogram of forces. This resultant will determine the thickness of the bearing, a trial diameter being first assumed. It is possible that this bearing will have to be increased, so that there will be enough iron to transfer the stresses from the batter brace, hip verticals, and diagonals to the chord, as will be explained in Chapter XIII. An approximate test of the sufficiency of the bearing in this respect may be obtained as follows:—

Let

A = the area of the section of the end panel of the top chord,
d = depth of chord channels,
t = thickness of web of an end chord channel;

then the bearing should not be less than that given by the formula

$$B = \frac{A}{2d} + t.$$

Next find the distance l between the centre of the bearing of the chord and that of the diagonal, also the distance l' between the former and that of the hip vertical, the latter being on the inside. Calling the stress in the hip vertical F, and that in the diagonal S, the vertical moment will be Fl', and the inclined one Sl. Next lay out these components to any convenient scale in their proper directions, and find their resultant by the parallelogram of moments. This resultant will determine the diameter of the pin.

If the diameter found agrees with the one assumed, or if it does not agree, provided that the bearing was not determined by the trial diameter, all right; but if the bearing were so determined, and the two diameters do not agree, another trial must be made.

Where there are more than two main diagonals coupled at the hip, as is the case in double-intersection and in very heavy single-intersection bridges, one pair is coupled on the outside of the bearing, and the other on the inside; so that theoretically the greatest bending-moment is equal to the stress in the outer bar multiplied by the distance between the centre of the bar and the centre of the bearing. But practically the moment may be greater, for the distribution of stresses among the diagonals may not be as assumed: so it is well to determine the moment by imagining the outer bar not to exist, and proceeding as explained above for the case of only two main diagonals at the hip, excepting, of course, that the thickness of the bearing must be ascertained by finding the resultant of the stresses in the two diagonals and the hip vertical.

To calculate the size of an intermediate upper chord pin, the widths of chord and post bearings are to be determined as shown in Chapter XIII. The former is given approximately by the last formula, where A is the section of the panel of the chord on the side of the pin towards the middle of the bridge, and t the thickness of the corresponding channel. The other is given by the formula

$$B = \frac{A_1}{h},$$

where A_1 is the area of the section of the post, and h the depth of one of its channels.

Next resolve one-half of the diagonal stress vertically and horizontally into P and P'' respectively. Let l represent the distance between the centre of the diagonal and that of the extension plate, and l' the distance between the former and that of the chord-bearing ; then

$$V = Pl,$$

$$H = P'l',$$

and

$$M = \sqrt{H^2 + V^2}.$$

If the bridge be a small one, it will be necessary to calculate only the size of the pin at the top of the first vertical post from the end of the bridge, and to make all the intermediate top chord pins of the same size. But, if the bridge be a large one, it will be better to calculate the diameter of the pin on the post midway between the end vertical post and the middle of the span, and to make all the pins between these places of this diameter, and all the others of the same diameter as that at the end of the first vertical post. After the diameters of the top chord pins are determined, the post and chord bearings should be tested by applying one of Tables XXVI. and XXVII., although in most cases they will be found ample.

In double-intersection bridges, where the diagonals are halved, and coupled on pins passing through the middle of the posts, the size of any one of these pins may be found from the moment

$$M = \frac{Sw}{2},$$

where S is the stress on the diagonals as given on the diagram of stresses, and w the width of one of the main diagonals.

In all pin proportioning it must be kept in mind that the diameter of the pin is never to be less than eight-tenths of the depth of the deepest bar coupled thereon.

The author wishes to call attention to the superiority (in his opinion) of the simple method given in this chapter for proportioning lower chord pins by formula over the apparently more accurate one given in the last chapter.

In this method, when the proper proportion of width to depth of bars is adhered to, the diameter of the pins will be almost eight-tenths of the depth of the bars, and will be great enough to resist the bending-moments produced by any legitimate method of packing. Moreover, after the diameters of the pins have been determined, the chord can be packed, if it be advisable, so as to reduce the bending-moments. This superabundance of strength in the pins is obtained at the expense of a slight increase in the weight of iron; and the increased sizes of heads for diagonals can do no harm, because they do not enter any limited space, as do the heads at their other ends.

But if, by a skilful arrangement of the packing, we can so reduce the bending-moments on the pins, that the diameters may be made small, and the proportion of width to depth of bars larger than that found in the last chapter, the pins may not be as strong as we imagine them; for we cannot be sure that all the bars are going to pull as we have assumed that they will. It may be that one of the outer bars is a trifle long, and will not pull at all until the others are well stretched: what, then, becomes of our calculated bending-moments?

Any one of them may be so greatly exceeded, that the pin will be strained beyond the elastic limit, and will bend perceptibly, so changing the distribution of stress in the panel that one or more of the bars also may be strained beyond the elastic limit.

But if the pin be large enough, or more than large enough, it cannot bend perceptibly: consequently the distribution of stress will be much more uniform, even if the bars be of slightly unequal lengths.

CHAPTER XII.

RIVETING.

THE subject of riveting is one, which, like that of pin proportioning, has never received its due amount of attention from bridge designers. Many structures otherwise very strong are extremely weak in detail, owing to the insufficient number of rivets employed in the connections and to their improper arrangement. The principal rules for riveting have been given in Chapter II., pp. 17, 18.

Rivets should be proportioned for bending and for bearing pressure; i.e., for any given connection, the number of rivets necessary to resist properly each of these stresses should be determined, and the greater number chosen.

Tables XXXVI. and XXXVII. give the working bending-moments and permissible bearing-pressures for bridges of Class A and for those of Classes B and C. For the lateral systems of both classes, Table XXXVII. is to be used. In these tables the first and second horizontal lines of vulgar fractions and decimals give the widths of bearings; and the other horizontal lines in the portions pertaining to bearing give the working bearing-stresses for rivets of different diameters. The rest of the tables needs no explanation.

The sizes of rivets ordinarily employed for highway-bridges are from five-eighths to three-quarters of an inch; though half-inch rivets are used for very light channels, and seven-eighths inch rivets for very heavy ones.

The weight of a pair of rivet heads for any diameter can be found in Table XXIX. It is well to memorize these weights.

Where two plates are riveted together, the rivets, driven when hot, contract, or tend to contract, in length when cooled, thus drawing the plates together, and producing a friction, which it is necessary to overcome before shear can come upon the rivets. Whether this friction will continue indefinitely is doubtful, for rivets occasionally become loosened when the structure is subjected to oft-repeated loads: so it is not legitimate to depend upon the friction in order to reduce the number of rivets. Perhaps it is on account of this factor that rivets are seldom, if ever, proportioned to resist the bending-moments that come upon them, notwithstanding the fact that it is this last consideration, which, in most cases, should determine the number of rivets to be employed.

Again: if the friction were to be depended upon, it would be only right to allow for the initial tension on the rivets, which tension is sometimes great enough to force off the heads.

It will probably have been noticed by the reader, that shearing-stress upon rivets has been omitted altogether from consideration. The author would hesitate before making the broad assertion that rivets cannot shear, although it is probable that bending is the stress which ruptures rivets that are generally considered sheared. This much, though, he will state as the result of both theoretical investigation and many practical cases of designing, *that, when rivets are proportioned for bending and bearing, they will have more than sufficient strength to resist shear.* Sharp edges on rivet holes will certainly cut the rivets, but this is not shear proper; and it may be possible that there is a certain kind of fixedness about a well-driven rivet which will make the bending-moment less than its calculated value.

Should the reader wish to verify the statement concerning bending and shearing stresses, he can do so by using an intensity of shearing-stress of three tons for bridges of Class A, and one of three tons and three-quarters for those of Classes B and C. The theoretical proof is identical with the one for pins given in Chapter X.

"Countersinking" is a term used to denote the sinking of rivet heads into the plate so as to make them flush with its surface. The least allowable depth for the countersinking is a quarter of an inch, and the least thickness of plate used for this purpose should be three-eighths of an inch: for rivets exceeding three-quarters of an inch in diameter, these dimensions should be increased by an eighth of an inch. Rivets may be countersunk at one or both ends.

Making parallel rows of rivets staggered avoids unnecessary weakening of the parts riveted together.

There has been much discussion as to whether punched or drilled holes are preferable; the general conclusion being, that drilled holes weaken the plates less, and when slightly countersunk, so as to avoid sharp edges, do not increase the shear upon the rivets, but that punched holes are so much more economical as regards shop-work, that, when properly made, they are preferable to drilled ones. The improvements made of late years in riveting-machines have increased the efficiency of work with punched rivet holes.

Should, for any reason, it ever be necessary, in bridge designing, to put a rivet through a plate whose thickness is greater than the diameter of the rivet, the rivet hole should be drilled.

Machine riveting is preferable to hand riveting, but there are cases when the latter has to be employed.

Field riveting is nearly always inferior to shop riveting.

When a stress is transmitted from one plate, through one or more plates, to another plate, the number of rivets must be increased. The rule given by Weyrauch is, that, "for every single shear connection, the indirect force transferrence requires for m intermediate plates $m + 1$ times as many rivets as for direct transferrence." Keeping this in view, the designer will avoid using more than one flange plate in floor beams, or more than one plate for covering the channels of the top chord.

CHAPTER XIII.

PROPORTIONING OF OTHER DETAILS.

THE sizes of stay plates used at the ends of systems of latticing or double-riveted lacing are given in Table XXXII., and the sizes of those used at the ends of systems of single riveted lacing, in Table XXXIII. The headings of these tables fully explain their use.

Stay plates are to be employed at the middle of posts (*vide* Plate II., Fig. 15) when the diagonals are halved, and connected by pins passing through the posts; their sizes being taken from the before-mentioned tables. Stay plates, if they can be so called, are also to be used on the lower portal struts, for the purpose of attaching the knee braces.

Pin bearings are sometimes figured, counting in both re-enforcing plates and web; but the latter is often omitted. This would be necessary when the holes in the web are bored independently of those in the re-enforcing plates, for then it is very improbable that the different holes will coincide; but, when the re-enforcing plates are riveted to the web before boring, such a precaution is not only unnecessary, but is a waste of material.

By consulting Table XXVIII. can be found at a glance, accurately enough for all practical purposes, the thickness of web of any Union Iron-Mills channel bar, when the weight is given, or *vice versa*.

Where re-enforcing plates act also as splice plates, there should be one on each side of the web in order to insure a good, substantial joint; although the practice in the building of small bridges is to omit the outer plate when the pin bearing does not demand its use.

The length of a simple re enforcing plate depends upon the number of rivets required, and is thus determined. Find, by dividing the stress given on the diagram of stresses between the various thicknesses of iron which constitute the bearings, the amount of stress which the plate considered is to carry. It is well, though, to make a liberal allowance, say twenty per cent, for the possibility that the stress may not be divided proportionately to the thicknesses. Next multiply the stress so obtained by the perpendicular distance between the central plane of the re-enforcing plate and that of the plate or web re-enforced: the product will be the moment of the stress upon the re-enforcing plate Divide this moment by the working bending-moment, taken from Table XXXVI. or XXXVII., for a rivet of the diameter to be employed for the connection: the quotient will be the number of rivets required to resist bending. Next find, from one of the same tables, the working bearing-stress for one of the rivets upon a plate of the thickness of the re-enforced plate or web, and divide it into the stress which the latter carries: the quotient will be the number of rivets required to afford sufficient bearing. The greater of the two numbers thus obtained is the one to be employed. Next make to scale a drawing of the re-enforcing plate, laying out the rivets, if it be possible, symmetrically, and thus determine the length of the re-enforcing plate. In case of a re-enforced pin hole, if the diameter of the hole exceed one-half the width of the plate, it will be necessary to put more rivets in front of the pin hole than behind it; the ratio of the number in front to the whole number being equal to that of the diameter of the hole to the width of the plate.

The method of proportioning splice plates or connecting plates is somewhat similar. For instance, let us take the plates at a joint in the top chord; which joint, for reasons to be stated in Chapter XVIII., is always to be placed a few inches to that side of the pin hole farthest from the middle of the span. The stress on the portions of the plates to this side of the joint is that due to the stress in the panel where the joint occurs; while that on the other portion of the plates is due to the stress in the next panel towards the middle of the span. The number of rivets on each side of the joint will be dependent upon the stresses carried by the channel bars of the two adjacent panels. The simplest way to find the stress on any channel is to multiply its area by the intensity of working-stress, which was found from either Table X. or XI. This stress is then to be divided equally, or otherwise, between the outer and inner plates which splice the abutting channels; and the number of rivets necessary to resist bending and bearing are to be ascertained in the manner explained for re-enforcing plates.

To determine the length of a cover plate, find in the same manner the number of rivets *upon each side of the joint*, which will take up the stress carried by the chord plate, and lay out the cover plate with the rivet spacing to scale. The stress carried by the chord plate is equal to its sectional area multiplied by the intensity previously found for the channels.

At the hip joint it is obvious, that, where the chord and batter brace are hinged upon the pin, the resultant of the thrust in the batter brace and the pulls in the diagonals and verticals must equal the thrust upon the chord, and that the bearing must be figured for this thrust; but, where they are not hinged, the section of the splice plates must answer two requirements: first, their area (neglecting, on account of its being bent, the effect of the cover plate) must be sufficient to transfer to the chord a stress equal to that in the first panel; and, second, that the pin bearing be sufficient for the resultant of the tensions in the diagonals and verticals meeting at the hip. The length of the cover plate at the hip cannot be calculated; for it carries no stress, simply adding to the rigidity of the joint, and keeping the rain therefrom.

If the posts be figured for one fixed end, the inner splice plate of the chord can be extended downward to act as a connecting plate for the post; and in this case there must be enough rivets used in respect to bearing and bending to transfer all the compression in the post to the chord by the connecting-plate, under the supposition that the ends of the post channels do not touch the flanges of the chord channels. If they do touch, so much the better; but it would not be safe to count upon their doing so. The thickness of the connecting-plate should be such that it would not bend between the end of the post and the pin hole when the post would be on the point of rupture by compression. Where the ends of the posts are figured hinged, which is a decidedly better construction, the extension plates pass inside the splice plates of the chord, and are attached to the pins. As before, there must be enough rivets to transfer the stress in the posts to the plate.

The thickness of the re-enforcing plates at the lower end of a post is determined by the bearing required, and their length in the manner already described. It is better to place these plates on the inside of the posts; then, if the flanges of the channels be partially cut away, an extra plate (at least three-eighths of an inch thick) can be placed on the outside of each channel. The reason for cutting away the bottoms of the post channels is merely to pack the chord more closely, and thus reduce the bending-moments on the pins. But, if the method of pin proportioning given in Chapter XI. be adopted, the necessity for cutting away the channels, to any extent, vanishes; for at the middle of the span the web stresses are so small, that their moments are neglected, and the pins at the feet of the other posts have an excess of strength.

In high double intersection truss bridges with long panels, the diagonals become so long, that it is convenient to halve them, and connect the halves by pins. It is then advisable to let these pins pass through the webs of the post channels where the diagonals cross, for the latter then tend to stiffen the posts. If intermediate struts also be used at the middle of the trusses, the posts can be figured for half length, with both ends hinged. On account of the stretch of the main diagonal, there would be a tendency to deflect the post. If the diagonals were forty-two feet long, the stretch of the upper half of them would be about one-eighth of an inch; so that, to avoid this objection, it will be necessary to elongate the pin hole that amount on the lower side, in the direction of the main diagonal. The pin holes should, of course, be well re-enforced in order to compensate for the material cut from the channels. The stretch of the counters being less than that of the main diagonals, and the posts crossed by the heavy ones generally having an excess of strength, it is not necessary to elongate the pin holes in the direction of the length of the counters.

In nearly all iron bridges the batter braces are made with ends fixed at the pedestals (i.e., they are rigidly attached to the shoe plates), although hinged pedestals are not unknown: the advantage gained by their use is the certainty of a uniformly distributed pressure on the rollers, and the disadvantage a great increase in the section of the batter braces.

The shoe plate can be attached to the batter-brace channels by bent plates on the inside, the outside, or both, or by pieces of channels, with one flange removed, placed on the inside, and riveted through their webs to the webs of the batter-brace channels, and through their flanges to the shoe plate, as shown on Plate IV. The lower end of the batter-brace plate should be turned up horizontally, and riveted to the shoe plate.

The area of a section of the connecting channel or plate made by a plane perpendicular to the direction of the batter brace should be equal to the area of one batter-brace channel, or greater if the shoe pin require greater bearing than this would afford; and there should be enough rivets to transfer the stress from the batter-brace channel to the connecting channel or plate. Should the batter-brace channels bear against the shoe plates, as they ought to do, there will be more rivets than necessary; but such a bearing should not be counted upon. Details of shoes are shown on Plates II., III., IV., and VI. The rules for proportioning shoe, roller, and bed plates, are given on p. 16.

A very good connection for the hip joint is the one shown on Plates III. and IV. The inner splice plate has five sides, the under one passing entirely below the joint; and the outer splice plate is cut to fit closely to the webs of the chord and batter-brace channels, being made as wide as the flanges of the channels and the rivet heads therein will permit. The objection to this detail is, that it requires a good deal of field riveting.

Another good detail for this joint is that shown in Fig. 14, Plate II. Here there are two connecting-plates on the outside of the chord, and two on the inside of the batter brace, through all of which the pin passes. Those on the chord abut against plates riveted to the outside of the batter-brace channels; and those on the batter brace abut against plates riveted to the inside of the chord channels, all abutting surfaces being planed to fit exactly; so that, when the pin is driven into place, the whole joint will be as rigid as if it were riveted. Of course this detail demands neat workmanship, and is consequently somewhat expensive; but the satisfactory result attained more than counterbalances the extra cost of the shop-work, and there is no necessity for figuring on a hinged end at the hip when proportioning the batter brace and the end panel of the top chord.

A good method of attaching the upper lateral struts to the chords is the following, which is illustrated on Plates II., IV., and VI. Let the web of the upper channel lie upon the cover-plate of the chord, extending to its outer edge, and be riveted thereto; and let the under face of the lower channel, its flanges being turned downward, lie in the same horizontal plane as the faces of the lower flanges of the top chord channels. The length of the lower channel of the lateral strut should be a couple of inches shorter than the clear roadway of the bridge. The connection is made by a plate in the form of the letter T, the head being riveted to the lower flanges of the inner chord channels, and the stem passing between the flanges of the lower channel of the lateral strut, to the web of which it is to be riveted. The thickness of the T-plate should be five-eighths of an inch, and the re-entrant angles should be rounded off with a radius of an inch and a half or two inches. The width of the stem should be made as great as the distance between the flanges of the lateral strut will permit, and that of the head equal to the width of the flanges of the chord channels.

The number of rivets for either stem or head must be calculated for bending and bearing resistances corresponding to the greatest stress that could ever come upon the channel, which stress is to be calculated by multiplying the area of the channel by the intensity found in Table XI.

A good connection for the intermediate struts to the posts is by means of two bent plates at each end of the strut (*vide* Plates IV. and VI.). One leg of each plate is riveted to the web of the inner channel of the post, and the other to the web of the I-beam, which is placed horizontally. The vibration rods are attached by bolts that pass through the two connecting-plates and the web of the I-beam. The connection at the upper ends of the vibration rod may be similar, if the width of the T connecting-plate be great enough to permit of the passage of a bolt.

At the intermediate strut connection, there should be enough rivets used in respect to bending and bearing to transfer the calculated stress upon the strut to the connecting-plates.

If there be but one portal strut at each end of the span, it may be connected to the batter brace by two large bolts passing through a jaw plate, as shown in Fig. 11, Plate II. These bolts may have square heads placed so near the sides of the jaw that they cannot turn, the nut having to be screwed up on the inside of the batter brace. But, if there be two portal struts at each end of the span, the channels are to be turned around ninety degrees, and brought nearer together; so that it will be better to use exterior bent plates attached to the flanges of the channels, as shown on Plates IV. and VI., in addition to a single large bolt through the jaw.

Concerning the best method of connecting the lower lateral rods, there is much diversity of opinion; although, in ninety-nine cases out of a hundred, they are attached to the floor beams, which are thus made to act as struts for the wind pressure. Some bridge designers put bent eyes on the lateral rods, and run bolts through the web, usually near the middle, which is very objectionable, for two reasons: First, the laterals take hold of the weakest part of the beam; and second, being attached at such a distance from the pins, they permit of too much vibration. Another detail is to rivet two 4 by 6 inch angles to the web, and drop a pin through the six-inch legs: this is a little better detail, but the same objections apply here. Another is to let the rods pass through the webs, and through rods and plates bent so that one face is perpendicular to the direction of the lateral rod, another face parallel to it, and the other two end faces parallel to the web of the beam, to which they are riveted. The same objections apply to this, together with two which are still more important; viz., that, as at each connection there are two such plates and two lateral rods from adjacent panels crossing each other, the longitudinal components of the stresses in the latter produce a moment tending to revolve the beam about the upper edge of the web. Then, again, the bent plates must be made so heavy that they would withstand, before buckling, the ultimate pull of the lateral rods; and it is very seldom that such a detail is made strong enough to stand the ultimate pull of an inch and a half round rod. Another way is to rivet a plate across the top of the beam, and two bent plates or large angles opposite each other, just below the top flange, dropping pins through the jaws thus formed. This is the best arrangement yet employed. But in the author's opinion all these details are defective, for the reason that the lateral rods all take hold of the floor beams, which are simply suspended from the pins that are several inches above them; so that, unless the hangers be screwed up very tightly, any wind stress in the lateral rod will cause a rocking at the point of suspension, and, even if the hangers be screwed up tightly, the *tendency* to rock still exists. The only correct place to attach the lateral rods is to the chord pins, *and their stresses should not be transmitted through the floor beams.* Then come the questions, "How shall they be transferred?" and "How shall the rods be arranged so as to clear the joists?" The detail about to be described will answer these questions.

Upon the floor beam place a stick of square timber (about eight inches for ordinary highway-bridges), and let the ends fit into wrought-iron jaws, which screw up against the chord pins; then fasten the timber every few feet on alternate sides of the web, by half-inch bolts, to the flanges of the beam, and rest the joists on the timber. The laterals can either be attached by bent eyes to the chord pins (which would be preferable if their diameters do not exceed an inch and three-fourths), or by ordinary eyes to vertical pins passing through the wrought-iron jaws. In this way the timber not only acts as a lower lateral strut, but serves to give additional stiffness to the floor beam; although the section of the latter should not be diminished on that account.

Now, what objections can be raised to this method?

Some may say that it is a clumsy contrivance, but that is a matter of taste. Others may suggest that it reduces an iron structure to a combination bridge. Not at all, — no more than the employment of wood for the floor and joists; because, at the same time when the latter are renewed, the wooden struts can be replaced. There is a slight objection for short through-spans, viz., that it reduces the headway; but it would not greatly increase the expense to add eight inches to the depth of the trusses.

Another method of avoiding the difficulty is to rivet the floor beams to the posts. But will not this be equally objectionable? Certainly such a connection is better for the beams, as it partially fixes their ends; but what about the deflecting effects of wind stresses and passing loads upon the posts? The transverse components of the lateral rod stresses act with great leverage, for the beams are always attached above the bottom chords; and the weight of a heavy wagon coming suddenly upon the beam must certainly cause the posts to vibrate transversely to the planes of the trusses, but to what extent, and with what injurious effect upon the posts, it is at present impossible to say. Even if there be but little known concerning this attachment, it is certain that a floor beam should never be riveted to only one of the channels of each post. Such an arrangement would produce indirect stresses of a destructive character: consequently the posts should be turned one-quarter way round in order to let the beam pass between them.

Floor beams in deck bridges may either rest upon the chords, be hung from the chord pins, or be riveted to the posts. In neither case should they be used as lateral struts when the lateral rods are attached to the chord pins, because of the leverage that would be afforded to the lateral stresses to produce distortion.

It is not customary to calculate the thicknesses of beam-hanger plates, for they are usually made from three-fourths of an inch to an inch thick for ordinary highway-bridges; but under certain assumptions their thicknesses can be calculated. If the load on a plate be considered uniformly distributed over the portion between the beam-hanger holes, and if the flange of the beam be supposed to take up no bending-stress, the plate may be considered as a beam supported at the ends, and uniformly loaded. For instance, take the case of a twenty-foot panel and an eighteen-foot clear roadway, the re-action at each end of the beam is about nine tons. Suppose the centres of beam-hanger holes to be situated on the corners of a four-inch square, and the plate to be seven inches square, then the bending-moment is

$$M = \tfrac{1}{8} Wl = \tfrac{1}{8} \times 9 \times 4 = 4.5 \text{ inch tons.}$$

The resisting moment is $\dfrac{RI}{d_1}$, where $R = 5$ tons, $I =$ moment of inertia $= \frac{1}{12}bd^3 = \frac{7}{12}d^3$, and $d_1 = \dfrac{d}{2}$. Equating the moments, substituting, and solving, gives $d =$ about seven-eighths of an inch, a result agreeing with good practice. It is almost needless to say that this method is very approximate; for the plate is greatly stiffened by the rigidity of the flange of the beam, while, on the other hand, no reduction has been made for the beam-hanger holes.

Lacing, or, as it is often improperly termed, single latticing, is about the most common detail for keeping pairs of channel bars in line: nevertheless, it must be inferior to latticing, especially when the lattice bars are riveted together at their intersection. By inspecting Tables XXXII. and XXXIII. it will be seen that a system of lacing-bars with one rivet at each end of a bar requires much larger stay plates at the ends than does a corresponding system of latticing or double-riveted lacing.

The actual sizes of lattice or lacing bars for any strut can be determined only by experiment: it is thought that those given in Tables XXX. and XXXI. are so strong, that the struts on which they are employed would break in the channels rather than in the bars, and yet not so heavy as to cause much unnecessary use of material. It will be seen also in these tables, that the requisite dimensions of latticing and lacing bars depend not only upon the sizes of the channels which they connect, but also upon the distance apart of these channels: this is due to the fact that the bars are subject to compression as well as to tension. The lengths and weights of latticing and lacing bars can be found from Table XXIX. It must not be forgotten that these lengths are to be used for *estimates only;* as they were obtained from a diagram, and not checked by calculation.

The smallest trussing-bars used should be no less than a quarter of an inch by three inches, and the bend for attachment should be no less than three inches long, so as to permit of the use of two staggered rivets. The heavier the trussed bars, and the greater the distance between them, the greater should be the section of the trussing-bars. At the ends of a system of trussing, the bars should be turned and attached, as shown on Plate II., Fig. 8, and on Plate VI.

The lightest bracket used should be no weaker than a $2\frac{1}{2}'' \times 2\frac{1}{2}''$ 4.9# angle iron, which section is to be employed only to attach intermediate struts to posts. Where there is no vertical sway bracing, the stresses on the brackets are to be calculated as shown in Chapter VI., and the sections are to be proportioned by using the following table of approximate intensities of working-stress.

LENGTH OF STRUT, IN FEET.	INTENSITIES OF WORKING-STRESS.		
	$2\frac{1}{2}'' \times 2\frac{1}{2}''$ L.	$3'' \times 3''$ L.	$3\frac{1}{2}'' \times 3\frac{1}{2}''$ L.
4	3.0	3.5	4.0
6	2.5	3.0	3.5
8	2.0	2.5	3.0

The number of rivets that connect the bracket to the lateral strut and post must be sufficient to transfer all the stress in the bracket to each of these members.

To prevent the pedestal at the free end of a span from slipping in the direction of the length of the rollers, the latter can be notched about a quarter of an inch in depth, for a length of about two inches at the middle, and the shoe plate and roller plate be planed down so as to leave projections which will exactly fill the notches. This detail is illustrated in Plate VI.

For short spans, a sliding-joint such as shown on Plate III. is to be used.

When it becomes necessary to anchor down the expanding end of a bridge, it should be done in such a manner that the shoe could not rise more than an eighth of an inch: thus the projection on the under side of the shoe plate will be prevented from being lifted out of the notches on the rollers.

Bed plates and roller plates should be anchored to the abutments by rods with nuts. When the abutments are of stone, a good method of attachment is to drill holes therein just below the anchor bolt holes in the bed plates, enlarging them, if practicable, at the bottom. Split the ends of the anchor bolts several inches, insert small iron wedges in the splits, drive the bolts into place, so that the wedges force the split ends apart, thus partially filling the enlarged bottoms of the holes, and pour in molten sulphur.

In figuring lengths of fillers for pins, a clearance of from a quarter to half an inch should be made, so as to allow for variation in thickness of eye-bar heads, re-enforcing plates, etc.: such an allowance will save a good deal of trouble in erection. When the end lower lateral strut is of such dimensions that it will not fit, without being turned from the vertical between the flanges of the batter-brace channels, filling-rings can be used between the batter-brace webs and the ends of the strut. Such rings will be necessary, if there be four chord bars in the end panel, and the outer ones be not let into the channel flanges far enough to lie against the webs.

In making turn buckles, a little expense can be saved by having only one adjusting-end; the other having a hole, through which passes one end of the rod, which is enlarged into a head. One advantage of this style is, that the turn buckle can never be lost from the rod. Such a turn buckle should always be used on portal vibration rods, for a reason that will be given in Chapter XX.

Jaws are not a very desirable detail, although so convenient that they are often employed. In the first place they have not a pleasing effect to the eye; and in the second, on account of the bent plates, are liable to be weaker than might be estimated. If the flanges of the channels be cut away, as is sometimes unavoidable, the jaw plate, from the cut flanges to the bend, should be able to resist more compression than the rest of the strut. Such a detail occurs often on the ends of the struts which keep the pedestals apart. It is generally difficult to make a satisfactory design for this member, as it interferes with the joists; but, with the lower lateral system previously described, all the difficulty vanishes.

Concerning the proportioning of eye-bar heads, there is a variety of both opinion and practice. Many specifications call for a section at the eye equal to one and a half times that of the bar for welded bars, or one and a third times the same for hammered eyes, not taking into account the effect which the different ratios of diameter of pin to width of bar have upon the strength of the eye. Specifications for the better class of both railroad and highway bridges have of late made this distinction; but there seems to be some uncertainty as to what is the exact effect of each ratio upon the strength. On p. 20 is given a table for sizes of chord heads, prepared from actual experiments by C. Shaler Smith, C.E., who is considered the best American authority upon all matters connected with the designing of bridge superstructures. The subject of chord-head proportioning is further treated in Chapter XVIII.

Bent eyes do not make a very good detail, but are such a convenience that they are often used by good designers. If the diameters of the rods do not exceed one inch and three-fourths, there is no objection to using such eyes. The principal point to be raised against them is because of the eccentric pull which they give upon the pin nut. This objection may be removed by using either extra large nuts, or the detail shown in the upper lateral rod connection of Plates II. and IV., in which the bent eyes pull against a piece of channel riveted to the strut. A still greater improvement is shown on Plate VI., in which a piece of bent plate is substituted for the channel: this permits of more rivets in the connection, and avoids the possibility of having to insert a filling-plate between the channel and the strut.

In connection with this detail, on Plate VI. is another and a rather peculiar one. The plate, which was originally in the form of the letter T, is bent so that the stem may be riveted to the strut channels, and the head may afford a bearing for the vibration-rod pin. This connection is to be used when the lateral strut channels are so small that there is no room for a pin to pass through the connecting T-plate which attaches the lower channel. When, because of their large diameter, the lower lateral rods cannot be attached to the chord pins, but must be connected by vertical pins passing through the lateral strut jaws, they must be made to pull on the middle point of each of the latter pins by using a double eye on one of the rods, with a space between large enough to admit the eye of the other rod. This is to avoid all tendency to rotate the lateral strut about its axis. The rods can be retained in place by fillers above and below.

With this detail, there is a tendency to break the jaw through the pin holes, because of the moment of the longitudinal component of the lateral rod stress: the jaw plate must therefore be made wide enough to properly resist this moment. The easiest way to proportion the plate is to assume its dimensions, and to find its resistance to bending, neglecting the area lost by the pin holes (which area is close to the neutral surface), and making up for the omission by providing a little extra resistance.

To illustrate the method, let us take a two-inch lateral rod, making an angle of forty-five degrees with the planes of the trusses, and let the distance between centres of pin bearings be six inches. The stress on such a rod is $3.14 \times 7.5 = 23.55$ tons, and the bending-moment on the pin is $\frac{1}{2} \times 23.55 \times 3 = 35.3$ inch tons, corresponding (*vide* Table XII.) to a diameter of three inches and a fourth. The distance from the axis of the pin to the centre of the jaw bearing will be about $1\frac{5}{8}'' + 2'' + 1'' + \frac{3}{8}'' = 5''$. The longitudinal component of the stress on the lateral rod is $23.55 \times 0.7 = 16.5$ tons, making the moment on the jaw about $5 \times 16.5 = 82.5$ inch tons. The thickness of the jaw plate should be $\frac{5}{8}''$, and let us assume the width to be $7''$. The resisting-moment is given by the well-known formula,

$$M = \frac{RI}{d_1},$$

where $R = 11.25$ tons, $I = \frac{1}{12}bd^3 = \frac{1}{12} \times \frac{10}{8} \times (7)^3$, and $d_1 = \frac{7}{2}$. Substituting, gives

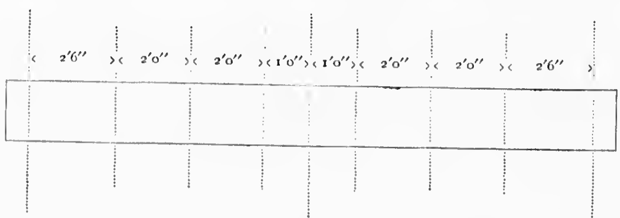
$$M = \frac{11.25 \times \frac{1}{12} \times \frac{10}{8} \times 49 \times 7 \times 2}{7} = 115 \text{ inch tons, nearly.}$$

The difference between 115 and 82.5, or 32.5 inch tons, is greater than the resisting-moment of the material lost by the pin hole: so the dimensions assumed are ample.

U-nuts are objectionable in every case; for, if they are made strong enough to resist without buckling the ultimate pull of the rods, they will have a very clumsy appearance. The insertion of a cast-iron washer will relieve the bending of the U, but not the appearance: besides, it is better not to introduce cast-iron into a wrought-iron structure.

It is now in order to take up the omitted portions of Chapter IX.

First, to find the number and distribution of the rivets in the flanges of the beam there designed, let us divide the fifteen feet between centres of supports, as shown in the accompanying diagram, and calculate the stresses at the points of division.



The re-action at each end is about 8.5 tons, and the uniformly distributed load about 0.0944 ton per lineal inch. The moment at the first point of division from the support is

$$8.5 \times 30 - 0.0944 \times 30 \times 15 = 212.5 \text{ inch tons.}$$

At the next point of division the moment is

$$8.5 \times 54 - 0.0944 \times 54 \times 27 = 321.3 \text{ inch tons,}$$

and at the next point it is

$$8.5 \times 78 - 0.0944 \times 78 \times 39 = 375.8 \text{ inch tons.}$$

From the last equation of Appendix II. we have for the value of the flange stress at any section,

$$S = \frac{M}{D\left(1 + \frac{A'}{6A}\right)}.$$

In this case

$$D\left(1 + \frac{A'}{6A}\right) = 26\left(1 + \frac{\frac{1}{4} \times 27}{6 \times 2.64}\right) = 36.7.$$

Dividing each of the moments by 36.7 gives, for the horizontal stresses at the three points of division, respectively 5.8 tons, 8.74 tons, and 10.24 tons. Therefore, between the centre of the support and first point of division, there must be enough rivets to take up a horizontal stress of 5.8 tons; between the first and second points, enough for a horizontal stress of $8.74 - 5.8 = 2.94$ tons; and between the second and third points, enough for a horizontal stress of $10.24 - 8.74 = 1.5$ tons.

The vertical pressure upon the rivets of the upper flange is about $12 \times 0.0944 = 1.133$ tons per lineal foot, making the total vertical stresses for the three divisions respectively 2.83 tons, 2.27 tons, and 2.27 tons. Combining these by the parallelogram of forces with the horizontal stresses last found, gives the total stresses for each division 6.45 tons, 3.71 tons, and 2.72 tons respectively.

From Table XXXVI. we find the resisting bending-moment of a five-eighth inch rivet to be 0.18 inch ton, and the working bearing-pressure on a quarter-inch plate, 0.938 ton.

Let us first consider the stress of 6.45 tons. It is equally divided between the two angles, making the stress on each 3.22 tons. The lever arm of this last stress is $\frac{1}{2}(\frac{1}{4} + \frac{5}{16}) = \frac{9}{32}''$, and the moment $\frac{9}{32} \times 3.22 = 0.906$ inch-ton, dividing which by 0.18 gives five as the number of rivets required to resist bending. Dividing 6.45 by 0.938 gives seven as the number required for bearing. If there be but seven rivets in two feet and a half, the spacing will be five inches, which would be practically too great. It is better to space the rivets two and a half inches near the ends of the beam; and, if it be thought advisable, the distance may be increased to four or even five inches near the middle.

From the above, we may conclude that calculating rivet spacing for flanges of floor beams is, as a rule, too much refinement for highway-bridge designing.

If the depth of the beam be reduced near the ends, or if, by reason of lack of headway beneath the bridge, shallow beams be used, it might be well to go through the above investigation.

Next let us make the design for a trussed floor beam, taking a twenty-foot panel and a twenty-four foot roadway of a bridge belonging to Class A. Table XIX. gives the weight of an ordinary built beam for these dimensions as ninety-four pounds per lineal foot: so let us assume the weight of the trussed beam to be eighty pounds per foot, also the length of beam between centres of supports to be twenty-five feet. The live load will be

$$\frac{24 \times 20 \times 100}{2000} = 24 \text{ tons.}$$

Table XV. gives 3339 as the number of feet of pine lumber per panel, the weight of which is

$$\frac{3339 \times 5}{2 \times 2000} = 4.174 \text{ tons};$$

and the weight of the beam itself is

$$\frac{26 \times 80}{2000} = 1.04 \text{ tons};$$

making the total load equal to 29.214, or 1.1686 tons per lineal foot. Let us use two posts. The central panel should be ten feet long, and each of the others seven and a half feet. Let us assume the beam to be a 10″ 30# **I**, and the depth of the truss five and a half feet centre to centre. Then in the formula

$$A + A'' = \frac{wl_2^2}{12dC} + \frac{Pl}{2C'D} - \tfrac{2}{3}A'$$

we will have $w = 1.1686$, $l_2 = 10$, $d = \frac{9}{12}$, nearly, $C = 5$, $P = \frac{1}{2} \times 1.1686 \times 17.5 = 10.225$, $l_1 = 7.5$, $C' = 3$, $D = 5.5$, and A' about $8 \times 0\ 32 = 2.56$. Substituting these values gives $A + A'' = 3.21$ as the area of one flange. The total area of the section would then be $2 \times 3.21 + 2.56 = 8.98$ square inches, which corresponds almost exactly with the area of a thirty pound **I**-beam.

The design for the post agrees with that shown in Fig. 16, Plate II., with the exception that the end diagonals are not adjustable. The stress on a post is $P = 10.225$ tons; that on the bottom chord is

$$\frac{Pl_1}{D} = \frac{10.225 \times 7.5}{5.5} = 13.944 \text{ tons};$$

that on the end diagonals is

$$P \sec \theta = 10.225 \times 1.69 = 17.28 \text{ tons};$$

that on the counters is

$$\tfrac{3}{10} P \sec \theta' = 0.3 \times 10.225 \times 2.16 = 6.625 \text{ tons}.$$

The intensity for the tension members should be four tons, making the sections required for the chord bars and main diagonals respectively 3.48 and 4.32 square inches. Referring to Carnegie's "Pocket-Companion," p. 94, we find that two $\frac{5}{8}'' \times 2\frac{3}{4}''$ bars will do for the former, and two $\frac{7}{8}'' \times 2\frac{1}{2}''$ bars for the latter. From Table IX. we find that two one and a quarter inch rods will be required for the counters.

To the stress on a post must be added the vertical component of the initial tension on the counters, which is about

$$2 \times 1.5 \times 0.46 = 1.38 \text{ tons};$$

making the total stress 11.605. Before applying Table XI., we must multiply this stress by about 1.5, the ratio of the factors of safety for wind bracing and floor-beam struts; making the total stress 17.407 tons. Using the column for one fixed and one hinged end, we find that a 6″ 15# **I**-beam will be required.

To find the thickness of the pin plate at the end of the beam, let us assume it at five-eighths of an inch; then the lever arm of the diagonal stress will be $\frac{1}{2}(\frac{7}{8} + \frac{5}{8}) = \frac{3}{4}$ inch, and the moment,

$$\frac{3}{4} \times \frac{17.28}{2} = 6.48 \text{ inch tons}.$$

Consulting Table XII., we find that the necessary diameter of pin is two inches and an eighth. Referring to Table XXVI., and looking down the column for a two and an eighth inch pin, we find that the necessary bearing will be, for 8.64 tons, eleven-sixteenths of an inch. It will be more economical to increase the diameter of the pin to two inches and three-eighths than the thickness of the plate to eleven-sixteenths.

Next let us find the number of rivets necessary to attach the plate to the I-beam. The horizontal and vertical components of the end diagonal stress are respectively

$$17.28 \times 0.8 = 13.82 \text{ tons}$$

and

$$17.28 \times 0.6 = 10.37 \text{ tons.}$$

The first of these stresses produces bending; and the second, direct tension on the rivets. The moment of the first stress is about

$$13.82 \times \tfrac{1}{2}(\tfrac{5}{8} + \tfrac{3}{4}) = 9.5 \text{ inch tons,}$$

which, divided by 0.493, the resisting-moment for a seven-eighths inch rivet, found in Table XXXVI., gives twenty as the number of rivets to resist bending. To resist tension the number required will be

$$\frac{10.37}{5 \times 0.6} = 4,$$

making twenty-four rivets in all for the connection. Seven-eighths inch rivets are rather large for the flanges of a ten-inch beam, as there is not room for full heads: nevertheless, it is better to use them, on account of the increased bending resistance. Using twelve rivets on a side, and spacing them two inches and a half apart, will make the length of the plate about thirty-two inches. It is evident that there is no need of figuring for bearing in this connection.

Next let us proportion the connecting-plate over a post, assuming the thickness to be three-eighths of an inch, and using five-eighths inch rivets. The moment on the rivets will be

$$11.605 \times \tfrac{1}{2}(\tfrac{3}{8} + \tfrac{1}{2}) = 5.08 \text{ inch tons,}$$

which, divided by 0.18 (the resisting-moment of a five-eighths inch rivet), gives twenty-eight as the number of rivets required, or fourteen for each lug. Using staggered rivets spaced two inches apart will make the depth of each lug about fifteen inches.

The number of rivets necessary for attaching the plate to the beam is partly dependent on the counter stress, and partly upon the length of plate which we consider requisite for fixing the end of the post. About eighteen inches ought to suffice for this purpose. The horizontal component of the counter stress, including initial tension, is $9.625 \times 0.89 = 8.566$ tons, and its moment on the rivets is

$$8.566 \times \tfrac{1}{2}(\tfrac{3}{8} + \tfrac{3}{4}) = 4.82 \text{ inch tons,}$$

which, divided by 0.311 (the resisting-moment for a three-fourths inch rivet), gives sixteen as the number of rivets required. Making them staggered, and spacing them two and a quarter inches apart, would make the length of plate just twenty inches.

Let us assume the sections of the re-enforcing plates at the feet of the posts to be $\tfrac{1}{2}'' \times 5''$; then the lever arm for the chord stress will be $\tfrac{1}{2}(\tfrac{7}{8} + \tfrac{5}{8}) = \tfrac{3}{4}$ inch, and that for the vertical component of the end diagonal stress $\tfrac{1}{2}(\tfrac{7}{8} + \tfrac{1}{2} + \tfrac{1}{2}) = \tfrac{15}{16}$; making the horizontal and vertical component moments on the pin respectively,

$$\tfrac{3}{4} \times \frac{13.944}{2} = 5.23 \text{ inch tons}$$

and

$$\tfrac{15}{16} \times \frac{10.225}{2} = 4.79 \text{ inch tons.}$$

The resultant moment is

$$\sqrt{(5.23)^2 + (4.79)^2} = 7.09 \text{ inch tons.}$$

It is evident, that, to obtain the lever arms used, the chord bars must be packed on the outside and the end diagonals, between the chord bars and the post. The diameter of pin corresponding to 7.09 inch tons is $2\tfrac{1}{8}''$; but a $2\tfrac{1}{4}''$ pin is the smallest that can be used with a $2\tfrac{3}{4}''$ bar. The post bearing is ample, and needs no testing.

If we divide the bearing-stress equally between the post and the re-enforcing plates, there will come upon each of the latter a stress of 2.9; making a moment upon the rivets equal to $\tfrac{1}{2} \times 2.9 = 1.45$ inch tons, which, divided by 0.18, gives eight as the number of five-eighths inch rivets required for each plate. Adding two for safety, spacing the rivets two inches apart, and allowing room for the eye-bar heads, will make the length of each re-enforcing plate about sixteen inches.

The moment on a counter pin is $4.81 \times \frac{1}{2}(\frac{3}{8} + 1\frac{1}{4}) = 3.91$ inch tons, corresponding to a $1\frac{3}{4}''$ pin. By examining Table XXVI., it will be seen that a $2\frac{1}{8}''$ pin will be required to give sufficient bearing.

Referring now to the list of details for a trussed beam, given on p. 30, so as to omit nothing, we can make out the bill of iron as follows: —

Upper chord beam . .	1	$10''$	30# I	$26'$	780#
Lower chord bars . . .	2	$\frac{5}{8}''$	$2\frac{3}{4}''$	$13'$	150 "
End diagonals	4	$\frac{7}{8}''$	$2\frac{1}{2}''$	$13'$	379 "
Counters	4	$1\frac{1}{4}''$	⊙	$15'$	245 "
Posts	2	$6''$	15# I	$5.3'$	159 "
Connecting-plates . . .	2	$\frac{3}{8}''$	$14''$	$3'$	105 "
Re-enforcing plates . .	4	$\frac{1}{2}''$	$5''$	$16''$	44 "
Pin plates	2	$\frac{5}{8}''$	$15''$	$32''$	167 "
Stiffeners	4	$2'' \times 2\frac{1}{2}''$	7 3# L	$8''$	30 "
Pins	2	$2\frac{3}{8}''$	⊙	$10''$	25 "
Pins	2	$2\frac{1}{8}''$	⊙	$10''$	20 "
Pins	2	$2\frac{1}{4}''$	⊙	$13''$	29 "
Fillers	4	@	2#	each	8 "
Rivet heads	-	-	-	about	50 "
Total weight of beam					2,191#

The weight of a plain beam for the same place would be $26 \times 94 = 2,444$, showing a saving of 253 pounds by using a trussed beam. At five cents a pound, this would amount to $12.65; which is considerably more than the cost of the field riveting, and extra trouble in putting such a beam in place. A similar investigation for a trussed beam with one post will show that the weight of such a beam will exceed that of a corresponding plain one: so there would be no economy in such a design for this case.

CHAPTER XIV.

BILLS OF MATERIALS, AND ESTIMATE OF COST.

In making out bills of materials, the list of members given in Chapter III. will prove of great assistance. By its use, one can avoid an underestimate due to an omission of any of the parts of the structure. A good way to make out a bill of material is to prepare six vertical columns, in the first of which write the name of the member; in the second, the number of pieces; in the third and fourth, the dimensions determining their section; in the fifth, their length; and in the sixth, the weight of all the pieces, or, if of wood, the number of feet, board measure, that they contain.

The following examples will serve to explain the method: —

BILL OF WROUGHT-IRON.

Chord channels	12	7″	10½# [	22′	2,772#
Batter-brace channels .	8	8″	12½# [	33′ 9″	3,375 "
Plate	1	¼″	12″	262′	2,620 "
Post channels	8	5″	6½# [	22′	1,144 "
Lateral struts	4	4″	6# [	15′	360 "
Lateral struts	4	5″	6½# [	15′	390 "
Main diagonals	8	¾″	1½″	34′	1,020 "
Counters	8	¾″	¾″	35′	525 "
Etc.	-	-	-	-	-

BILL OF LUMBER.

Joists	55	4″	14″	22′	5,647
Flooring	110	3″	12″	14′	4,620
Hand-rail caps	20	2″	6″	22′	440
Hand rail posts. . . .	30	4″	6″	4′	240
Hub planks	2	2″	12″	110′	440
Felly planks.	2	6″	6″	112′	672
Lateral struts	3	8″	8″	14′	224
Total number of feet, board measure					12,283

It is to be noticed that it is often convenient, as in the case of the "Plate" in the "Bill of Wrought-Iron," or that of the "Hub planks" in the "Bill of Lumber," to combine several lengths in one.

To the length of each chord bar, main diagonal, and hip vertical is to be added three feet to allow for the weight of the heads; and to that of each adjustable rod about five feet, for the heads, upset ends, and sleeve nuts or turn buckles. Should greater accuracy be required for the weight of an adjustable rod, it will be necessary to ascertain what length will be needed at each end for the heads, and how much for the upset ends and adjusting-nuts by the following

TABLE OF EQUIVALENT LENGTHS OF RODS FOR UPSET ENDS, NUTS, SLEEVE NUTS, AND TURN BUCKLES.

$\frac{1}{2}''-1''$	1 upset end and 1 nut	$1\frac{1}{7}$ feet of rod
$1\frac{1}{16}''-1\frac{1}{2}''$	1 upset end and 1 nut	$1\frac{3}{8}$ feet of rod
$1\frac{9}{16}''-2''$	1 upset end and 1 nut	$1\frac{3}{5}$ feet of rod
$2\frac{1}{8}''-2\frac{1}{2}''$	1 upset end and 1 nut	$1\frac{7}{10}$ feet of rod
$\frac{1}{2}''-1\frac{1}{4}''$	2 upset ends and 1 sleeve nut	$\frac{3}{8}$ feet of rod
$1\frac{5}{16}''-2\frac{1}{2}''$	2 upset ends and 1 turn buckle	3 feet of rod

These equivalent lengths do not include the lengths of the upset ends themselves: they represent simply the extra lengths to be added to the bar to equalize the weight of the nuts, sleeve nuts, or turn buckles, and the extra iron for enlarging the ends, which are six or eight inches long.

It is not necessary in a preliminary estimate to find the exact quantities of materials, so approximations to actual dimensions can be made. This will be fully illustrated in Chapter XVI.

Before considering a bill of material as finished, it is well to look it over to see that no mistake has been made in the number of the pieces. It is not an uncommon error to put down only half the correct number.

As soon as the bills of iron and lumber are made out and checked, the dead load per foot should be calculated, to see if it agree with the one assumed within the limit specified on p. 6.

Estimates of cost should be liberal; for, as a rule, the actual profits on bridges fall short of the amounts estimated. They can be made very readily by using a blank similar to the following: —

Estimate on .... Bridge across ....

....

Length span............ft. Height,............ft. Clear Roadway,............ft. Static Load per lineal ft.,............lbs. Moving Load per lineal ft.,............lbs. No. Panels,............ Length Panels............ft.

	$	Cts.
Wrought-iron, lbs. .... @		
Cast-iron, lbs. .... @		
Lumber, ft. .... @		
Piles, ft. .... @		
Hauling, ... *loads* @		
Freight		
Framing		
Falsework		
Erection		
Spikes		
Painting		
Blacksmithing		
Coal		
Freight on tools		
Travelling expenses		
Men's time travelling		
Bidding expenses		
Teaming during construction		
Incidentals		
Total cost of bridge		
Cost per lineal foot		

On fair country roads, a load for a team of horses may be taken to be a ton and a half of iron, a thousand feet of pine lumber, or six hundred feet of oak lumber.

The designing of falsework will be treated in Chapter XX. Its cost will include that of the piles in place, if any be required, and that of the lumber, to which should be added about three dollars per thousand for framing and raising, and a dollar or more per thousand for taking down. Falsework timber can generally be sold for something when the bridge is completed: so a reduction may be made in its cost when the estimate is to be a close one.

The cost of erection can be found approximately for ordinary conditions from Table XXXVIII., by multiplying the number of days' labor there given by the average rate of wages for ordinary bridge hands. It must not be forgotten that there is a great variation in the cost of erection; for it depends upon the locality, weather, skill of laborers, efficiency of foreman, etc. Those who feel inclined to question the correctness of this table should make some allowance for the difficulty which the author has experienced in getting any data whatsoever upon the subject. Few bridge engineers care to part with the knowledge which has cost them both time and money.

In regard to cost of painting, the same difficulty has been encountered: so, for lack of more accurate data, the following table will have to suffice. It has been prepared from a few figures of cost of painting, obtained at a time when wages were a dollar and a half a day.

SPAN	12′ roadway.	16′ roadway	20′ roadway	24′ roadway
50′	$6.00	$7.00	$9.00	$11.00
100′	22.00	25.00	28.00	31.00
150′	–	45.00	51.00	57.00
200′	–	80.00	90.00	100.00
250′	–	125.00	140.00	155.00
300′	–	–	170.00	190.00

Before making an estimate on a bridge, one should endeavor to obtain as many as possible of the following

DATA FOR DESIGNING IRON HIGHWAY-BRIDGE SUPERSTRUCTURES, AND ESTIMATING THEIR COST.

Class of bridge required.

Length of span or spans.

Width of clear roadway.

Headway required in clear above floor.

Live load, if different from the ordinary.

Wind pressure per square foot, if different from the ordinary.

Any extraordinary load, such as paved flooring, heavy falls of snow, etc.

The velocity of passing loads.

Distance of bridge site from nearest railway-station or seaport.

Quality and condition of the roads between these places.

Nature of bed of river, and velocity of stream.

Height of lower chord above bed of river.

Cross section of stream at crossing, showing borings, if any have been made.

Angle which the direction of bridge makes with axes of piers or abutments.

Nature of the country at the site.

Any special difficulty that may be anticipated for the raising.

Kind of falsework it would be advisable to use.

Cost of piles at various places in the neighborhood, if any be required.

Cost of transport of same to site.

Cost of timber per thousand for falsework.

Probable value of falsework timber after bridge is finished.

Cost of withdrawing piles, if necessary.

Number of lineal feet of piles required.

Number of feet of lumber for falsework.

Cost of spikes, bolts, and nails for falsework.

Cost of driving piles.

Cost of transporting pile-driver to and from site.

Common laborer's wages.

Skilled laborer's wages.

Foreman's wages.

Wages for team and teamster.

Cost of superintendence by engineer or engineers.

Number of days' teaming on work.

Date when bridge must be finished.

Probable length of time it will take to raise and complete bridge.

Chances of fair or foul weather during this time.

Chances of having falsework carried away by a sudden rise or an ice-gorge.

Chances of a scarcity of laborers.

Chances of sickness among laborers.

Expenses attendant on same.

Cost of tents or other housing for laborers, if any.

Cost of iron at mill or foundry.

Cost of transport of same to nearest railway-station or seaport.

Cost of lumber per thousand at mill or market.

Cost of transport of same to nearest railway-station or seaport.

Probable expenses for blacksmithing and coal.

Cost of tools, if it be necessary to buy special ones.

Wear and tear of plant, and loss of tools.

Loss of bolts and timber.

Actual cost of raising similar structures under similar circumstances.

Travelling expenses of employees to and from site.

Bidding expenses, if any.

Office expenses in preparing plans, etc.*

Advisable allowance for contingencies.

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* This is usually not considered, as it is a constant expense, and comes out of the annual gross profits of the company.

CHAPTER XV.

ECONOMY.

THE first point to be considered, when deciding upon the style of bridge for a certain stream crossing, is the number of spans. It is, in reality, a consideration of economy which determines this; for the best bridge to build, provided that the water-way be not too much contracted, is the one for which the sum of the cost of superstructure and the cost of foundations is a minimum. If the water-way be too much interrupted, the design would not be an economical one, even if its first cost were the least, because of the risk of washout to which the bridge would always be subject.

In most cases, there is not much choice concerning the number of spans, local considerations often determining it; but there is occasionally a choice between two or even three numbers. The only way, then, to decide is to make a rough estimate of the cost of the superstructure and the foundations for each number; then, if the choice fall about equally between two numbers, it is better nearly always to adopt the longer spans, because the actual expense for the foundations usually exceeds the amount of the preliminary estimate.

Another preliminary point to be settled is whether it would be most economical to build an iron, a combination, or a wooden bridge. Although this work treats of iron bridges only, still this is a point which ought to be considered.

The following mathematical treatment of the problem was given by the late Ashbel Welsh, C.E., past president of the American Society of Civil Engineers:—

TO FIND THE COMPARATIVE ECONOMY OF TWO BRIDGES OF DIFFERENT COST AND DURABILITY, THAT WILL ANSWER THE SAME PURPOSE EQUALLY WELL WHILE THEY LAST.

'Let C be the cost and assumed real value of one of them, T the time it will last, a the compound interest on one dollar for that time, at whatever rate money is worth to the party paying for the bridge, and L the loss on the bridge at the end of the time T, or the amount which it would take to make it as good as new. Let R be the real value of the other bridge, C' its cost, T' its duration, a' the compound interest on one dollar for that time, and L' the loss on the bridge at the end of the time T', or the amount required to make it as good as new. And let V be the real value of the bridge that would last forever if all circumstances should remain constant.

'Now, supposing that the money required for building had been borrowed for an indefinite time, the actual expense at the end of the time T to the party paying for the bridge which would last forever would be aV; and the actual expense at the end of the same time for the first bridge, after making it as good as new, would be $aC + L$. These two quantities are equal: therefore the hitherto unknown value of V is

$$C + \frac{L}{a}.$$

'Similarly, at the end of the time T', the expense for the bridge which would last forever would be $a'V$; and that for the second bridge, after making it as good as new, *if the cost had been the real value* R, would be $a'R + L'$. As before, these two values are equal; and therefore,

$$V = R + \frac{L'}{a'}.$$

Equating the two values of V gives

$$C + \frac{L}{a} = R + \frac{L'}{a'},$$

and

$$R = C + \frac{L}{a} - \frac{L'}{a'}.$$

Now, if the value thus found for R be greater than the cost C', the second bridge is more economical than the first; while, if it be less, the first bridge will be the more economical.'

The next economic consideration is that of depth of truss. Upon this subject much has been written, and many investigations have been made; the general conclusion being, that the depth should be from one-seventh to one-tenth of the span: some English writers say from one tenth to one-fourteenth of the span; while only one, as far as the author knows, — Benjamin Baker, C.E., in his treatise on "Beams, Columns, and Arches," — makes it from one fifth to one-seventh of the span.

Such investigations being purely mathematical, and involving the use of the differential calculus, are of little practical value, as they cannot take into account the numerous variables that ought to be considered. Not only do the stresses in a truss vary with the depth, but also the intensities of working-stress in the compression members. These, again, vary with the number of panels; and this variation is according to a law or laws altogether too complicated to be handled by the calculus. Again: the intensity of working-stress varies, or should vary, according to the position and importance of the member.

In view of the complexity of the question, and wishing to determine the most economic depths for Pratt and Whipple trusses, the author, about a year ago, undertook to solve the problem in a practical manner by assuming the most common clear roadway (sixteen feet), and figuring out a number of diagrams of stresses, and bills of materials. At first he considered that it would be necessary to calculate the total actual cost for every case, but upon further investigation found that it would be sufficient to figure out the sections and weights per lineal foot of the different members of one truss, multiply these by their respective lengths, and sum up the products, neglecting all consideration of details, because the differences in the weights of the latter balance each other. Thus, if the depth of a truss be increased by one foot, there would be a little increase in the weights of the lattice bars and rivets and a decrease in that of the pins and eye-bar heads. These may be taken as balancing each other, without making any appreciable error.

The most economic length of panel was at the same time investigated, and was determined, without preparing complete bills of materials, by considering only those portions of the structure which are affected by the variation in the number of panels.

Economy in pony trusses is an element which ought seldom to influence the design, for a good bridge of this kind will generally require more iron than the ordinary calculations demand. Instead of trying to avoid a little expense, regard should be paid to obtaining a good distribution of plenty of material, in order to partly compensate for the lack of rigidity which is characteristic of the pony truss. In very wide pony-truss bridges, especially when the length of span approaches its superior economic limit, it might be well to make a few calculations concerning the economic depth; but the number of panels should be regulated by the slope of the batter braces, which should never be less than two and a quarter horizontal to one vertical.

The superior economic limit of the pony truss is not a fixed quantity, but decreases as the width of the bridge and the load increase, and as the intensities of working stresses diminish. For example, comparing a pony truss and a through bridge of sixty-five feet span in four panels, sixteen feet clear roadway, designed according to Class C, there is found a difference of three hundred pounds of iron in favor of the pony truss; while with the same span, for a twenty-foot clear roadway, and bridge designed according to Class A, there is a difference of eleven hundred and fifty pounds of iron in favor of the through bridge. For a clear roadway of twelve feet, the superior economic limit of the pony truss would reach as high as seventy-five feet; and, for very wide bridges, the inferior economic limit of the through bridge would reach as low as fifty-five feet: but, on account of rigidity, the superior limit of the former may be placed at sixty-five feet; and, on account of appearance, the inferior limit of the latter at the same length.

After making out diagrams of stresses, and bills of materials, for over one hundred spans, the author came to the following conclusions: —

That if the economic depth be calculated for any span, where the panel length is twenty feet, or the nearest length below twenty feet, and if the economic depth for the same span, but with one panel less, be calculated, the latter will exceed the former by one or two feet.

That, in places where lumber is expensive, it will not be well to make panels over twenty feet long, or, in places where it is cheap, to make them over twenty-four feet long, because timbers exceeding the latter length are not easily procured. Then, too, in designing iron bridges, which are supposed to last indefinitely, it must be remembered, that, as time goes on, long timbers will become more and more expensive, and less easily procurable, even in timber districts; so that panels exceeding twenty feet in length should be employed very cautiously.

For appearance, through spans of one hundred feet and under should have five panels.

The principal objections to the use of the double intersection for short spans are, that, as the rods are long and slender, they will vibrate more than the shorter and larger ones of the single intersection. Any flaw in a small rod will have a proportionately greater injurious effect than the same sized flaw in a larger rod. Long and slender rods are difficult to transport, and are liable to become twisted and bent; though this objection can be partially removed by halving them, and, as the posts are light, they will spring more under the shock of rapidly moving loads.

As the width of roadway and the live load increase, and as the intensities of working-stresses diminish, the inferior limit of the double intersection may be lowered. The table on p. 8 gives the limits which the author would recommend.

The common idea among highway-bridge builders, that a double-intersection bridge should, for economy's sake, have more panels than a single-intersection bridge of the same span and loading, is incorrect.

The economic depth for a double-intersection truss is about three feet greater than that for a single-intersection truss of the same span, and number of panels.

Tables IV. and V. give the principal results of the before-mentioned investigations. The first is the one to be ordinarily used: the second may be employed for districts where the timber is large and plentiful.

There seems to be an unfounded prejudice in the minds of county commissioners and bridge supervisors against long panels. Practically they make a better bridge than do short panels; for the members are fewer and larger, and therefore less affected by flaws, besides less subject to vibration, and less liable to inaccuracy of construction. The floor beams and joists being larger, there is less probability of often receiving their maximum working-loads. The only real objection to long panels is the extra cost of the joist timbers when they are to be replaced.

In addition to what precedes, the following general economic considerations should always receive attention.

Field riveting should be avoided as much as possible, and designs should be made so that all the parts will come together readily during erection.

Rivets should be spaced with regularity, so as to facilitate the punching of the holes by riveting machines.

It is generally better, in through bridges, to pack all but the end chord bars outside of the posts, and reduce the width of top chord plate to a minimum.

It is not always better to employ the apparently most economical depth of channels. For instance, if there be a choice of using ten or twelve inch channels for the top chords and batter braces, and if the *sections alone* would indicate a saving of say three hundred pounds of iron by the use of the twelve-inch channels, the others would be more economical; for the twelve-inch channels require larger stay plates, lattice bars, and re-enforcing plates, besides a wider top chord plate, which would increase the weights of the cover plates, chord pins, post latticing, post stay plates, shoe plates, etc., and even add a little to the lengths of the floor beams.

CHAPTER XVI.

COMPLETE DESIGN FOR A BRIDGE.

LET the bridge to be designed have a span of one hundred and sixty feet, and a clear roadway of fourteen feet with no sidewalks, and let it belong to Class A. Referring to the table on p. 8, we see that the trusses should be of single intersection. On p. 5 we find that the live load should be eighty pounds per square foot of floor, which corresponds to eleven hundred and twenty pounds per lineal foot of bridge.

Table I. gives the dead load as seven hundred and forty-two pounds per lineal foot, say seven hundred and forty pounds.

Table IV. gives eight for the number of panels, and twenty-four feet for the economic depth.

The diagonal upon 20 and 24 is 31.24, which divided by 24 gives 1.3 for the secant; and 20 divided by 24 gives 0.833 for the tangent.

The panel live load, w, is equal to

$$\frac{1}{2} \times \frac{1120 \times 20}{2000} = 5.6 \text{ tons.}$$

The panel dead load, W_1, is equal to

$$\frac{1}{2} \times \frac{740 \times 20}{2000} = 3.7 \text{ tons.}$$

Let us assume that about a third of this is concentrated at the upper panel point, making

$$W' = 1.2 \text{ tons.}$$

The sum of the live and dead panel loads, or W'', is

$$5.6 + 3.7 = 9.3 \text{ tons.}$$

One-eighth of w is 0.7 ton, which multiplied by 1.3 gives 0.91 ton.

The panel dead load multiplied by the secant is

$$3.7 \times 1.3 = 4.81 \text{ tons.}$$

W'' multiplied by the tangent is

$$9.3 \times 0.833 = 7.747 \text{ tons.}$$

The following table of data can now be written: —

$n = 8$	$W'' = 9.3$
$l = 20$	$W' = 1.2$
$d = 24$	$\frac{1}{8}w = 0.7$
diag. $= 31.24$	$\frac{1}{8}w \sec\theta = 0.91$
$\sec\theta = 1.3$	$W_1 \sec\theta = 4.81$
$\tan\theta = 0.833$	$\frac{1}{2}W_1 \sec\theta = 2.405$
$w = 5.6$	$W'' \tan\theta = 7.747$
$W_1 = 3.7$	$\frac{1}{2}W'' \tan\theta = 3.873$
$\frac{1}{2}W_1 = 1.85$	

Next let us draw the skeleton diagram shown on Plate V., and number the panel points, commencing with zero at the right-hand end.

First let us find the stresses in the diagonals, using Table XLI.

The stress in the counter at the point 2 is

$$\tfrac{3}{8}w \sec\theta - \tfrac{3}{2}W_1 \sec\theta = 3 \times 0.91 - 3 \times 2.405,$$

a negative quantity, which shows that there is no stress on this member. Let us mark it zero on the diagram.

The stress in the counter at the point 3 is

$$\tfrac{6}{8}w \sec\theta - \tfrac{1}{2}W_1 \sec\theta = 6 \times 0.91 - 2.405 = 3.055.$$

Let us mark this and all succeeding stresses on the diagram.

The stress in the main diagonal at the point 4 is

$$\tfrac{10}{8}w \sec\theta + \tfrac{1}{2}W_1 \sec\theta = 10 \times 0.91 + 2.405 = 11.505.$$

That in the next main diagonal is

$$\tfrac{15}{8}w \sec\theta + \tfrac{3}{2}W_1 \sec\theta = 15 \times 0.91 + 3 \times 2.405 = 20.865.$$

That in the end main diagonal is

$$\tfrac{21}{8}w \sec\theta + \tfrac{5}{2}W_1 \sec\theta = 21 \times 0.91 + 5 \times 2.405 = 31.135.$$

That in the batter brace is

$$\tfrac{28}{8}w \sec\theta + \tfrac{7}{2}W_1 \sec\theta = 28 \times 0.91 + 7 \times 2.405 = 42.315.$$

That in the middle post is

$$\tfrac{6}{8}w - \tfrac{1}{2}W_1 + W' = 6 \times 0.7 - 1.85 + 1.2 = 3.55.$$

That in the next post is

$$\tfrac{10}{8}w + \tfrac{1}{2}W_1 + W' = 10 \times 0.7 + 1.85 + 1.2 = 10.05.$$

That in the next is

$$\tfrac{15}{8}w + \tfrac{3}{2}W_1 + W' = 15 \times 0.7 + 3 \times 1.85 + 1.2 = 17.25.$$

The stress in the top chord at the panel next to the centre is

$$\tfrac{7}{2}W'' \times \frac{4l}{d} - (1 + 2 + 3)\,W''\frac{l}{d} = 8\,W'' \tan\theta = 61.976.$$

That in the next panel is

$$(8 - \tfrac{1}{2})\,W'' \tan\theta = 7\tfrac{1}{2}\,W'' \tan\theta = 58.103.$$

That in the next is

$$(7\tfrac{1}{2} - 1\tfrac{1}{2})\,W'' \tan\theta = 6\,W'' \tan\theta = 46.482.$$

That in the lower chord at the panel next to the centre is the same numerically as that in the top chord at the second panel from the centre; viz., —

$$7\tfrac{1}{2}\,W'' \tan\theta = 58.103.$$

Similarly, that in the next panel of the lower chord is

$$6\,W'' \tan\theta = 46.482.$$

That in the remaining panels is

$$(6 - 2\tfrac{1}{2})\,W'' \tan\theta = 3\tfrac{1}{2}\,W'' \tan\theta = 27.114.$$

A check by moments about the hip gives the stress in the lower chord at the end panel $3\frac{1}{2} W''' \tan\theta$, which shows that the chord stresses are all right.

Next let us determine if any stiffening be required in the end panels.

An examination of Table XXV. shows that the diameter of the end lower lateral rod is one and eleven-sixteenths inches. Consulting Table IX., we find that the greatest working-stress that can ever come upon such a rod, including the initial tension, is

$$14.399 + 2.375 = 16.774 \text{ tons.}$$

The cosine of the angle which the rod makes with the planes of the trusses is about 0.8: therefore the component of its stress in the direction of the chord is

$$16.774 \times 0.8 = 13.419.$$

Referring to Appendix I., we see that it will be necessary to assume values for A_1, h, and c, in order to find the reduced dead load W_2. From previous experience these values may be taken as follows: $A_1 = 10$, $h = 9$, and $c = 1$, making

$$W_2 = 370 - \frac{30 \times 10 \times 9}{15} = 190 \text{ pounds.}$$

The reduced panel dead load will therefore be

$$\frac{190 \times 20}{2000} = 1.9 \text{ tons,}$$

and the stress on the end panel of the windward lower chord, when the structure is subjected to a wind pressure of thirty pounds per square foot of surface, will be

$$3\tfrac{1}{2} W_2 \tan\theta = \tfrac{7}{2} \times 1.9 \times 0.833 = 5.54 \text{ tons,}$$

showing that stiffening is decidedly needed. This result could have been predicted with certainty from what was stated in Chapter IV. concerning Table I.

Next let us find the sections required for the tension members.

Dividing the stress in the counter at the point 3 by 2 gives 1.528; then, looking down the column marked "Intensity of Working-stress = 4 tons," we find the nearest number to be the one corresponding to a diameter of seven-eighths of an inch: so we will use two seven-eighth inch rods for this place. In reality there is no counter needed in the third panel; but it will be as well to use a single three-quarter inch rod there to aid in adjusting the trusses, and to take up the shock of passing loads.

The intensities of working-stress for the main diagonal are $4\frac{1}{3}$, $4\frac{2}{3}$, and 5 tons. Dividing these into the respective stresses, we find the sections required as marked on the diagram.

As the lower chords at the first and second panels are to be stiffened, the intensity of working-stress for the inner bars at these places will be 4 tons: the intensity for the rest of the chords will be 5 tons. Dividing these intensities into the stresses will give the sections required, which are marked on the diagram. The section for the first and second panels was obtained by supposing that there are four bars of equal size used there; so that the average intensity is $4\frac{1}{2}$ tons.

These two trussed bars of the end panel will not be strong enough to resist the difference between the compressive stress of 13.42 tons and the tensile stress of 5.54 tons or 7.88 tons: so we will have to use an **I**-beam between them, the trussing-bars being attached to the web. This is a more economical arrangement than two channels laced or latticed. Let us try a 4″ **I**. Consulting Table XI., we see that for two round ends the strength of a 4″ 10# **I** is 5 tons, because it is held by the trussing from lateral deflection. Subtracting this from 7.88 leaves 2.88 tons to be resisted by the two bars, or 0.88 ton per square inch, which (*vide* Table XI.) is by no means excessive.

The stress in the top chord is probably so great that the minimum width of top plate will determine the packing on the bottom chord; so that the next step will be the proportioning of the top chord.

Let us take first the stress, 58.103, and try nine-inch channels, which will give $26\frac{2}{3}$ as the ratio of length to least diameter. Referring to Table X. for both ends fixed, we find 3.226 for $26\frac{1}{2}$ diameters, so may use 3.222, which, divided into 58.103, gives

18.03 square inches. From p. 15 we find that the minimum size of top plate for nine-inch channels is $\frac{5}{16}'' \times 11\frac{1}{2}''$, corresponding to an area of 3.59 square inches. Subtracting this from 18.03, and dividing the remainder by 2, gives 7.22 square inches for the area of one channel, which corresponds to a weight per foot of 24.07 pounds.* Referring to Carnegie's "Pocket-Companion," p. 65, we find that nine-inch channels vary in weight from eighteen to thirty pounds per foot; so the nine inch channels required will be procurable. This calculation is not final, for it is not improbable that ten-inch channels will be found more economical.

The best way to settle the point is to ascertain the average weight per foot of chord for both cases. Dividing, then, 46.482 and 61 976 by 3.222, subtracting 3.59 from each quotient, multiplying the remainders by 10, and dividing by 6, gives 18.07 and 26.08 as the weights of the channel bars for the second and fourth panels; which weights are both procurable. The average of the three sections will therefore be 17.23 square inches, corresponding to a weight per foot of 57.43 pounds.

If we employ ten-inch channels, the ratio of length to least diameter will be 24, for which Table X. gives 3.369 as the intensity of working-stress. Dividing this into each of the three stresses gives 18.40, 17.25, and 13.80 as the sections required. The minimum size of top plate (see p. 15) is $\frac{5}{16}'' \times 12\frac{1}{2}''$, corresponding to an area of 3 91 square inches. Subtracting this from 13.80, and multiplying the remainder by $\frac{10}{6}$, gives 16.48 pounds per foot as the weight of the channels in the end panels of the top chord; but the lightest ten inch channel procurable (see Carnegie, p. 64) weighs seventeen and a half pounds per foot: therefore the area of the section will have to be 14.41 square inches.

The average of the three sections will be 16.69 square inches, corresponding to a weight of 55.63 pounds per lineal foot. The difference between 57 43 and 55.63 is 1.8, which, multiplied by

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* The reader may have forgotten that a bar of iron one square inch in cross-section and three feet long weighs exactly ten pounds: consequently, when the area of a section is given in square inches, multiply by ten, and divide by three, to find the weight per foot; and when the weight per foot is given, multiply by three, and divide by ten, in order to find the area.

240, the total length in feet of the two top chords, gives 432 pounds as the apparent saving of iron in the chords by using ten-inch channels: to this must be added the saving in the batter braces, which could be calculated in the same way. It is, however, unnecessary to make this calculation; for we can see, that, all things considered, it is better to adopt the ten-inch channels.

The sections and weights of the top chord panels are now to be entered on the diagram.

It is about time to look to the bottom chord packing, and see if there be sufficient room inside the posts for the diagonals and beam hangers; but we must first proportion the diagonals as marked on the diagram by means of the table on pp. 94, 95, of Carnegie's "Pocket-Companion," and the hangers by referring to Table XXII, which shows that $\frac{7}{8}''$ square bars will be required, square bars being adopted because there will be very little room to spare inside the posts. Referring to Table XXVIII., we find the width of flange for a 10" 24.15# channel to be 2.63 inches. Doubling this, and subtracting the product from the width of plate, leaves 7.24 inches for the width between channels. The thickness of the inner splice plate will be about seven-sixteenths of an inch; doubling which, adding an eighth of an inch for play, and subtracting the sum from 7.24, will leave 6.24 as the distance between inner faces of post channels. The thickness of each inner re-enforcing plate at the foot of a post cannot exceed half an inch, which would leave 5.24 inches for packing the diagonals and hangers. For the second and fourth panel points this will be sufficient; but at the third there would be room enough to let the counters in, and not enough to permit of turning up the sleeve nuts. We can either substitute a single counter, or widen the chord plate. The former will be preferable, as the counter stresses do not affect the sizes of the bottom chord pins, and the central pin of the upper chord should have an excess of strength in any case.

From Table IX. we find that the size of the counter required will be $1\frac{1}{16}''$ square.

Next let us proportion the batter brace. The ratio of length to least diameter is about $37\frac{1}{2}$, for which Table X. gives 2.639

as the intensity of working-stress, which divided into 42.315 gives 16.03 as the section required. Subtracting 3.91, and multiplying the remainder by $\frac{10}{6}$, gives 20.2 pounds per foot as the weight of each channel of the batter brace.

Next let us proportion the posts. We see immediately, from the small stress in the centre post, that its section will be the smallest ever used, viz., that of 5″ 7# channels (*vide* p. 8): so there is no need of calculating the section required. Let us assume six-inch channels for the next post: the number of diameters will then be forty-eight, and the intensity for two hinged ends 1.335, which, divided into 10.05, gives 7.53 square inches, corresponding to two 12.55-pound channels. These are not so economical as seven-inch channels: so we will try the latter. The ratio is $41\frac{1}{7}$, and the corresponding intensity 1.656, which divided into 10.05 gives 6.07 square inches, corresponding to channels weighing 10.12 pounds per foot. The smallest procurable seven-inch channels weigh 10.5 pounds per foot, which size we will therefore adopt.

Let us assume nine-inch channels for the next post, making the ratio 32, and the intensity 2.193, which divided into 17.25 gives 7.86 square inches, corresponding to channels each weighing 13.1 pounds per foot. As the lightest nine-inch channels weigh 14.5 pounds per foot, it will be necessary to employ these, unless eight-inch channels be more economical. Let us try. The ratio is now 36, and the intensity 1.937; making the area 8.91, and the weight of one channel 14.85 pounds per foot, On account of the smaller sizes of lattice bars and stay plates, the eight-inch channels will prove more economical, in spite of their larger section: so we will adopt them.

Next let us proportion the bottom chord, recollecting, that, in the two end panels, an allowance must be made for one rivet hole in each inner bar, the rivets being half an inch in diameter. It is to be noticed that the proportion of width to depth of chord-bars in the centre panels is about one to five, because there are four bars in a panel, and that the depth of the end panel bars approaches the limit for stiffened bars.

From Table VI. we find the size of the hip verticals to be $1\frac{1}{16}$″ square.

Next let us determine the sizes of the pins.

If we take the average thickness of one chord bar at the centre of the span to be $\frac{13}{16}''$, we make a little allowance for accidental thickening of the heads.

Substituting in the formula given on p. 85, viz.,

$$M = \frac{Tw}{2},$$

we find the moment to be 23.56 inch tons, and, referring to Table XII., determine the size of the pin to be $3\frac{1}{4}''$. The least allowable diameter of pin for a $3\frac{3}{4}''$ bar is $3.75 \times 0.8 = 3''$: so we will use $3\frac{1}{4}''$ pins for the five middle panel points of the bottom chords. The chord bars of the end panels being necessarily out of proportion, we have to use at the pedestals and first panel points pins $2\frac{5}{8}''$ in diameter, the smallest that can be used with bars $3\frac{1}{4}''$ deep. It may be well to check the size of these pins. The horizontal component of the moment on the pin at the first panel point is $\frac{1}{2} \times \frac{27.1}{2} = 6.8$ inch tons, nearly. The stress in one hip vertical is equal to one-half of the section required, as given in Table VI., multiplied by the intensity of working-stress for hip verticals, or $\frac{1}{2} \times 2.14 \times 4 = 4.28$ tons. This may be assumed without appreciable error as the load on a hanger. The sum of the thicknesses of a hip vertical and a hanger is almost 2 inches, making the lever arm 1 inch, and the moment, about 4.3 inch tons. The total moment is, therefore, $\sqrt{(6.8)^2 + (4.3)^2} = 8$ inch tons, which corresponds to a $2\frac{1}{4}''$ pin; so that the diameter previously determined is ample.

Next let us find the size of the hip pin. From the formula on p. 86, and Table XXVIII., we find that the approximate thickness of the bearing is about

$$\frac{14.4}{2 \times 10} + 0.3 = 1.02'', \text{ say } 1''.$$

The lever arm for the diagonal stress will be $\frac{15}{16}''$, say $1''$, and, for the hip-vertical stress, $\frac{7}{8}'' + \frac{1}{2}(1'' + 1\frac{1}{16}'') =$ about $2''$, and the corresponding moments respectively, 15.6 and $2 \times 4.28 = 8.6$ inch-tons. Laying out these moments in their respective directions, we find the resultant moment to be about 22.7 inch tons, which corresponds to a diameter of $3\frac{1}{4}''$. The resultant of the stress on one end diagonal and one hip vertical, found by diagram, is about 19 tons. Looking in Table XXVI., we find that a bearing of $1''$ is sufficient.

Next let us find the size of the next chord pin from the hip. From the formula on p. 86, and Table XXVIII., we find that the approximate thickness of the chord bearing is

$$\frac{17.25}{2 \times 10} + 0.45 = 1.31 \text{ inches},$$

and that of the post bearing (p. 87),

$$\frac{8.91}{8} = 1.11 \text{ inches}.$$

Allowing a little for play, the lever arms for the vertical and horizontal components of the stress on a diagonal are respectively $1''$ and $2\frac{1}{4}''$: the components found by diagram are about 8 tons and 6.7 tons respectively, making the moments 8 inch tons and 15.1 inch tons, the resultant of which is 17.1 inch tons, corresponding to a diameter of $2\frac{7}{8}''$. This is large enough for a three-inch bar. There is no need of testing for bearing. A similar investigation for the next panel point shows that a $2\frac{1}{4}''$ pin will be required, which size will also be used for the central pin.

By referring to Table XXV., we can write upon the diagram the sizes of all the lateral and vibration rods and the sections of the upper lateral, portal, and intermediate struts. It will be sufficient to write the sizes of post vibration rods and intermediate struts on one post only, as they are of the same dimensions throughout the bridge.

From p. 54 we take the formula

$$C_n = \frac{2n - 1}{4} w + I \cos\theta + \frac{n - 1}{4} w' - \frac{1}{4}\left(\frac{W}{4} - V\right)$$

to find the stress on the end lower lateral strut between expanding pedestals. Here

$$n = 8 \qquad w = \frac{7.5 \times 30 \times 20}{2000} = 2.25$$

$$I = 2.375 \text{ (see p. 10)}$$

$$\cos\theta = 0.57 \qquad w' = \frac{2.5 \times 30 \times 20}{2000} = 0.75$$

$$W = \frac{160 \times 740}{2000} = 59.2$$

and V (*vide* Appendix I) = one-half the length of span multiplied by the release of pressure per lineal foot on the windward truss, or

$$\frac{80 \times 30 \times 10 \times 9}{15 \times 2000} = 7.2 \text{ tons.}$$

Substituting these values gives $C_n = 9.2$ tons. Assuming four-inch channels, the ratio of length to least diameter will be 42, for which, with one fixed and one hinged end, Table XI. gives an intensity of 2.245: therefore the section required is 4.1 square inches, corresponding to two 6.83# channels. It will be more convenient for riveting to use 5″ 7# channels. At the fixed end of the span a 5″ 10# **I** will answer for a strut between pedestals.

We are now ready to proceed with the "Bill of Iron," in making which, close approximations of lengths are allowable.

Let us prepare the blank form recommended in Chapter XIV., then turn to the list of members given in Chapter III., and fill out the form, proportioning as we go any details whose sizes have not been previously determined. The filling-out of the part denominated "Main Portions" is a very simple matter, and needs but little explanation. It is to be noticed that the lengths of the chord bars and main diagonals have been increased by three feet to allow for the weights of the heads, and those of all adjustable rods by five feet to allow for the weight of the eyes, upset ends, and adjusting-nuts. The intermediate and portal struts are placed seven feet below the level of the upper chord pins, so as to allow a clear headway of fifteen feet.

The size of the floor beam is taken from Table XIX.

The grouping of members having some similar dimensions is to be observed. It involves considerable economy of labor, if one has to estimate on many bridges. In filling out the last vertical column, the tables on pp. 88–93 and 104–109 of Carnegie's "Pocket-Companion" will be found very useful. Let us employ latticing for the top chords, batter braces, posts, and portal struts, and single-riveted lacing for the lateral struts.

Referring to Tables XXXII. and XXXIII., we find the size of stay plates for the top chords and batter braces to be

$\frac{5}{16}'' \times 8''$, since $d = 0.75D$;

that for the middle posts,

$\frac{1}{4}'' \times 5\frac{1}{2}''$, d being equal to $1.4D$;

that for the next larger posts,

$\frac{1}{4}'' \times 6\frac{1}{4}''$, d being equal to D;

that for the largest posts,

$\frac{5}{16}'' \times 6\frac{1}{2}''$, d being equal to $0.88D$;

that for the portal struts,

$\frac{1}{4}'' \times 4\frac{3}{4}''$, d being equal to $1.18D$;

that for the upper lateral struts,

$\frac{1}{4}'' \times 8''$, since d exceeds $2D$:

and that for the end lower lateral strut,

$\frac{1}{4}'' \times 9''$, d being equal to about $1.5D$.

If at the hip joint we make the thickness of each inner and outer connecting-plate $\frac{3}{8}''$, the cross section of the plates through the pin hole by a plane perpendicular to the length of the batter brace will be greater than either that of the batter brace or that of the end panel of the top chord: moreover, the bearing will be slightly in excess of that needed to resist the stresses in an end diagonal and a hip vertical, so we may conclude that these thicknesses will suffice.

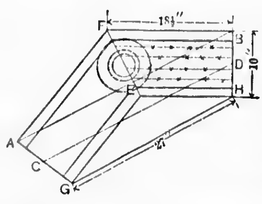
Without committing any grave error, we may assume that the total stress in the end panel of the chord is equally divided between the four connecting-plates, making that on each plate about 11.6 tons.

The thickness of the web of a $10''$ 17.5# channel is $0.3''$ (see Table XXVIII.): therefore the lever arm for the stress in a connecting-plate is $\frac{1}{2}(0.3 + 0.375) = 0.338$ inch, making the moment $11.6 \times 0.338 = 3.92$ inch tons, which divided by 0.311, the resisting-moment for a $\frac{3}{4}''$ rivet, as given in Table XXXVI., gives thirteen as the number of rivets required to resist bending. From the same table we find by interpolation about 1.36 tons as the bearing-resistance for a $\frac{3}{4}''$ rivet on a $0.3''$ plate. The stress transferred to the channel is $2 \times 11.6 = 23.2$ tons, which divided by 1.36 gives seventeen as the number of rivets required for bearing. It will be convenient to use sixteen rivets, in four rows of four in a row. We can do so legitimately, as the calculation calling for seventeen is merely approximate.

It is evident, without calculation, that sixteen rivets will be enough for the connecting plates on the batter-brace side of the pin hole, for the stress is less and the thickness of web slightly greater.

To make the outer plate fit between the flange rivet heads, we cannot have it much more than seven inches wide, unless the said rivet heads be countersunk.

Next let us lay out the hip to scale, as in the accompanying figure, spacing the rivet holes according to the rules given in Chapter II., and allowing three inches of length extra for the part which connects with the batter brace, so as to provide for the portal-strut connection. This approximation is accurate enough for a bill of iron. The circles are those for the pin and the limiting distance for non-countersunk rivets. The rivet spacing is three inches along the horizontal lines.



To calculate the weight of an inner plate, we may divide it into two parts by the line AB in the figure. The area of the lower part is equal to the length of CD multiplied by the perpendicular distance between AB and GH, and that of the upper part by one-half the product of AB and EF. These dimensions are recorded approximately in the "Bill of Iron." The length of the outer connecting-plate is, of course, measured along its centre line.

The area of a section of the four connecting-plates at the first intermediate panel point of the top chord should equal the area of a section of the two chord channels of the third panel, or 13.34 square inches. Let us use

$$\left.\begin{array}{r} \text{two } \tfrac{3}{8}'' \times 10'' = 7.5 \\ \text{and two } \tfrac{7}{16}'' \times \ \ 7'' = 6.12 \end{array}\right\} = 13.62 \text{ square inches.}$$

The stress carried by the channels of the third panel is equal to their area multiplied by the intensity of working-stress, or $13.34 \times 3.369 = 44.94$ tons; which may be divided equally between the four plates, making the stress on each plate about 11.2 tons. Table XXVIII. gives the thickness of web of a 10″ 22.23# channel as 0.45 inch, which will make the lever arm of the stress on the outer plate $\frac{1}{2}(0.45 + 0.43) = 0.44$ inch, and the moment $11.2 \times 0.44 = 4.93$ inch tons, which divided by 0.311 gives sixteen as the number of rivets required to resist bending. The web is so thick that there is no need for figuring upon bearing.

For the other side of the joint the stress on the channels may be taken as

$$10.5 \times 3.369 = 35.37 \text{ tons,}$$

or about 8.8 tons per plate.

The lever arm is $\frac{1}{2}(0.3 + 0.43) = 0.36$ inch, making the moment $8.8 \times 0.36 = 3.17$ inch tons, which divided by 0.311 gives eleven as the number of rivets to resist bending. Dividing 17.6 by 1.36 gives thirteen as the number of rivets to resist bearing: for convenience we can call it twelve, as the stress is not quite so great as we assumed it. It is to be observed that at the hip we suppose that all the chord stress is carried by the connecting-plates, while at the next panel point we assume that the connecting-plates carry only the portion of the stress transferred by the channels, the remainder being transmitted by the cover plate. The reason for this is, that the cover plate at the hip, being bent, cannot be relied upon to carry stress.

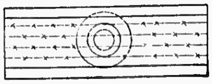
At the next panel point the stress on one plate is

$$\frac{14.49 \times 3.369}{4} = 12.2 \text{ tons.}$$

The sections of the plates will have to be

$$\left.\begin{array}{l}\text{two } \tfrac{7}{16}'' \times 10'' = 8.75 \\ \text{two } \tfrac{7}{16}'' \times \;\;7'' = 6.13\end{array}\right\} = 14.88 \text{ square inches.}$$

The thickness of the web is found to be 0.5″: therefore the lever arm is $\frac{1}{2}(0.5 + 0.43) = 0.46$ inch; and the moment, $12.2 \times 0.46 = 5.61$ inch tons, which divided by 0.311 gives eighteen as the number of rivets for bending.

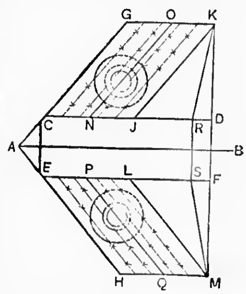


To find the lengths of the connecting-plates we must make, as before, a drawing to scale, as in the accompanying diagram. We thus determine the length of plates for the first intermediate connection to be thirty inches. The length of the plates at the next panel point will be greater by the space required for six rivets, or thirty-four inches and a half, and that at the middle panel point greater by the space required for eight rivets, or thirty-six inches.

Continuing down the "List of Members," we come to the re-enforcing plates on bottom chord struts. Let us make them $\frac{5}{16}'' \times 3''$ in section. It is not worth while to calculate the number of rivets required to connect them to the web of the **I**-beam; because four five-eighths inch rivets will give an excess of strength, making the length about ten inches. Next come the shoe connecting-plates. Let us employ the connection illustrated in Plate VI. From Table XXVI. we find the thickness of bearing for a $2\frac{5}{8}''$ pin and a stress of 13.6 tons to be $\frac{7}{8}''$; subtracting from which 0.38″, the thickness of web of batter-brace channels, leaves $\frac{1}{2}''$ for the thickness of the re-enforcing plate. Assuming the greatest width of plate in a direction perpendicular to the length of the batter brace to be sixteen inches, gives the sectional area of the connecting-plate equal to sixteen square inches, or that of the batter brace: so, provided we have such a width, the half-inch plate will answer the purpose.

The stress carried by the batter-brace channels is $12.12 \times 2.639 = 32$ tons, nearly, or 16 tons on one channel. The lever arm of this stress is $\frac{1}{2}(\frac{1}{2} + \frac{3}{8}) = \frac{7}{16}''$, and the moment, $\frac{7}{16} \times 16 = 7$ inch tons, which divided by 0.493, the resisting-moment of a seven-eighths inch rivet, gives fifteen as the number of rivets required to resist bending. It is better to use seven-eighths inch rivets here, on account of their large bending-resistance. There is no need of calculating for bearing. To determine the dimensions of the connecting-plate, we will proceed as follows; the distance between the channels at the shoe being $12.5'' - 2 \times 2.51'' = 7.5''$.

In the accompanying diagram let us lay out a centre line AB, and the two parallel lines CD and EF each at the distance $3\frac{3}{4}''$ from AB. From any point A lay off the lines ACG and AEH, making angles with AB equal to the inclination of the batter brace to the horizontal. Join CE. Draw the lines JK and LM parallel to CG and EH, and ten inches therefrom: draw also the centre lines NO and PQ. To allow sufficient clearance for the chord heads, the pin holes should be five inches and a half above the top of the shoe plate. By crowding the rivets as near as possible to the flanges of the channels, we are able to use four rows. Laying out the circles for the pin holes, and limiting distance for rivet centres, we determine the height of the box plate to be about $14''$.



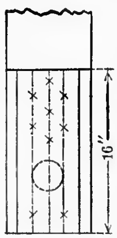
If the vertical sides KD and MF be adopted, the shoe plate will be $28''$ long, which is probably too much. To ascertain, let us find the number, size, and arrangement of the rollers. The total pressure on one shoe is

$$\frac{1}{4} \times 160 \times \frac{1860}{2000} = 37.2 \text{ tons.}$$

Let us assume the dimensions of a roller to be $2''\odot$ by $12''$. Turning to Table XXXIV., we find the permissible pressure on such a roller to be 4.24 tons, which divided into 37.2 gives nine rollers. Spacing them $3''$ centre to centre, and allowing a projection of $1\frac{1}{2}''$ at each end, would make the shoe plate $29''$ long. A plate $12\frac{1}{2}'' \times 29''$ is not a very good shape. Let us try rollers $2\frac{1}{4}''\odot$ by $15''$, the permissible pressure for one of which is

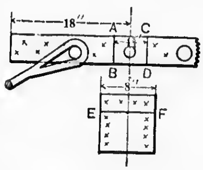
5.63 tons; making the necessary number (37.2 ÷ 5.63) seven. Spacing them $3\frac{1}{4}''$, and allowing the same projection as before, will make the shoe plate $15\frac{1}{2}'' \times 25''$, a better shape. Allowing the shoe plate to project $3''$ beyond the front end of the channels will make the length of the connecting-plate $22''$, which distance is laid off from C to R. The perpendicular distance of R from AG exceeds $16''$: so a plate of the shape $CGKRSMHEC$, before bending, will fulfil all the requirements. To find its weight let it be divided into a rectangle, two triangles, and two parallelograms, as indicated in the "Bill of Iron."

The next details on the "List of Members" are the re-enforcing plates at feet of posts. From Table XXVI., we find that a $3\frac{1}{4}''$ pin requires, for a stress of 8.6 tons, a bearing of less than half an inch, but, in order to compensate for a slight trimming of the flanges of the channels, there must be a plate on the inside, and another on the outside, of each channel; and the least thickness for one of these plates is three-eighths of an inch. We will not trim the five-inch channels, so will not have to use an outer re-enforcing plate: this is because there would be no room for a $3\frac{1}{4}''$ pin through such a plate. The requisite length for these plates cannot be exactly determined; for it is impossible to say how much of the bearing-stress is taken up by the web, and how much by each plate. Let us assume that the inner plate of the largest post channel takes up half the stress on the channel, or 4.3 tons. Table XXVIII. gives the thickness of the web as 0.35 inch. Using $\frac{5}{8}''$ rivets, and figuring for bending and bearing, we find the number of rivets required to be nine. Laying out to scale the foot of the post, as in the accompanying diagram, and allowing five inches and a half between the centre of the pin hole and the foot of the channel, we find that the required length of plate is sixteen inches. In the same way, the lengths of the re-enforcing plates at the feet of the other posts might be calculated: but it is hardly worth while; for, if we make them all of the same length, they will be sufficiently strong without causing much waste of material.



After entering these dimensions on the "Bill of Iron," we refer again to the "List of Members," and, after omitting re-enforcing plates at middle of posts, come to the connecting-plates for lateral struts to top chords. The thickness of these plates should be $\frac{5}{8}''$, and the average width of the legs $2\frac{1}{2}''$. The area of a $4''$ 6# channel is 1.8 square inches, and the intensity of working-stress for forty-two diameters with both ends fixed is, by Table XI., 2.74 tons; making the greatest stress that could ever come upon the channel $1.8 \times 2.74 = 4.93$ tons. The lever arm of the stress is $\frac{1}{2}(\frac{1}{4} + \frac{5}{8}) = \frac{7}{16}$ inch, making the moment $\frac{7}{16} \times 4.93 = 2.16$ inch tons, which divided by 0.389, the resisting-moment for a $\frac{3}{4}''$ rivet, as given in Table XXXVII., gives six as the number of rivets required for attachment to the lateral strut channel. Although the leverage is a little greater for the attachment to the chord-channel flanges, still six rivets will suffice, on account of the liberal estimate for stress, and using rivet tables which have a surplus of strength for lateral system connections. The length of each leg of the T will be about eighteen inches, for various circumstances will necessitate wide rivet spacing in this detail.

The stress and leverage being the same in the two attachments, it is evident that six rivets will be required at each end of the upper channel of the lateral strut for connection to chord. There will be just room for this number; putting two through the channel flanges, and four through the plate between the channels. Were these not strong enough, we could use seven-eighths inch rivets.



The next item upon the "List" is connecting-plates for portal struts to batter braces. These should have a greater strength than ordinary calculations would indicate, in order to provide against the racking effect of the wind. If we use a jaw plate, as in the first of the accompanying diagrams, and two bent plates, as in the second, to attach to the flanges of the strut channels and the web of the batter brace, we provide against all contingencies. These plates are bent at right angles about the lines *AB*, *CD*, and *EF*. It may be well to test the number of rivets for the jaw plate, because it has to act as a re-enforcing plate also. First we must determine the size of the pin which attaches the vibration rods. The diameter of each rod being $1\frac{1}{8}''$, the greatest working-stress thereon is $7.5 \times 0.994 = 7.5$ tons, nearly. The lever arm is $\frac{1}{2}(\frac{1}{4} + \frac{3}{8} + \frac{9}{8}) = \frac{7}{8}''$, making the moment $\frac{7}{8} \times 7.5 = 6.56$ inch tons. Consulting Table XII., we find $1\frac{7}{8}''$ as the diameter required. Table XXVII. shows that there is more than sufficient bearing. Assuming five tons upon the re-enforcing plate, we find the number of eleven-sixteenths-inch rivets required to resist bending to be

$$\frac{5 \times \frac{1}{2}(\frac{1}{4} + \frac{3}{8})}{0.299} = 6,$$

so that the dimensions in the drawing are sufficient.

Let us assume the dimensions for portal connecting-plates to brackets and name plates as $\frac{1}{4}'' \times 8'' \times 18''$.

The section of a connecting-plate for an intermediate strut should be $\frac{3}{8}'' \times 3''$; and we will use three rivets for the connection to the post, and four for that to the strut: it would be useless to figure upon these numbers, as the stress is so small. Owing to the peculiarity of the vibration-rod connection, each plate will have to be about two feet long, as can be seen on Plate VI.

Omitting side-brace connection, the next item is the end lower lateral strut connection to pedestal, which is by means of a jaw plate $\frac{5}{8}'' \times 5''$. The stress on the strut was found to be 9.75 tons, making 4.88 tons on each channel. The number of three-fourths inch rivets required will therefore be

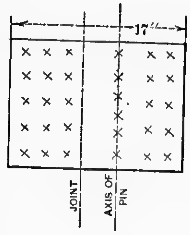
$$\frac{4.88 \times \frac{1}{2}(\frac{1}{4} + \frac{5}{8})}{0.389} = 6.$$

There is no need of figuring for bearing. This would make the total length of jaw plate about three feet, as noted on the "Bill."

For the strut at the fixed end, a plate $\frac{1}{2}'' \times 5'' \times 2'$ will answer the purpose.

The next item is the hip cover plate, which we will make of the same section as the chord plate, and eighteen inches long.

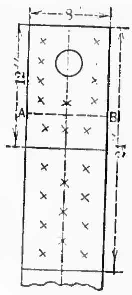
For the intermediate joints we must calculate the lengths of the cover plate thus: the stress on the top plate is $3.91 \times 3.369 = 13$ tons nearly; making the moment on the rivets $13 \times \frac{5}{16} = 4.06$ inch tons, which divided by 0.311 gives fourteen as the number of three-fourths inch rivets required to resist bending. For bearing, the number required will be less. The arrangement of the rivets determining the size of the plate is shown to scale in the accompanying drawing. Next come the filling-plates. Let us average those for the top chord at $\frac{1}{16}''$ thick. For the thickness of the filling-plates over end floor beams, we must subtract from the distance between centre of pin hole and foot of post the half-depth of the chord heads in the end panels, thus,



$$5\tfrac{1}{2}'' - \tfrac{1}{2}(3\tfrac{1}{4} \times \tfrac{3}{2} + 2\tfrac{5}{8})'' = 1\tfrac{3}{4}''.$$

The width will be equal to the diameter of the pin, and the length equal to that of the pin between shoulders.

Next come the extension plates. Let us make them in two thicknesses, the shorter piece extending down to the stay-plates. For the largest post, the total thickness will have to be $\frac{8.91''}{8}$, or $1\frac{1}{8}''$; making that of each plate $\frac{9}{16}''$. Neglecting the effect of the stress on the outer plate, the moment on the rivets will be $8.6 \times \frac{1}{2}(0.35 + 0.56) = 3.91$ inch tons, which divided by 0.311 makes the number of three-quarter inch rivets to be employed equal to thirteen. For reasons advanced in Chapter XII., we must count in only one half of those rivets which pass through the double portion of the plate and the web. Laying off the end of the post to scale, as in the accompanying diagram, we determine the lengths of the plates to be twelve and twenty-four inches respectively. The rivets above the line AB are to be countersunk: their use is simply to make the two plates act as one. We might



calculate the required lengths for the extension plates of the other posts, but it would be unnecessary labor; for if, in the posts with the seven-inch channels, we use two rows of three-quarter inch rivets, instead of three, and in the posts with the five-inch channels two rows of five-eighths inch rivets, making the plates of the same length, we will provide sufficient strength with very little waste of material.

The next on the list are the shoe plates, the area for which we have determined to be $15\frac{1}{2}'' \times 25''$: their thickness (see p. 16) should be $\frac{7}{8}''$.

To determine the size of the roller plate, we will adopt $3'' \times 3''$ 5.9# angles to enclose the rollers, and allow for a motion of two inches, which would make the area $21\frac{1}{2}'' \times 33''$: the thickness should be $\frac{7}{8}''$. The area of the plate in square inches multiplied by two hundred pounds makes about seventy-one tons, which is nearly double the greatest pressure on the shoe; showing that the dimensions decided upon are large enough. The area of the shoe plate multiplied by two hundred pounds per square inch is equal to 38.75 tons; and, as the greatest pressure on the shoe has been calculated to be 37.2 tons, it is evident that we may use the shoe plate as a bed plate by properly anchoring it to the masonry.

Next come the beam-hanger plates. It will not be necessary to calculate their thickness, as the method was fully illustrated by an example in Chapter XIII.; and experience would suggest a thickness of $\frac{7}{8}''$. From Carnegie's "Pocket-Companion," p. 126, we find that $\frac{7}{8}''$ square bars upset to $1\frac{3}{8}''$; and from p. 131 of the same book we see that the longest diameter for the corresponding nut is 2.89″, say 3″; so that, allowing 1″ for clearance, the distance between centres of beam hanger holes will be 4″, and the width of plate for full bearing 7″. We can average the lengths of the plates at 8″.

The weight of a name plate need not exceed forty pounds.

Next come the latticing and lacing bars. Referring to Tables XXX. and XXXI., we find for the top chords and batter braces,

where $d = 0.75\ D$, the bars should be $\frac{5}{16}'' \times 2\frac{1}{4}''$;

for the middle posts,

where $d = 1.4D$, they should be $\frac{1}{4}'' \times 1\frac{5}{8}''$;

for the next larger post,

where $d = D$, $\frac{1}{4}'' \times 1\frac{3}{4}''$;

for the largest posts,

where $d = 0.88D$, $\frac{1}{4}'' \times 1\frac{7}{8}''$;

for the portal struts,

where $d = 1.18D$, $\frac{1}{4}'' \times 1\frac{1}{2}''$;

for the upper lateral struts,

where $d > 2D$, $\frac{1}{4}'' \times 2\frac{1}{8}''$;

and for the end lower lateral struts,

where $d = 1.5D$, $\frac{1}{4}'' \times 2\frac{1}{8}''$.

The distance between centre lines of rivets in the chord and batter-brace channel flanges is about ten inches; the space per panel in chord over which the latticing extends is about eighteen feet; the corresponding distance in the batter brace is twenty-seven feet: so, if we space the rivet holes for the latticing as nearly as possible ten inches apart, there will be twice twenty-two lattice bars required for each chord panel of one truss, and twice thirty-two bars for each batter brace, making seven hundred and eighty-four bars in all. Their length, from Table XXIX., is found to be $1.18' + 0.215' = 1.395'$, say $1.4'$.

We can average the lengths of the lattice bars for the posts thus: assuming a stretch of nine inches, a spread of eight inches and a half, and $1\frac{7}{8}''$ as the width of a bar, gives the total length $1.034 + 0.18 = 1.214$, say $1\frac{1}{4}'$. The average length of space on the posts occupied by the latticing is about twenty feet six inches; making the number of bars per post four times twenty-seven.

The spread, or distance between centre lines of rivets in channel flanges of portal struts, is about six inches and a half, and the latticing extends over about ten feet on the average, after deducting for various plates; which would make the number of lattice bars per strut about four times nineteen. The length will be about $0.768' + 0.145' = 0.92'$ nearly.

The spread for the lateral strut rivet centres is eleven inches and a half; and, as lacing-bars are used, the stretch must be about six inches and three-quarters in order that the angle between the bars may be sixty degrees. This distance is most readily determined by diagram. The length of a bar is, then, $1.113' + 0.197' = 1.31'$. The extent of the lacing is about eleven feet, making the number of bars per strut twice nineteen.

For the lacing-bars of the lower lateral strut, the spread of the rivets is about nine inches and a half, and the correspond ing stretch about six inches. The length of the lacing is about eleven feet, and the number of lacing-bars twice twenty-two. The length of each bar is $0.936' + 0.197' = 1.133'$, say $1\frac{1}{8}'$.

Next on the "List" comes the chord trussing, of which we will assume the section to be $\frac{1}{4}'' \times 3''$. By a rough approximation, we can find the average length for one panel of one truss to be about thirty-three feet. The lengths of the pins are calculated so as to include the weights of the nuts by adding, in most cases, an inch and a half for each nut. The diameter of the intermediate vibration-rod pins is assumed to be $1\frac{7}{8}''$. The lengths of the bolts include an allowance for heads and nuts.

To find if there be any anchorage required at the roller end of the bridge, we must compare the overturning and resisting moments, or, what is the same thing, the release of pressure on the shoe and the weight thereon when the bridge is empty and there is no wind. In finding the stress on the end lateral strut, we determined the release of pressure to be 7.2 tons, and one-fourth the weight of the empty bridge to be 14.8 tons. The latter being more than twice as great as the former, no extra anchorage will be required at the expansion pedestals.

As there is vertical sway bracing, the brackets may be light. Let us make them of $2\frac{1}{2}'' \times 2\frac{1}{2}''$ 4.9# angle-iron, and let them extend vertically and horizontally four feet. Allowing six inches at each end for attachment would make the total length of a bracket about 6.7 feet.

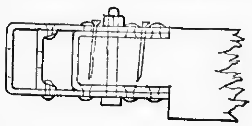
An allowance of 100# for ornamental work will be sufficient.

The equivalent length of a beam hanger can be thus approximately calculated: twice the distance from the centre of the pin to the top of floor beam equals 11″; twice the diameter of pin equals about 6″; twice the depth of floor beam equals 54″; twice the length of hanger below the floor beam equals 6″; allowance for two upset ends and nuts equals 33″; total length equals 110″ = say 9′.

Let us average the diameters of the fillers at $3\frac{1}{2}''$, and their weight at 10# per foot. The average length of filler is not far from 3″. Special fillers will be required at the free end of the span, so as to keep the lateral strut clear of the batter-brace channels, also similar fillers at each end of the span to lie between the outer chord bars and the channels. Let us assume that the channel flanges are notched out to a depth of one inch; then the thickness of the last-mentioned fillers will be $1\frac{1}{8}''$, and that of the others say $\frac{5}{8}''$. Let the external diameter be 7″, and the internal diameter $2\frac{3}{4}''$: the weight per lineal foot will then be (see Carnegie, pp. 105–107) $128.3 - 14.8 = 113.5$.

Turn buckles and sleeve nuts have already been included, and there are no connecting chord heads.

Next come the jaws for lower lateral struts. From the centre of the lower chord pin to the top of the floor beam being $5\frac{1}{2}''$, the depth of the wooden strut will have to be 9″; but the jaws need not be more than 7″ deep, as shown on the accompanying diagram.



The width of the strut need not exceed 7″, nor that of the jaw plate 6″. The thickness of the latter should be $\frac{1}{2}''$. The greatest stress upon any lateral strut, found by resolving the stress upon the $1\frac{7}{16}''$ lateral rod, is about 7 tons, which stress has to be resisted by the rivets connecting the inner and outer jaw plates. The number of rivets required is $\frac{7 \times \frac{1}{2}}{0.389} = 9$, which will make the total length of the two jaw plates about 5′. A piece of 6″ 8.5# channel will be strong enough for the bent eye bearing. It is not worth while to calculate the number of rivets for the combined upper lateral strut jaw and vibration rod bearing plate: so we will average the dimensions as in the "Bill of Iron."

Next on the "List" comes the angle iron around the edges of the roller plates, which we will assume to be 3″ × 3″, weighing 5.9# per foot. The length on one side is 33″, and at the end say 4½″ on each side of the anchor bolt hole; making seven feet in all for each plate.

Next come the pieces of channels, which we will assume to be of the sizes marked on the "Bill," and next the rivet heads, for which we will make a separate bill, then enter the total weight with the other items. Considerable approximation is used in ascertaining the numbers; and the floor-beam rivets are omitted, for their weights are included in the weight of the beams. The total length of top plate for chords and batter braces is about 370′: let us average the rivet spacing therein at 3½″, making the total number $2 \times 370 \times 12 \times \frac{2}{7} = 2537$.

We may say that there is one rivet for each latticing or lacing bar for attachment to channels, and one to every two lattice bars for attaching latticing to latticing. Half-inch rivets will be used for the latter purpose, so as not to weaken the bars unnecessarily. Let us assume that half the stay plates are attached by three-fourths inch, and half by five-eighths inch, rivets, and that there are six or eight rivets per plate. Let us average the number at each joint of the chord at sixty-four, and at each pedestal, not including those through the shoe plate, at thirty-two; and let us assume eighteen rivets at the foot of each post, six per bracket, and fourteen per jaw. The following will then be the approximate

BILL OF RIVET HEADS.

CONNECTIONS.	DIAMETERS.			
	$\frac{1}{2}''$	$\frac{5}{8}''$	$\frac{3}{4}''$	$\frac{7}{8}''$
Plate to chords and batter braces . .	–	–	2,537	–
Latticing and lacing to channels . . .	–	–	784	–
Latticing and lacing to channels . . .	–	432	–	–
Latticing and lacing to channels . . .	–	648	–	–
Latticing and lacing to channels . . .	304	–	–	–
Latticing and lacing to channels . . .	190	–	–	–
Latticing and lacing to channels . . .	44	–	–	–
Latticing to latticing	1,084	–	–	–
Stay plates to channels	–	440	330	–
Connecting-plates to channels	–	–	896	128
Re-enforcing plates to channels . . .	–	180	–	–
Connecting-plates to channels . . .	–	96	244	–
Connecting-plates to channels. . . .	96	60	–	–
Connecting-plates to channels and I .	–	–	80	–
Cover plates to chord plates	–	–	368	–
Extension plates to posts	–	–	280	–
Connecting-plates to shoe plates . . .	–	–	–	32
Trussing to bars	480	–	–	–
Brackets	–	84	–	–
Jaws.	–	140	196	–
Angle iron to roller plates	–	–	–	56
	2,198	2,080	5,715	216

2,198 @ 0.08# = 176#
2,080 @ 0.16 " = 333 "
5,715 @ 0.25 " = 1,429 "
216 @ 0.40 " = 86 "

Total weight of heads = 2,024#, say 2,000# after deducting for countersunk heads.

The number of spikes required will be ten per plank of floor (each plank being 9″ wide), six per post of hand railing, ten per panel for felly plank to flooring, two per joist, and two per jaw; in all, 2,612. The spikes should be $\frac{5}{16}''$ square by 7″ long.

Consulting Carnegie's "Pocket-Companion," p. 129, we find that there are 662 spikes to a keg of 150#; so that we require four kegs, or 600#.

With the exception of the bolts attaching the lateral struts to the floor beams and jaws, each wood bolt must have two washers; making in all 385, say 400. Each washer weighs about a pound.

The weights of most of the nuts have already been included: let us add fifty pounds for lock and pilot nuts.

The total amount of lumber may be estimated in three ways: first, as in the "Bill of Lumber;" second, by consulting Table XV., which gives 2,085 as the amount per panel, multiplying this by 8, and adding 515, the number of feet in the lateral struts; and, third, by consulting Table I., which gives 269 as the weight of lumber per lineal foot, and multiplying this by $160 \times \frac{4}{10}$. The first two methods are accurate; the last, approximate.

BILL OF IRON.

Top chord channels	8	10″	24.15#		
Top chord channels	8	10″	22.23#	20′	10,221
Top chord channels	8	10″	17.5#		
Batter-brace channels	8	10″	20.2#	32′	5,171
Post channels	4	5″	7#		
Post channels	8	7″	10.5#	24′	5,539
Post channels	8	8″	14.85#		
Upper lateral strut channels .	10	4″	6#	15′	900
End lower lateral strut channels	2	5″	7#	14′	196
Portal strut channels	8	4″	6#	14.3′	686
Chord plate	2	$\frac{5}{16}$″	$12\frac{1}{2}$″	120.5′	4,830
Batter-brace plate	4	$\frac{5}{16}$″	$12\frac{1}{2}$″	32.5′	
Intermediate struts	5	4′	8# I	14.4′	576
End lower lateral strut . . .	1	5″	10# I	14′	140
Bottom chord struts	4	4″	10# I	20.5′	820
Main diagonals	8	$\frac{1}{2}$″	$2\frac{3}{4}$″		
Main diagonals	8	$\frac{3}{4}$″	3″	34.25′	6,307
Main diagonals	8	$\frac{7}{8}$″	$3\frac{3}{4}$″		
Counters	4	$\frac{3}{4}$″	⊙	36.25′	759
Counters	4	$1\frac{1}{16}$″	□		
Hip verticals	8	$1\frac{1}{16}$″	□	27′	813

Upper lateral rods	4	$1\frac{1}{4}''$	⊙		
Upper lateral rods	4	$1\frac{1}{8}''$	⊙		
Upper lateral rods	4	$\frac{15}{16}''$	⊙		
Lower lateral rods	4	$1\frac{1}{16}''$	⊙	29.5′	3,455
Lower lateral rods	4	$1\frac{7}{16}''$	⊙		
Lower lateral rods	4	$1\frac{1}{4}''$	⊙		
Lower lateral rods	4	$1''$	⊙		
Vibration rods at portals	8	$1\frac{1}{8}''$	⊙	21′	557
Vibration rods at posts	10	$\frac{7}{8}''$	⊙	19′	380
Chord bars	8	$\frac{3}{4}''$	$3\frac{3}{4}''$		
Chord bars	8	$\frac{13}{16}''$	$3\frac{3}{4}''$		
Chord bars	16	$\frac{5}{8}''$	$3\frac{3}{4}''$	23′	10,459
Chord bars	32	$\frac{1}{2}''$	$3\frac{1}{4}''$		
Floor beams	7	27″	$54\frac{1}{2}$# b. b.	16′	6,104
Total weight of main portions	. .	. . . .	. . . . .	. . . .	57,913
Stay plates on chords and batter braces	32	$\frac{5}{16}''$	8″	$12\frac{1}{2}''$	278
Stay plates on posts	8	$\frac{1}{4}''$	$5\frac{1}{2}''$	$10\frac{1}{2}''$	32
Stay plates on posts	16	$\frac{1}{4}''$	$6\frac{1}{4}''$	11″	76
Stay plates on posts	16	$\frac{5}{16}''$	$6\frac{1}{2}''$	11″	99
Stay plates on upper lat. struts	20	$\frac{1}{4}''$	8″	$13\frac{1}{2}''$	150
Stay plates on lower lat. struts	4	$\frac{1}{4}''$	9″	11″	28
Stay plates on portal struts	16	$\frac{1}{4}''$	$4\frac{3}{4}''$	8″	42
Connecting-plates hip inside	8	$\frac{3}{8}''$	9″	31″	353
Connecting-plates hip inside	8	$\frac{3}{8}''$	4″	36″	
Connecting-plates hip outside	8	$\frac{3}{8}''$	7″	35″	204
Connecting-plates int. inside	8	$\frac{3}{8}''$	10″	30″	250
Connecting-plates int. inside	8	$\frac{7}{16}''$	10″	$34\frac{1}{2}''$	335
Connecting-plates int. inside	4	$\frac{7}{16}''$	10″	36″	175
Connecting-plates int. outside	8	$\frac{7}{16}''$	7″	30″	204
Connecting-plates int. outside	8	$\frac{7}{16}''$	7″	$34\frac{1}{2}''$	235
Connecting-plates int. outside	4	$\frac{7}{16}''$	7″	36″	123
Connecting-plates, b. ch. struts	16	$\frac{5}{16}''$	3″	10″	42
Connecting-plates at shoes	4	$\frac{1}{2}''$	$7\frac{1}{2}''$	22″	
	8	$\frac{1}{2}''$	9″	7″	367
	8	$\frac{1}{2}''$	10″	$18\frac{1}{2}''$	
Re-enforcing plates, ft. of posts inside	8	$\frac{3}{8}''$	8″	16″	
Re-enforcing plates, ft. of posts inside	8	$\frac{3}{8}''$	7″	16″	
Re-enforcing plates, ft. of posts inside	4	$\frac{3}{8}''$	5″	16″	380
Re-enforcing plates, ft. of posts outside	8	$\frac{3}{8}''$	6″	16″	
Re-enforcing plates, ft. of posts outside	8	$\frac{3}{8}''$	5″	16″	

Connecting-plates, lateral strut to chords	10	$\frac{5}{8}''$	$2\frac{1}{2}''$	$3'$	156
Connecting-plates, portal strut to batter braces	8	$\frac{3}{8}''$	$4''$	$3'$	120
Connecting-plates, portal strut to batter braces	16	$\frac{3}{8}''$	$8''$	$8\frac{1}{2}''$	113
Connecting-plates, portal strut to brackets	4	$\frac{1}{4}''$	$8''$	$18''$	60
Connecting-plates, portal strut to name plate	2	$\frac{1}{4}''$	$8''$	$18''$	
Connecting-plates, int. strut to posts	20	$\frac{3}{8}''$	$3''$	$2'$	150
Connecting-plates, end lower lateral strut	2	$\frac{5}{8}''$	$5''$	$3'$	63
Connecting-plates, end lower lateral strut	2	$\frac{1}{2}''$	$5''$	$2'$	33
Cover plates at hips	4	$\frac{5}{16}''$	$12\frac{1}{2}''$	$18''$	78
Cover plates at int. joints	10	$\frac{5}{16}''$	$12\frac{1}{2}''$	$17''$	185
Filling-plates at hips	8	$\frac{1}{16}''$	$10''$	$16''$	22
Filling-plates at int. joints	16	$\frac{1}{16}''$	$10''$	$12''$	33
Filling-plates over beams	4	$1\frac{3}{4}''$	$2\frac{5}{8}''$	$12''$	61
Extension plates	8	$\frac{9}{16}''$	$8''$	$12''$	360
Extension plates	8	$\frac{9}{16}''$	$8''$	$24''$	
Extension plates	8	$\frac{7}{16}''$	$7''$	$12''$	245
Extension plates	8	$\frac{7}{16}''$	$7''$	$24''$	
Extension plates	4	$\frac{7}{16}''$	$5''$	$12''$	88
Extension plates	4	$\frac{7}{16}''$	$5''$	$24''$	
Shoe plates	4	$\frac{7}{8}''$	$15\frac{1}{2}''$	$25''$	377
Roller plates	2	$\frac{7}{8}''$	$21\frac{1}{2}''$	$33''$	345
Beam-hanger plates	14	$\frac{7}{8}''$	$7''$	$8''$	191
Name plates	2	@	40#	each	80
Lattice bars on chord and batter braces	784	$\frac{5}{16}''$	$2\frac{1}{4}''$	$1.4'$	2,572
Lattice bars on posts	432	$\frac{1}{4}''$	$1\frac{7}{8}''$	$1\frac{1}{4}'$	844
Lattice bars on posts	432	$\frac{1}{4}''$	$1\frac{3}{4}''$	$1\frac{1}{4}'$	788
Lattice bars on posts	216	$\frac{1}{4}''$	$1\frac{5}{8}''$	$1\frac{1}{4}'$	365
Lattice bars on portal struts	304	$\frac{1}{4}''$	$1\frac{1}{2}''$	$0.92'$	350
Lacing-bars on upper lateral struts	190	$\frac{1}{4}''$	$2\frac{1}{8}''$	$1.31'$	441
Lacing-bars on lower lateral strut	44	$\frac{1}{4}''$	$2\frac{1}{8}''$	$1\frac{1}{8}'$	88
Chord trussing	8	$\frac{1}{4}''$	$3''$	$33'$	660
Pins, top chord	4	$3\frac{1}{4}''$	$\odot$	$16''$	147
Pins, top chord	4	$2\frac{7}{8}''$	$\odot$	$18''$	130
Pins, top chord	6	$2\frac{1}{4}''$	$\odot$	$18''$	120
Pins, bottom chord	10	$3\frac{1}{4}''$	$\odot$	$22''$	507
Pins, bottom chord	8	$2\frac{5}{8}''$	$\odot$	$20''$	241

Pins, portal vibration	8	$1\frac{7}{8}''$	⊙	$12''$	74
Pins, post vibration	10	$1\frac{7}{8}''$	⊙	$8''$	61
Bolts, name plates	4	@	1#	each	4
Bolts, vibration rods	10	@	5#	each	50
Bolts, anchor	8	$1\frac{1}{4}''$	⊙	$3'$	98
Bolts, portal struts to batter-br.	8	@	8#	each	64
Bolts, hand-rail	68	$\frac{3}{4}''$	⊙	$12''$	100
Bolts, lower lat. strut to beams	49	$\frac{5}{8}''$	⊙	$14''$	58
Bolts, lower lat. strut to jaws .	28	$\frac{3}{4}''$	⊙	$12''$	41
Bolts, felly plank to floor . .	66	$\frac{3}{4}''$	⊙	$15''$	121
Bolts, felly plank to hand-rail posts	34	$\frac{3}{4}''$	⊙	$18''$	75
Brackets	14	$2\frac{1}{2}''\times2\frac{1}{2}''$	4.9# L	$6.7'$	460
Ornamental work	. . .	. . .	. . . . .	. . .	100
Beam hangers	28	$\frac{7}{8}''$	□	$9'$	643
Expansion rollers	14	$2\frac{1}{4}''$	⊙	$15''$	232
Roller frames, sides	4	$\frac{1}{4}''$	$2''$	$22''$	12
Roller frames, rods	6	$\frac{1}{2}''$	⊙	$17''$	6
Fillers	40	@	10# per ft.	$3''$	100
Fillers	8	$7''$⊙	113.5# per ft.	$1\frac{1}{8}''$	97
Fillers	2	$7''$⊙	113.5# per ft.	$\frac{5}{8}''$	
Jaw plates	14	$\frac{1}{2}''$	$6''$	$5'$	700
Jaw plates	10	$\frac{3}{8}''$	$14''$	$12''$	175
Angle iron on roller plates . .	2	$3''\times3''$	5.9#	$7'$	83
Pieces of channels	14	$6''$	8.5#	$6''$	60
Rivet heads	. . .	. . .	. . . . .	. . .	2,000
Spikes	. . .	. . .	. . . . .	. . .	600
Washers	. . .	. . .	. . . . .	. . .	400
Nuts	. . .	. . .	. . . . .	. . .	50
Total weight of details . .	. . .	. . .	. . . . .	. . .	19,850
Weight of main portions . .	. . .	. . .	. . . . .	. . .	57,913
Total weight of iron . . .	. . .	. . .	. . . . .	. . .	77,763

BILL OF LUMBER.

Joists	80	$4''$	$14''$	$20'$	7,467
Flooring (equivalent)	160	$3''$	$12''$	$14'$	6,720
Hand rail	32	$2''$	$6''$	$20'$	640
Hand-rail posts	34	$4''$	$6''$	$4'$	272
Hub planks	16	$2''$	$12''$	$20'$	640
Felly planks	16	$6''$	$6''$	$20'$	960
Lateral struts	7	$7''$	$9''$	$14'$	515
Total number of feet, board measure	. . .	. . .	. . . . .	. . .	17,214

Table I. gives the weight of iron per lineal foot of bridge as 479 pounds, which multiplied by 160 gives 76,640: adding 600 pounds for the spikes, makes the total weight of iron 77,240 pounds. This indicates an error in the table of only seven-tenths of one per cent, — a very satisfactory result.

If we deduct the weight of the end lower lateral struts, rollers, roller plates, anchor bolts, etc., which really do not come upon the bridge, in all about 1,400 pounds, the dead load per lineal foot will be $\frac{76363}{160} + 269 = 746$; which agrees within six pounds with that assumed.

It may appear to the reader who has carefully followed out all the calculations in this chapter, that the designing of iron bridges, and estimating weights thereof, involve a great deal of work, and demand considerable time: but such is not necessarily the case; for an expert could have made this design in from two to three hours, because his experience would have told him the sizes of many of the details and the number of rivets to employ. In this chapter everything has been figured out carefully enough for making working-drawings, instead of merely an estimate of weight; for the author considers that it is better to teach the beginner exact methods in the first place, and leave him to develop approximate ones as his practical experience increases.

A useful deduction which can be made from the "Bill of Iron" in this chapter is the proportion which the weight of the rivet heads bears to the weight of the rest of the iron, excluding that of the floor beams, spikes, and washers. In this case the ratio is about $\frac{2000}{68600} = 2.92$ per cent. The average for a number of estimates made by the author is 2.85 per cent, the greatest being 3, and the least 2.4 per cent. The knowledge of this fact will save considerable time for any one who has many estimates of weight to make.

The author at one time, when in haste, used to figure out the total weight of [illegible] portions, and divide by a certain quantity less than unity, in order to determine the total weight of iron, but has now abandoned the method as giving too loose an approximation, finding that the correct divisor varies considerably with the length of span and the class of bridge. Tables I., II., and III. give the weights of iron for all cases far more accurately than will any such approximation.

CHAPTER XVII.

BRIDGE LETTINGS.

THE ordinary *modus operandi* of bridge lettings is by no means the most perfect that could be devised.

A couple of months before the letting, advertisements are inserted in some of the local newspapers, stating that on a certain day at noon, in the county town, at the court-house, there will be let the contract for building a bridge, or several bridges, in the county. The length of span and clear roadway are nearly always given; and sometimes this is all, for the commissioners, as a general rule, do not know whether they want an iron or a combination bridge. Sometimes, even, they accept a wooden one after advertising for an iron bridge. Occasionally a very fair list of data is advertised, but such is not the rule. In addition to the local advertisements, circulars are often sent to the various bridge companies, requesting them to send representatives to attend the letting. Little do the commissioners think, that in the end the county has to pay the travelling expenses of each representative who attends, as well as for his time. Instead, they say, "The more, the merrier," and congratulate themselves when they have a good attendance, thinking, that, the more representatives, the greater the competition. It may be so in certain cases; but ultimately some one has to pay each traveller's expenses, and who but the counties is there to do it?

It is true that mailed bids are received: but they are very seldom accepted, even if the figures be the lowest; for the commissioners are generally unable to resist the combined eloquence of half a dozen bridge-men. It would be much better for all parties concerned if bids were all sent by mail, and if the awards were made by a competent engineer. It would permit of the reduction of the staff of each bridge company, the lessening of cost to the counties, and, what is more important, the building of better structures. When, by means of much competition, the contract price for a bridge is reduced to cost, or even below it, what does the successful (?) competitor generally do? Lose money? Not at all if he can help it: that is not his way of doing business. He puts up a cheap bridge, cutting down weight on the details, and shaving as much as he dare on the sections. The author does not wish it to be understood that such is the method of the better class of bridge companies. They generally know better than to let their travelling men take contracts for nothing; and when they do get bitten, as they all do occasionally, they put up the bridge at a loss, and take it out of the next county where they obtain a contract.

When remonstrated with for collecting a large crowd to attend the letting of a little bridge, county commissioners have been known to respond, "You see, we don't know exactly what kind of a bridge would be best for the place, nor what style of bridge the money at our disposal will pay for; and when we get a lot of you bridge-men here, who know all about it, we are able to find out exactly what we need." Travelling bridge-men who know all about it! Bridge companies are not willing to send their engineers travelling about the country to attend county bridge lettings. They cannot afford to pay for this purpose salaries of two or three thousand dollars per annum, when men can be obtained to do the work for one-third of that amount. When an engineer is found at a bridge letting, it is generally because he has tired himself out at office-work, and needs a little change.

It is surprising how little the average travelling bridge-man really knows about bridges, and how incapable he is of giving advice of any value to a commissioner. What he does know is how much bridges will probably cost, and this knowledge he obtains from the company's engineer. His *forte* is to do the heavy talking, in which it is by no means necessary for him to stick to the truth.

On the day of the letting, four or five honest farmers (they *are* honest usually, though there have been and are exceptions) meet to determine upon who shall have the bridge. In some cases, after the bids are opened, the contract is immediately let, without discussion, to the lowest bidder. At other lettings, each company's representative is allowed to hold forth, in turn, before the assembly, and show in what way his bridge is superior to the rest.

Some of the arguments advanced are really amusing. One will say "Mine is the best bridge, for it has the most iron in the chords" (ignoring the fact that his bridge has a less depth of truss than any of the others). Another says, "My bridge is the best, because it has the most panels; and it is an acknowledged fact, that, the greater the number of panels, the stronger the bridge." Another will point to the size of his floor beams, forgetting that his bridge has one less panel than have any of the others. With such nonsense are the minds of the poor commissioners crammed, until they do not know the difference between a counter and a batter brace (in fact, it is more than probable that they never did know); and the result is, either that the letting is broken up, or that the contract is let to the one who has done the most talking, and has impressed the most falsehoods upon the understandings of the honest farmers.

Sometimes the commissioners conclude to have the letting done in style, so engage the services of an engineer. Their acquaintance with the members of the engineering profession being rather limited, they employ to decide for them the county surveyor, whose technical knowledge is confined to the use of the compass and transit, and whose mathematical education never went much farther than arithmetic. Or perhaps they will find some one much looked up to in the county as an engineer, who has been plucked at some technical school, and returned home to enjoy the honors of having been a college-man.

As Professor Vose, not long ago, stated in a very able article published in the "Journal of the Association of Engineering Societies" and "Van Nostrand's Magazine," in order to insure the building of none but good bridges, there must be a State inspector, whose duty it would be to pass judgment on the plans of all bridges, before permitting them to be erected in the State. Such an inspector should be, not an ordinary engineer, but an expert in bridge designing. He should receive a handsome salary, and be allowed enough assistance to enable him to do his work in a satisfactory and efficient manner. His tenure of office should be for life, or for a long term of years, and should be beyond the reach of politics and politicians. As long as his work be done efficiently, his position should be assured to him; for a man of the requisite experience and ability would not be willing to accept the position under other conditions.

The letting of bridge contracts to the lowest bidder is the worst method that could be adopted, even when plans and specifications are on file; for the work generally goes to the most unscrupulous bidder, who will secure his margin of profit by diminishing the weight of the details. This weight should be about twenty-five per cent of the total weight of iron-work in the bridge, and it is quite possible to reduce it to one-half of this amount.

If ignorant commissioners must have a rule for letting, it would be better to award the contract to the highest bidder. But the proper way would be to engage the services of a man who knows something about bridge construction, and have him figure out the probable cost of the bridge, allowing a fair margin for profit. A margin of from fifteen to twenty per cent is not excessive, even upon a liberal estimate of cost; for such a margin by no means represents the contractor's actual profits. From it must be subtracted, not only a portion of the annual office expenses, including salaries of clerks, draughtsmen, and engineers, but also the bidding expenses of several lettings where the contractor has been unsuccessful. Then, too, there is the risk of bad weather, high water, rise in price of iron, delay in shop, etc., any one of which is liable to absorb the whole calculated profit, to say nothing of the liability of losing the bridge by a freshet during erection.

When the appropriation is small, it is much better to build a good combination bridge than a poor iron one, because the wood-work of the former can be replaced when it wears out; while the iron, if properly cared for, is as good as new. But a used-up iron bridge is worth little more than the cost of taking it down, and transporting it to where it can be sold for old iron.

The method of having plans and specifications on file for every competitor to bid upon is not a good one. In the first place, it necessitates the sending of engineers to the letting, or at least those who are capable of figuring out the weight of a bridge, thus greatly increasing the bidding-expense; then, if the plans are at all defective in design, a first-class company is unwilling to bid upon them. It is much better to let each company bid upon its own designs.

For most city bridges and for very large county bridges, it is worth a bridge company's trouble to prepare special drawings; but, for ordinary county lettings, standard drawings will answer every purpose when accompanied by a diagram of stresses and special specifications. If the letting is to be done by an engineer, a plate of details similar to Plate III. or Plate IV. will be sufficient; but the ordinary county commissioner does not understand such plans, and generally wants to see what the bridge will look like. Such a picture as the one given on Plate I. would be very taking with county commissioners, but there is a great deal of labor involved in making such a drawing. As a general rule, a sheet showing side and end elevations, and a plan of either the whole or one-half of the bridge, will be sufficient when supplemented by a sheet of details.

It is not unusual for a fancy drawing to take a contract when there are much better and even cheaper structures in competition.

The diagram of stresses should be filled out as shown on Plate V.

Specifications should be quite explicit without being long, or confusing to a reader of ordinary intelligence. The author would recommend the following or some similar form of blank for the use of bidders. By a few erasures it may be made to suit any case.

SMITH & WILLIAMS,

BRIDGE ENGINEERS AND BUILDERS,

PITTSBURGH, PENN.

SPECIFICATIONS FOR BUILDING A WROUGHT-IRON HIGHWAY-BRIDGE.

Length of Span. — To be .... feet .... inches between centres of end pins / bearings.

Clear Roadway. — To be .... feet .... inches between innermost portions of trusses.

Live Load. — To be .... pounds per lineal foot of bridge.

Dead Load. — To be .... pounds per lineal foot of bridge.

Depth of Truss. — To be .... feet .... inches between centre lines of chords.

Clear Headway. — To be .... feet .... inches between floor and lowest part of overhead bracing.

Upper Chords and Batter Braces. — To consist of two .... inch channels, with a plate .... inch by .... inches above, and lattice bars .... inch by .... inches, riveted together at their middle points, below.

Splicing of Joints in Upper Chords. — Shall be made by a plate on each side, as shown in the accompanying drawing. These plates shall be of such thickness as to afford sufficient bearing for the pins, and their combined sectional area shall not be less than that of the channels which they connect. No splice plate to be less than three-eighths ($\frac{3}{8}$) of an inch in thickness. These connections shall be designed under the supposition that the entire stress is carried by the plates and rivets, no reliance being placed on abutting ends of channels.

Cover Plates for Chords. — Shall be .... inch by .... inches by .... inches, and shall contain as many rivets on each side of the joint as will suffice to carry the greatest stress that can ever come upon the chord plates.

Stay Plates for Chords and Batter Braces. — Shall be .... inch by .... inches.

Posts. — Shall consist of two channels, of sizes as marked on the accompanying diagram of stresses. The latticing for same shall vary from .... inch by .... inches to .... inch by .... inches; the bars being riveted together where they cross each other. The posts are to be attached at their ends to the chord pins, and are proportioned for both ends hinged. At the upper ends the connections are to be made by extension plates, each of which is to have a sectional area between the pin hole and stay plate equal to twice that of the channel to which it is attached. The entire stress in the posts is to be considered as carried by the extension plates and their rivets, no reliance being placed on abutting ends of channels.

Upper Lateral Struts. — Shall consist of two channels, laced/latticed, of sizes given on the diagram of stresses, rigidly attached to the chords. The lacing/lattice bars for same shall vary in section from .... inch by .... inches to .... inch by .... inches; and the stay plates, from .... inch by .... inches by .... inches to .... inch by .... inches by .... inches.

Portal Bracing (at each end of the span). — Shall consist of one/two struts, each composed of two .... inch channels, as marked on the diagram of stresses, laced/latticed by bars .... inch by .... inches in section, with stay plates .... inch by .... inches by .... inches; also four adjustable rods, each .... inches in diameter. The struts are to be rigidly attached to the batter braces.

Vertical Sway Bracing (between posts). — Shall consist of two vibration rods, each .... inch in diameter, and an intermediate strut of .... inch I-beam, weighing .... pounds per foot, rigidly attached to the posts at a distance of .... feet .... inches below the level of the upper chord pins.

End Lower Lateral Struts. — Shall consist of ....
....
....
....

Intermediate Lower Lateral Struts. — Shall consist of .... inch by .... inch pine timbers lying upon the floor beams, and well bolted thereto, and attached by wrought-iron jaws to the chord pins.

Side Bracing. — Shall consist of .... inch by .... inch .... pound angle iron, well riveted to the top chord and to the floor beams, which are prolonged .... feet .... inches at each end beyond the trusses for this purpose, as shown on the accompanying drawing.

Bottom Chords. — Shall consist of eye bars, as marked on the diagram of stresses; those in the two panels next to each end of the span being trussed.

Main Diagonals. — Shall consist of eye bars of the sections marked on the diagram of stresses.

Counters. — Shall consist of adjustable rods with loop eyes, the sections being as marked on the diagram of stresses.

Upper Lateral Rods. — Shall be from .... inch to .... inches round iron, attached by bent eyes to the chord pins.

Lower Lateral Rods. — Shall be from .... inch to .... inches round iron, attached either to the chord pins by bent eyes, or to special pins passing through the lateral strut jaws by loop eyes.

Floor Beams. — Shall be rolled / built beams .... inches deep, weighing .... pounds per lineal foot; web .... inch by .... inches; upper flanges, two .... inch by .... inch .... pound angles; lower flanges, two .... inch by .... inch .... pound angles. Stiffeners to be of .... inch by .... inch .... pound angles, ... per beam, made flush with the vertical legs of the flange angles by filling-plates.

Beam Hangers. — Are to be of .... inch round / square iron, upset / not upset: there are to be four / two of them to each beam.

Beam-Hanger Plates. — Are to be .... inch by .... inches by .... inches.

Shoe and Roller Plates. — Are to be .... inch / inches thick.

Pins. — Are to be of the sizes marked on the diagram of stresses. They shall be turned so as to fit the pin holes within one-fiftieth ($\frac{1}{50}$) of an inch.

Pin Bearings. — All pin bearings are to be properly re-enforced.

Brackets. — A straight bracket of .... inch by .... inch .... pound angle iron is to be used to connect each post to the overhead strut. Those for the portals are to be of .... inch by .... inch .... pound angle iron. They are to be connected by .... rivets at each end.

Chord Heads. — Shall be of standard shapes, and so strong that the bar will break in the body rather than in the neighborhood of the eye.

Upset Rods. — All adjustable rods, unless otherwise specified, are to have their ends enlarged for the screw threads; so that the diameter at the bottom of the thread shall be one-sixteenth ($\frac{1}{16}$) of an inch greater than that of the body of the bar, square or flat bars being figured as if of equivalent round section.

Riveting. — Riveting shall in every respect be in accordance with standard authorities; and all riveted connections shall be designed for the rivets to resist the greatest shearing, bearing, and bending stresses that can ever come upon them, no reliance being placed upon friction between plates.

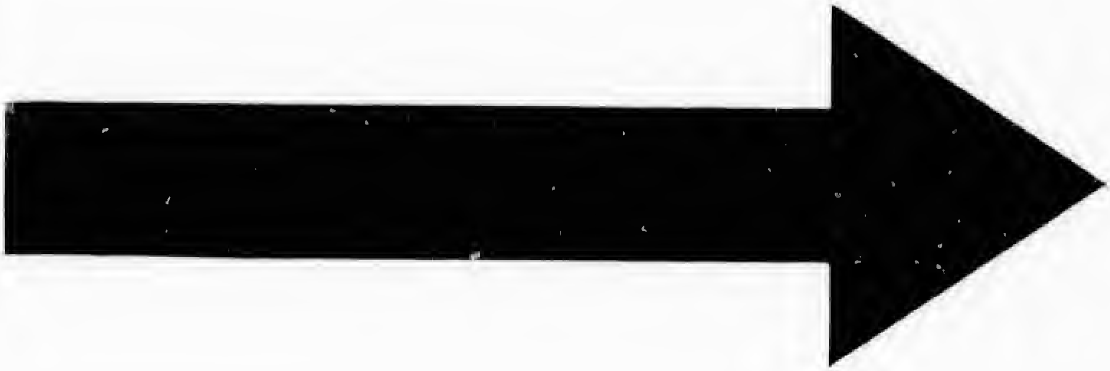
Expansion. — Shall be provided for by ....
....
....
....

Anchorage. — At one end of the / each span, the superstructure is to be anchored to the foundations by .... bolts, each .... inches in diameter, and at least .... feet .... inches long.

Camber. — Shall be at least .... inches when the bridge is empty, and at least .... inches when fully loaded.

Floor System. — Shall consist of .... runs of .... inch by .... inch pine / oak joists, dapped and spiked on the lateral struts; and the floor plank shall be of .... inch pine or oak plank laid diagonally or square across the bridge, as may be preferred, and well spiked to the joists. A felly plank of .... inch by .... inch pine is to be well fastened down on each side of the bridge, and at the middle if required.

Hand Railing. — To consist of ....
....
....
....




2.8
2.5
3.2
2.2
3.6
4.0
2.0
1.8
1.1
1.0

Details of Construction. — All details of construction to be properly proportioned with due regard to the various direct and indirect stresses that may come upon them. All joints to be machine dressed and to fit perfectly. Riveting in the field to be performed in a skilful manner, using the button set. There shall be no loose rivets in the bridge.

Quality of Materials. — Iron for tension members to have an ultimate strength of .... pounds per square inch, and an elastic limit of .... pounds per square inch. Iron for compression members to have the usual correspondence of strength. All timber to be sound and of good quality.

Workmanship. — All workmanship to be first class in every respect, and to be performed to the satisfaction of the engineer or commissioner in charge.

...., 188......

(Signed) SMITH & WILLIAMS.

With the diagrams of stresses, plans, and specifications, there should be handed in, or sent to the commissioner, a proposal in the following or some similar form : —

To the County Commissioners of .... *County,*

State of ....

GENTLEMEN, — We the undersigned hereby agree to build, and put in condition for travel, the superstructure of an iron highway-bridge of .... spans, across ...., in the County of ...., State of ...., according to accompanying plans and specifications, for the sum of .... dollars .... cents ($).

SMITH & WILLIAMS.

...., 188......

After the contract has been awarded, the successful competitor and the commissioners must sign it, and a bond must generally be given by the company as a guaranty that they will complete the work according to the specifications. It is well for the representative of each company to be provided with blank forms for contract and bond. The author would recommend the following for this purpose: —

BRIDGE CONTRACT.

This Agreement, made and entered into this .... day of ...., A.D. 188...., by and between SMITH & WILLIAMS, bridge builders, of the City of Pittsburgh, State of Pennsylvania, parties of the first part, and ....

....

....

County of ...., State of ....,

parties of the second part,

Witnesseth That the said parties of the first part hereby agree to furnish, and erect complete and ready for travel, the superstructure ....

....

of a .... bridge

....

across ...., in said county,

....

....

....

according to the plans and specifications hereunto attached, which are made a part of this contract.

And the said parties of the first part hereby agree to have said structure completed, and ready for inspection, on or before the .... day of ..................................A.D., 188...., allowing a reasonable amount of time in case of unavoidable delays in shipping, by reason of high water, or accidents in construction.

And the parties of the second part agree to have the abutments for said bridge completed by the .... day of ...., A.D. 188..... But in case the said abutments are not finished in the specified time, and the parties of the first part have delivered the material for said bridge at site of same, then the parties of the second part shall pay the parties of the first part seventy-five per cent of the contract price.

And in consideration of the above presents, the said parties of the second part contract, and agree to pay to the said parties of the first part the sum of .... dollars, payable as follows:

....

....

....

In witness whereof the said parties do hereunto affix their seals and signatures the day and year above written.

[SEAL.] ....

[SEAL.] ....

[SEAL.] ....

[SEAL.] ....

BOND FOR BRIDGE CONTRACT.

Know all men by these presents, That we, SMITH & WILLIAMS of Pittsburgh, Penn., as principals, and .... as sureties, are held and firmly bound to the .... in the State of .... in the penal sum of .... dollars, for the payment of which, well and truly to be made, we bind ourselves, our heirs, executors, administrators, and assigns, jointly and severally, firmly by these presents.

Dated at .... in the County of .... and State of .... this ....day of ...., 188....

The condition of this obligation is such, that if the said SMITH & WILLIAMS construct .... bridge in the aforesaid county, according to the plans, specifications, and contract hereto attached ....

....

....

then this obligation to be void and of no effect, or otherwise to remain in full force and virtue in law.

.... *Principals.*

.... } *Sureties.*
....

The previous remarks concerning methods of bridge lettings will probably not be altogether approved of by contractors. Mr. A. P. Boller, C.E., in his treatise on "Iron Highway-Bridges," writes, "It will be noticed in the last clause of the form for 'Invitation,' bidders are requested to be present at the opening of the bids, and hearing them read. This is simple justice. And when one considers the time required to make plans and estimates, even for a small piece of work, to say nothing of the expenditure of money incident thereto, with probable travelling-expenses in addition, no fair-minded man can object to rendering at least what satisfaction may be derived from the public opening of tenders. Bids secretly opened always lead, whether justly or unjustly, to the suspicion of unfair practices, an imputation that can be readily removed by the method of publicity suggested, a method which can be objected to by no one, unless those whose mode of doing business seeks darkness rather than light." This is a clear and well-stated argument, and it is difficult to propose a method that will overcome every objection advanced. However, there are some points in it that will bear criticising.

For a bridge worth, say, twenty thousand dollars, all that Mr. Boller says is certainly correct. But is it so for a small county bridge? The majority of county bridges do not exceed one hundred feet in length, and they are very often let singly. How long will it take, in a well-regulated office, to prepare plans and estimates for an ordinary one hundred-foot county bridge? Usually about thirty minutes; at any rate, no more than two or three hours. The work consists in taking out of their proper places a blue print of the diagram of stresses, a sheet of details, a general plan, and blank forms of specification and proposal, then filling out the two latter, and enclosing all in an envelope for the post. The making out of the estimate of cost should not take five minutes when the amounts of iron and lumber, and a complete list of data, are at hand. If the span be of unusual length, as sometimes happens when replacing an old structure, it may be necessary to make out a new diagram of stresses, but not to prepare a bill of material; for every bridge company should have tables of weights of iron, and amounts of lumber, for all ordinary cases. The actual cost to the contractor, of an ordinary county bridge of one hundred feet span and sixteen feet clear roadway, can be seen from the following estimate: —

Wrought-iron, 27,900 lbs. at 5c.	$1,395 00
Lumber, 10,880 ft. at $18 per M.	195 84
Hauling, 20 loads, at $1.50	30 00
Framing	7 00
Falsework	25 00
Erection	150 00
Painting	25 00
Blacksmithing	5 00
Coal	2 00
Freight on tools	15 00
Travelling-expenses	30 00
Men's time travelling	20 00
Bidding-expenses	40 00
Teaming during construction	10 00
Incidentals	50 00
Total cost of bridge	$1,999 84
Cost per lineal foot, say	20 00

Adding twenty per cent for profit would make the bridge cost the county $2,400. Now, suppose there are ten other bidders present, each of whose expense for time and travelling is forty dollars; then there will be an additional four hundred dollars to be added to the cost of this or some other bridge, for, as before stated, some one must pay it. Eleven is by no means an unusual number of bidders for a small span: there are often as many as fifteen or sixteen.

The estimate for forty dollars for time and travelling-expenses is not excessive, as the author, who has attended a number of lettings, can testify.

These four hundred dollars are worth saving, if it can be done legitimately.

As for bids secretly opened always leading to the suspicion of unfair practices, it is indeed true; and there is no way of avoiding the difficulty, except by having them opened by a committee of public men who are above suspicion. These could be government employees, and residents of the capital of the State, to which bids for all county bridges should be sent. The duty of the committee would be merely the opening of the bids, and the recording of the amounts. The inspector of bridges, who should also reside in the capital, should then examine the bids, and report to the county commissioners, which, in his opinion, is the best bridge for the money (i.e., which he would advise them to accept), and which bridges are up to the specifications, and which not, leaving the final decision to the commissioners.

A summary of his report should be advertised in certain of the engineering-papers, say the two which have the greatest circulation, so as to let the public see that there has been fair play, and to clear the inspector of any imputation of unfair practice. The advertising of the report, including the price for each bridge and the estimated weight of iron in same, would serve to prevent any connivance between contractors and commissioners; because any decided departure from the recommendation of the inspector would immediately awaken suspicion.

No bids without an estimated weight of iron should be received; and, should the inspector doubt the genuineness of the estimate, he could easily check it.

Then, too, the bridge should be weighed at the railway station nearest the site; and, if the weight be found wanting more than a certain per cent of the estimated amount, the contractor should be fined.

In this way the only possibility of fraud would be an agreement between a certain bridge company and all the members of the committee for the latter to insert the contract price in their bid so as to make it just a little lower than that of any other competitor. Considering that the committee would be composed of a number of the most prominent state officials, the probability of such a fraud ever occurring reduces to zero.

CHAPTER XVIII.

WORKING-DRAWINGS.

THE first points to be determined before commencing a working-drawing are the scale and the size of the paper. The least scale which it is convenient to use is one inch to the foot, and the greatest scale for a whole drawing should seldom exceed an inch and a half to the foot. If a smaller scale than one inch be used, difficulty will be experienced in writing the rivet spacing between the rivet holes. The width of the paper should be from three and a half to four and a half, or even five feet: and, as for the length, it is better to use roll-paper, and not to cut it until the limits of the drawing be determined; for it is a great convenience to be able to make all the working-drawings for a bridge upon a single sheet.

The following is a draughtsman's equipment for making working-drawings in a methodical and expeditious manner: a table from four to five feet wide, from six to eight feet long, and about three feet high; a pair of steps each three or four inches rise, and three feet long; a bevelled steel straight-edge, at least three feet long; a beam compass with tangent screw attachment; a couple of small triangles (rubber ones are the best); some four-H and six-H pencils; a little tracing-paper; a finely divided duodecimal boxwood scale (the subdivisions being quarters, eighths, and sixteenths); a good box of instruments, including a protractor and a pair of hairspring dividers; and the usual outfit of rubbers, tiles, pens, etc., that one finds in draughtsmen's offices. T-squares, large triangles, and parallel rulers should never be used in making a working-drawing. The first can never be depended upon, because of the impossibility of having both board and T-square always perfectly true; no wooden ruler can be relied on not to warp; and parallel rulers are a delusion.

For a few inches it is permissible to turn right angles with triangles, but for long distances the beam compass should be used; and parallel lines can be most accurately drawn by erecting a perpendicular near each end of the original line, and laying off on them equal distances. When distances are a little too great for the triangles, and too small for the beam compass, the large ordinary compasses can be used; but it will be found that they are seldom required. The four-H pencils are to be used for writing dimensions, etc., and the six-H ones for drawing lines. The draughtsman should always have at least one of the latter sharpened to a chisel edge for ruling, and another to a point for sketching. He will find it to be greatly to his advantage to keep his pencils always well sharpened, for an error of the width of a pencil-line will often cause a great deal of inconvenience. A piece of emery paper or a fine file will be found useful for sharpening pencils. The tracing-paper will be convenient in transferring drawings of similar chord heads, etc.: its function is merely the saving of a little time.

It is generally better to have both a long and a short scale. The long one may be divided into feet only, the inches and fractions of inches being taken from a diagonal or other small scale. If the draughtsman be not provided with a suitable scale, he can easily prepare a very fair one for himself on a strip of the roll of paper upon which the drawing is to be made.

The method of projecting one view of a piece from another view will not do for working-drawings, owing to the liability of the triangles to slip. All measurements should be transferred by the dividers; and, if there be any probability of the points of the dividers having been moved, the distance between them should be tested by laying it off once more upon the original length. There should be no more than a single transferrence of any one distance, for errors often increase, instead of balancing.

The general arrangement of a working-drawing consists merely in laying out a plan and elevation of one-half of the span, leaving at least a foot of space at each end, and six or eight inches above the elevation and below the plan, if there be room to spare, with the same distance, or a little more, between. As it is immaterial if different portions of the drawing cross each other, provided that such intersection cause no conflicting of the measurements, the various members may be shown in several views alongside of their respective positions in plan and elevation.

Thus the top chord may be represented in an under and an upper view above the elevation of the truss, and the batter brace may be shown in a similar manner above and to one side of the elevation. Projections of the posts on planes transverse to the bridge may be drawn alongside and a little below the elevation of these members, the amount of lowering being sufficient to bring the ends of the strut clear of the chords. Attached to the projections of the posts can be shown the intermediate struts and vibration rods, with their connections; and shortened views of the chord bars and diagonals can be placed alongside their elevations in order to represent the heads clear of all other members. Passing to the plan, on one side is drawn the packed lower chord, and attached thereto the lower lateral rods and struts in half-length; while alongside the latter can be represented an elevation of the same with the floor beams beneath, and an end view of the beams near by. At the other side of the plan, can be shown half-lengths of the upper lateral rods and struts in two views, and a projection of the portal bracing on the plane of the batter braces, and on planes at right angles thereto. Each detail can be delineated to any required extent in the neighborhood of its position in plan, elevation, or both. If necessary, the panel points on one side of the plan may be brought opposite the middle of the panels on the other side, in order to avoid too much intersection.

This arrangement, although a good one, is by no means the only one, and in some cases might not be the best. For instance, in skew bridges it would be well to show the whole of the lower lateral system in the plan, and the whole of the upper lateral system above the elevation, in connection with the uppermost view of the top chord, which should be the plan from above. Then, again, if the bridge be a large one, the height may be so great that it will be impossible to show the plan below the elevation; in which case it will be necessary either to make separate drawings for the plan and elevation, or to place one alongside of the other on the same sheet. In making tracings of the working-drawing, the tracing-cloth can be shifted about so as to group similar parts and so as to avoid too much intersection of different portions.

Provided that any piece be symmetrical about a plane cutting it at the middle of its length and at right angles thereto, it will be sufficient to show only one-half of the piece; and the measurement may be referred to the end of the member, to the central plane, or to both. Where the same detail is used in more places than one, it is not necessary to shew it more than once, provided that it be *exactly* the same in every respect.

As an illustration of how to make a working-drawing, take the case of the bridge treated in the last chapter, and assume that the paper and table are each four and a half feet wide. Using the scale of an inch to the foot, the depth of the elevation will be two feet, and the width of the plan one foot four inches. Allowing six inches above the elevation, and as much more between elevation and plan, will bring the lower side of the plan within two inches of the edge of the paper: this arrangement will do very well. The first step is to draw a line with the steel straight-edge, as nearly as possible, without taking too much trouble, parallel to the length of the paper, and at a distance of two feet six inches below the upper edge. This line should be very fine and perfectly straight. It can be made so by prolonging it half the length of the straight-edge at a time, and afterwards testing it in several places. On this line take a point a foot or more from the left-hand end of the paper, as the centre of the end lower chord pin. Lay off along this line with the greatest possible accuracy the panel length, until the centre of the bridge be reached: in this case twenty feet must be laid off four times. At the panel points erect short perpendiculars with the triangles, and on the perpendicular at the centre lay off the camber, which in this case is three inches (see p. 9). Had the bridge contained an odd number of panels, it would have been necessary to draw the middle panel, and lay off the camber of three inches at each end of this panel. Then, assuming the curve of the chord to be a parabola, the fall from the centre to any panel point is equal to the camber at the centre multiplied by the square of the ratio of the distance of the panel point considered from the middle of the span to the half-length of span.

Thus in the case considered, the falls at the first, second, and third panel points will be respectively $3(\frac{3}{4})^2$, $3(\frac{1}{2})^2$, and $3(\frac{1}{4})^2$, or $\frac{27}{16}''$, $\frac{3}{4}''$, and $\frac{3}{16}''$, making the heights of these points above the horizontal line respectively $3'' - \frac{27}{16}''$, $3'' - \frac{3}{4}''$, and $3'' - \frac{3}{16}''$, or $1\frac{5}{16}''$, $2\frac{1}{4}''$, and $2\frac{13}{16}''$; which distances are to be laid out upon the perpendiculars so as to locate the centres of the lower chord pins. The length of the panels as thus determined differ from those of their horizontal projections by an inappreciable quantity. If there be any lengthening of the chord, it may go against the play of the pins in the eyes.

Next join the consecutive pin centres, producing them each way a little more than a panel length, so as to facilitate the erection of perpendiculars thereto. Then at each of the different centres erect a perpendicular to each centre line meeting there, and bisect the angle between the perpendiculars: the line of bisection will be the centre line of the post. Great care must be exercised in turning these right angles with the beam compasses, two points on each of the perpendiculars being found, so that if these two points and the centre be in exact line, the perpendicular may be relied on as correct. On each of these centre lines lay off the depth of the truss, and complete the skeleton diagram.

A partial check on the accuracy of the construction may be had by measuring the panel length of the top chord, which should agree with the length calculated as follows. Let

p = the increase in the panel length of the top chord above that of the bottom chord,

c = the camber at the centre of the span,

s = length of span,

d = depth of truss,

and

n = number of panels.

Then, according to the method given in Trautwine's "Pocket-Book," $p = \frac{8dc}{sn}$, where d and s may be measured in feet, and c and p in inches. The panel length of the top chord will then be $l' = l + p$, where l is the panel length of the bottom chord.

This is not a certain proof of the accuracy of the work. Two consecutive post centre lines might be equally inclined from their correct positions, and on the same side, though this would be shown in the next panel. A certain check must be obtained by measuring the lengths of the diagonals, which should be equal to each other, and agree with that found by the formula

$$D = \sqrt{d^2 + \left(l + \frac{p}{2}\right)^2},$$

where D is the length required.

For double-intersection bridges, the length of the long diagonals can be found by the method given in Appendix III. The length of the diagonal as manufactured should be one sixty-fourth of an inch less than that calculated, so as to allow for the play of the pins in the eyes. As a slightly greater allowance for play is permissible, it is better to take the next smallest sixty-fourth, if, after making the reduction just indicated, the length should not come out on an exact sixty-fourth. Because of the play in the pin holes of the bottom chord bars, the panel length of the manufactured top chord should exceed the calculated length by about a thirty-second of an inch.

Next fill out the elevation of the chords, posts, diagonals, and batter brace, without showing details. Alongside of each tension member show the heads with their dimensions, and on the shortened distance mark the size of the bars, the number of them, and the length from centre to centre, as shown on Plate VI.

At the right-hand end of the drawing, or on a separate sheet, whichever be more convenient, draw out the heads full size, placing one on the other, if this can be done without confusion. Sometimes as many as six heads can be represented about one centre, provided that both pins and heads diminish together.

For hammered heads the method of construction is very simple. It consists in describing, as in Fig. 1 of the accompanying diagrams, a circle of radius *CA*, equal to that of the pin hole, and a portion of another circle with a radius *CB*, equal to that of the pin hole plus the product of one-half the depth of the bar *HK* by the ratio given in the table on p. 20; then drawing the lines *DE* and *FG* parallel to the sides of the bar, and at a distance therefrom equal to *CB*; and with *C* as a centre and a radius *CD*, equal to twice *CB*, describing an arc to intersect *DE* and *FG* in the points *D* and *F*; finally, with *D* and *F* as centres, and radii equal to *CB*, describing the arcs *HL* and *KM*, tangent to the sides of the bar at *H* and *K*, and to the outer circle at *L* and *M*.

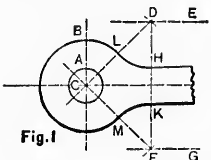


Fig. 1

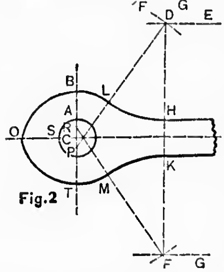


Fig. 2

For welded heads the construction is as shown in Fig. 2, where the pin hole and bar are laid out as before. The distance *AB* is equal to one-half of *HK* multiplied by the ratio given in the table on p. 20; and the distance *SO* is equal to *HK*, or the diameter of the pin hole, whichever be the greater. The centres *P* and *R* of the arcs *OBL* and *OTM* respectively are found by trial; then *DE* and *FG* are drawn parallel to the sides of the bar at distances therefrom *DH* and *FK*, equal to one and seven-tenths times *PB* or *RT*; and with *P* and *R* as centres, and radii equal to two and seven-tenths times *PB* or *RT*, or, what is the same thing, equal to *DH* plus *PB*, arcs are described cutting *DE* in *D*, and *FG* in *F*; finally, with *D* and *F* as centres, and with radii equal to *DH*, arcs are drawn tangent to the sides of the bar at *H* and *K*, and to the arcs *OBL* and *OTM* at *L* and *M* respectively.

These constructions, with slight modifications, are taken from Trautwine's "Pocket-Book."

Next show the posts and the attached sway bracing in two projections with all their details. There should be allowed a clearance of about an eighth of an inch for the ends of the posts inside of the chord. The positions for the stay plates should be as close to the pin as possible, allowing a little clearance for the diagonals. The proper positions can be ascertained from the general elevation. The lattice bars should be close to the stay plates: it will not be necessary to show more than a few of them on each strut, the positions of the others being indicated by their centre lines, as shown on Plate VI. This plate contains a portion of a working-drawing for a model of the bridge treated in the previous chapter. The small scale of three-quarters of an inch to the foot was chosen so as not to make the model too large; and the whole working-drawing is not given, because of the necessarily limited size of the plate.* The principal portions are represented; so that one can, by studying the plate closely, learn all that it is necessary to know in order to make working-drawings; and students are recommended to give this matter special attention.

Next show in two projections the top chord and batter brace with all their details, and give several views of each connecting-plate and other detail in the neighborhood of its position on the elevation. The joints in the channels and plate of the top chord should be located three or four inches to that side of each panel point which is farthest from the centre of the bridge, so that the pin holes shall be bored through a single piece, and through the thicker of the two abutting pieces. At the hip joint it is of course unavoidable to bore the pin hole through the abutting ends of the chord and batter-brace channels. Next pass to the plan, where the first thing to do is to draw parallel to the original horizontal line of the elevation traces of the central vertical planes of the trusses and of the central plane of the bridge, locating the panel points very carefully, and as nearly as possible vertically, below their corresponding positions on the elevation. Then arrange the chord packing on one side of the plan so as to make the bending-moments on the pins as small as possible without having any of the chord bars pull at too great an angle with the plane of the truss.

If any of the panels have trussed bars, the trussing should be here shown, and the spacing of the rivet holes for same in

* The scale has been further reduced by the engraver

the chord bars should be represented close to the plan of the trussed bars. Near the packing should be drawn separate views of the lower chord pins; giving their number, diameter, lengths between shoulders, diameters and lengths of reduced ends, and the total lengths, also the sizes of the nuts.

At the right-hand end of the plan, show the lower lateral struts, and complete drawings for the floor system, including beams, beam hangers, beam-hanger plates, bolts, joists, etc. Generally the floor beams will be all alike: so it will be sufficient to represent half a beam. It may even do to show only half of a lateral strut, although there are always several different lengths of them in a bridge, provided that there be written sufficient directions to enable the carpenters to frame all the struts without possibility of error. In writing dimensions, etc., upon a working-drawing, it is immaterial from which direction the writing be read; that is, it may be read sidewise, upside down, or in any direction most convenient to the draughtsman. In making tracings, this matter can be rectified if it be thought advisable. Full directions for the manufacturer should be written on the drawing. On the rest of the plan, show the upper lateral struts with their details; all the lateral rods with their turn buckles or sleeve nuts, and their eyes in two views; the end lower lateral strut with its details, and its connection to the pedestal; the whole of the portal bracing with its connections; the ornamental work; and the name plates.

Finally, take the list of members, and go carefully over the drawing with it; seeing not only that each piece is represented, but that there are sufficient measurements given to have it manufactured.

The following additional directions and hints may be found useful. Refer each group of rivets to some local line, which is itself referred to the end of the piece, or some other prominent part. Show a section of each member, and write the dimensions of all channels, angles, I-beams, etc., near the section. Write along each piece its extreme length or lengths, its length from centre to centre of eyes, and of what it is composed. The ends of the two pieces of an adjustable rod should be separated by at least three or four inches in the turn buckle or sleeve nut.

Mark what rivets are countersunk, and at which end. If the scale of the drawing be large enough, the countersinking can be thus represented: draw full parallel lines across the rivet for countersinking on the upper side, dotted parallel lines for countersinking on the lower side, and two sets of parallel lines crossing each other at right angles for countersinking on both sides. Be careful to always note how many rights and how many lefts of each piece will be required, when there are both rights and lefts.

Do not forget to write conspicuously the scale or scales of the drawing. Lay out all bevelled edges on an enlarged scale, say from half to full size, and mark their dimensions along the edges, referring all measurements to a transverse line through some well-defined point, as the centre of the pin hole. These measurements should be checked by calculation. The slight bevels at the joints of the top chord should be treated with as much accuracy as the bevels at the hip joints; but, as the bevel is very slight, it will be legitimate to put it all on one of the abutting ends, making the other a square cut.

The centre lines for lacing-bars on the under side of a strut should be dotted. In laying out a long row of rivets — for instance, lattice rivets, or those for the top plate of a chord or batter brace — calculate the distance of some of the intermediate rivet holes from one end of the strut. Lay out these holes, then interpolate the others; because, if the spacing be laid out continuously from one end with dividers, any error in the span of the dividers will be multiplied by the number of times the distance is laid off.

After laying out a complete system of rivets for any member, check by seeing that the sum of the distances between rivet holes plus the distance of each end rivet from the end of the member is equal to the total length of the member. Make duplicates of as many parts of the bridge as possible, even at the expense of a small amount of iron, not only to save time in draughting, but also in the shop, and to facilitate the work in erection.

Arrange to have as few loose pieces for shipment as possible, and mark on the drawing of each connecting-piece to what it is to be attached, or if it is to be left loose. Thus the hip connecting-plates should be attached to either the chords or batter braces, sometimes to both; those of the top chord, to that portion through which the pin hole is bored; those for the upper lateral struts should be left loose. If there be any reason to fear rough handling of the iron in transit, it may be necessary to send some of the connecting-plates separately; but the more loose pieces, the more field riveting, and the more field riveting, the greater the erecting expenses, and the longer the time and the greater the risk in raising the bridge.

Rivet spacing should be as regular as circumstances will permit; and all changes in spacing should be made suddenly, instead of gradually, so as to facilitate the punching of the holes by machine.

All measurements should be in feet, inches, and the following vulgar fractions of inches; viz., halves, quarters, eighths, sixteenths, thirty-seconds, and sixty-fourths. Workmen do not seem to understand decimals: so it is better not to use them.

Avoid also the use of the development method, as it is beyond the comprehension of ordinary workmen.

The length of all main members should be measured on the drawing, then checked by calculation.

When nuts are placed in a confined position, — for instance, pin nuts in jaws, — care should be taken that there be ample room for them to turn in; as it is very awkward, and sometimes impossible, to screw up a nut which is stationary, by turning the pin. Nuts in confined positions may be turned by hammering them eccentrically.

Be careful to design no connection in such a manner that there will be rivets that cannot be driven without inconvenience. This remark is especially applicable to field riveting.

It must be borne in mind, that, no matter how carefully the bill of iron was prepared, there will be many minor changes found necessary in making the working-drawings; but, as a rule, such changes cannot materially affect the total weight of iron in the bridge.

CHAPTER XIX.

ORDER BILLS AND SHIPPING BILLS.

When there is necessity for haste in building a bridge, as there generally is in America, time can be saved by sending a partial order bill to the manufacturers before starting to make the working-drawings, or after they are partially pencilled.

Such preliminary order bills should include only those portions which are termed in this treatise "Main Members," and those details of the sizes of which the designer is certain; for instance, stay plates, pins, brackets, and the plates and angles for built beams.

The length of the main members in the bill should be three-quarters of an inch greater than will actually be required, in order to allow for the dressing of rough ends; and, should there be any doubt in the designer's mind concerning the exact length of any piece, he should make the ordered length great enough to cover any variation which there may be in the design.

Of course, where there are bevelled ends on a piece, the extreme length plus the allowance for waste must be given.

Where a number of small pieces are to be cut from one large piece, an extra allowance of length must be made to provide for the waste in cutting, say from an eighth to a quarter of an inch for each short length. After finishing the pencilling for a working-drawing, the remainder of the preliminary order bill may be made out and sent. It should be divided into the following groups, containing the measurements indicated: —

Channels	No.	Depth	Weight per foot	Length	Finished length

Angles	No.	Thickness	Legs	Weight per foot	Length	Finished length

I-beams	No.	Depth	Weight per foot	Length	Finished length

Plates	No.	Thickness	Width	Length	Finished length

Eye bars	No.	Thickness	Depth	Depths of heads	Length centre to centre of eyes	Extreme length

Adjustable rods with plain eyes	No.	DIAMETER.		SHORT PIECE.			LONG PIECE.		
		Rod	Upset end	Diameter of eye	Length of loop	Length, centre of eye to end	Diameter of eye	Length of loop	Length, centre of eye to end

Adjustable rods with bent eyes	No.	DIAMETER.		SHORT PIECE.		LONG PIECE.	
		Rod	Upset end	Diameter of eye	Length, centre of bend to end	Diameter of eye	Length, centre of bend to end

Pins	No.	DIAMETER.		Length between shoulders	Extreme length
		Body	Reduced ends		

Rollers	No.	Length between shoulders	Extreme length

Any details which will not go into one of these groups will be made of material that the manufacturer keeps in stock; for instance, fillers, washers, nuts, turn buckles, sleeve nuts, ornamental work, name plates, bolts, and iron hand railing. It would not be a bad idea for bridge companies to keep blanks similar to the foregoing, for preliminary order bills.

Pins should be ordered an eighth of an inch greater in diameter than required in the bridge, so that they may be turned down; and shoe plates and roller plates, one-sixteenth of an inch thicker, to allow for planing.

Spikes are generally purchased separately, by the keg, from special dealers.

Lumber is, of course, bought separately: it should be ordered in the following form:—

No. pieces	Thickness	Width	Length	Kind of wood

After the working-drawing is finished, there should be prepared to accompany it a final order bill, in which are to be grouped all similar pieces, and all their details which are attached to them in the shop. The following grouping will cover any case of an ordinary iron highway-bridge designed according to the method of this treatise:—

TOP CHORD SECTIONS.

Channels	No.	Depth	Weight per foot	Finished length
Top plates	No.	Width	Thickness	Finished length
Cover plates	No.	Width	Thickness	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lattice bars	No.	Width	Thickness	Length centre to centre of end rivet holes
Connecting-plates . . .	No.	Width	Thickness	Finished length

BATTER BRACES.

Channels	No.	Depth	Weight per foot	Finished length
Top plates	No.	Width	Thickness	Finished length
Cover-plates (hip)	No.	Width	Thickness	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lattice bars	No.	Width	Thickness	Length centre to centre of end rivet holes.
Connecting-plates	No.	Width	Thickness	Finished length
Shoe plates	No.	Width	Thickness	Finished length

CHANNEL BOTTOM CHORDS.

Channels	No.	Depth	Weight per foot	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lacing-bars	No.	Width	Thickness	Finished length
Re-enforcing plates . .	No.	Width	Thickness	Finished length
Connecting chord heads . .	No.	Depth	Thickness	Length centre of eye to end, and extreme length

POSTS.

Channels	No.	Depth	Weight per foot	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lattice bars	No.	Width	Thickness	Finished length
Extension plates . . .	No.	Width	Thickness	Finished length
Re-enforcing plates . . .	No.	Width	Thickness	Finished length

UPPER LATERAL STRUTS.

Channels	No.	Depth	Weight per foot	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lacing-bars	No.	Width	Thickness	Finished length
Jaw plates	No.	Width	Thickness	Finished length

END LOWER LATERAL STRUTS.

Channels	No.	Depth	Weight per foot	Finished length
I-beams	No.	Depth	Weight per foot	Finished length
Angle irons	No.	Legs	Weight per foot	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lacing-bars	No.	Width	Thickness	Finished length
Jaw plates	No.	Width	Thickness	Finished length

PORTAL STRUTS.

Channels	No.	Depth	Weight per foot	Finished length
Stay plates	No.	Width	Thickness	Finished length
Lattice bars	No.	Width	Thickness	Finished length
Jaw plates	No.	Width	Thickness	Finished length
Connecting-plate to batter-brace	No.	Width	Thickness	Length of each leg
Connecting-plate for brackets to channels	No.	Width	Thickness	Finished length
Connecting-plate for name plates to channels . .	No.	Width	Thickness	Finished length

STIFFENED HIP VERTICALS.

Channels	No.	Depth	Weight per foot	Finished length
Flat bars	No.	Width	Thickness	Length centre to centre of eyes, and extreme length
Stay plates	No.	Width	Thickness	Finished length
Lacing-bars	No.	Width	Thickness	Finished length
Re-enforcing plates. . .	No.	Width	Thickness	Finished length
Trussing	No.	Width	Thickness	Finished length

INTERMEDIATE STRUTS.

I-beams	No.	Depth	Weight per foot	Finished length
Connecting-plates . . .	No.	Width	Thickness	Length of each leg

MAIN DIAGONALS AND PLAIN CHORD BARS.

No.	Depth	Thickness	Depth of heads	Thickness of heads	Diameter of eyes	Length centre to centre of eyes	Extreme length

HIP VERTICALS AND COUNTERS.

No.	Section	Diameter of enlarged end	Lengths of loop eyes	Lengths centre of eyes to ends, or centre of eye to centre of eye

LATERAL AND VIBRATION RODS.

No.	Diam.	Diameter of enlarged end	Length centre of eye to bend or loop	Length centre of bend or eye to end of short piece	Length centre of bend or eye to end of long piece

STRUTS OF TRUSSED CHORD BARS.

No. of struts	Section	Sizes of heads	Section of trussing	Length of trussing	Length of strut centre to centre of eye	Extreme length of strut

SIDE BRACING.

No.	Section	Size of connecting-plate	Extreme length of brace

IRON HAND RAILING.

No. of posts	Sizes of posts	No. of panels	Size of panel	Total length of railing

BUILT FLOOR BEAMS.

Plates . .	No.	Width	Thickness	Finished length
Angles . .	No.	Legs	Weight per foot	Finished length

TRUSSED FLOOR BEAMS.

I-beams . .	No.	Depth	Weight per foot	Finished length
Angles . .	No.	Legs	Weight per foot	Finished length
Plates . .	No.	Width	Thickness	Finished length

ROLLER AND BED PLATES.

Plates . .	No.	Width	Finished Thickness	Finished length
Angles . .	No.	Legs	Weight per foot	Finished length

NAME PLATES.

No.	Date.

OTHER SEPARATE PLATES.

No.	Width	Thickness	Finished length

PINS AND THEIR NUTS.

No.	Diameter	Size of nuts	Length under head, or extreme length

BOLTS AND THEIR NUTS.

No.	Diameter	Size of nuts	Length under head, or extreme length

BRACKETS.

Angles . .	No.	Legs	Weight per foot	Extreme length
Channels. .	No.	Depth	Weight per foot	Extreme length
Tee-iron . .	No.	Legs	Weight per foot	Extreme length

ORNAMENTAL WORK.

No. of pieces	Description

BEAM HANGERS AND THEIR NUTS.

No.	Section	Diameter of upset end	Diameter of eye	Size of nuts and lock-nuts	No. of nuts and lock-nuts	Length of one leg

SETS OF ROLLERS.

Rollers	No.	Diameter	Length between shoulders	Extreme length
Cross-rods	No.	Diameter	Length between shoulders	Extreme length
Side-bars	No.	Thickness	Width	Extreme length

FILLERS FOR PINS.

No.	External diameter	Internal diameter	Length

TURN BUCKLES AND SLEEVE NUTS.

No.	Taps

JAWS.

Plates . .	No.	Width	Thickness	Extreme length
Channels . .	No.	Depth	Weight per foot	Extreme length

WASHERS.

No.	Diameter	Diameter of bolts

SEPARATE RIVETS.

No.	Diameter	Length under head	Kind of head	Position in bridge	Parts connected

PIN PILOTS.

No.	External diameter	Internal diameter

Some companies send also a complete bill of rivets to be used in the shop; but this is scarcely necessary, as it is more properly the place of the manufacturer to prepare such a bill.

The following form will be needed for the purpose: —

RIVETS.

Member	No.	Diameter	Length between heads	Kind of heads	Parts connected

An allowance of three per cent should be made for waste in shop rivets, and from ten to twelve per cent in field rivets.

If the hip verticals be flat bars, they are to be transferred to the group of "Main Diagonals, etc." The posts, chord bars, and diagonals of trussed beams, are included under the general heads of "Posts," etc.

The corresponding form of "Shipping Bill" is as follows: —

STRUTS.

Member	No.	Length centre to end, or extreme length	Mark

BARS.

Member	No.	Section	Diameter of eyes	Sizes of heads	Length centre to centre of eyes	Mark

RODS.

Member	No.	Diameter	Diameter of eyes	Diameter of upset ends	Threads R. or L.	Length centre of eye to end, or centre of eye to centre of eye	Mark

SIDE BRACING.

No.	Section	Extreme length	Mark

IRON HAND-RAILING.

No. of posts	No. of panels	Mark, if any

FLOOR-BEAMS.

No.	Extreme length	Mark

ROLLER AND BED PLATES.

No.	Position (fixed or free end)	Mark, if any

NAME PLATES.

No.	Date

OTHER SEPARATE PLATES.

No.	Position	Mark

PINS AND THEIR NUTS.

No.	Diameter	Length between shoulders	Extreme length	Dimensions of ends	Mark

BOLTS AND THEIR NUTS.

No.	Diameter	Diameter of upset ends	Length under head, or extreme length

BRACKETS.

No.	Position	Extreme length	Mark

ORNAMENTAL WORK.

No. of pieces	Description	Mark

BEAM HANGERS AND THEIR NUTS.

No.	Diameter of eye	No. of nuts and lock-nuts	Mark

ROLLERS.

No. of sets

FILLERS FOR PINS.

No.	External diameter	Internal diameter	Length	Mark

TURN BUCKLES AND SLEEVE NUTS.

No.	Taps

JAWS.

No.	Position	Mark

WASHERS.

No.	Diameter	Diameter of bolt

SEPARATE RIVETS.

No.	Diameter	Length under head	Kind of head	Position in bridge	Parts connected

PIN PILOTS.

No.	External diameter	Internal diameter	Mark

The following is the system of marking iron before shipment which the author would recommend. It should be thoroughly comprehended by the manufacturer, the foreman in charge of erection, and the time-keeper or clerk, if there be either employed on the work.

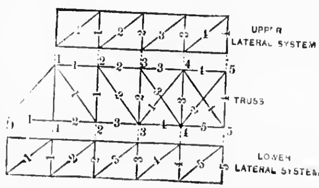
Where the work is very extensive, the time-keeper generally checks the material as it arrives on the ground.

First, if there be more than one span, each piece of each span should be marked with a daub of color peculiar to that span: thus the first span may be white, the second yellow, the third blue, etc.; care being taken to choose such colors as will be readily distinguished upon the iron-work.

The colors may be marked in the last column of each division in the "Shipping Bill" and at the end: thus the mark for a main diagonal may be "3 Bl.," or "2 W.;" the first representing the third set of main diagonals in the third span, and the other the second set in the first span. The letters Bl. are chosen for blue, so as not to be mistaken for the letter B used elsewhere. A similar precaution should be taken with the other colors.

In addition to this characteristic color-mark, each piece should be marked in white paint according to the following method.

R. and L. denote that the piece, if a main portion, lies to the right hand or to the left hand when one stands at the nearest portal, astride the centre plane of the bridge, and looks towards the centre of the span. If a detail, it denotes that it lies to the right or left hand when one stands astride the middle vertical plane of the truss to which the detail belongs, at the foot of the nearest batter brace, and facing the centre of the span. The numbers can be readily understood by referring to the accompanying diagram.



Chord sections are to be numbered, and marked R. or L.

Batter braces are to be marked R. or L.

Channel bottom chords are to be numbered.

Posts to be numbered, and marked R. or L.

Upper lateral struts to be numbered.

End lower lateral struts to be marked Fix. or Fr. (fixed or free end).

Portal struts to be marked U. or L. (upper or lower).

Intermediate struts to be numbered to correspond to the posts to which they are to be attached.

Main diagonals and counters to be numbered.

Hip verticals need no mark.

Upper lateral rods to be marked U. 1, U. 2, etc.; the numbers corresponding to those on the diagram.

Lower lateral rods to be marked L. 1, L. 2, etc.; the numbers corresponding to those on the diagram.

Vibration rods need no marks.

Chord bars to be marked 1 A, 1 B, 1 C, 2 A, 2 B, 2 C, etc.; the numbers corresponding to those on the diagram, and the letters denoting the position in the panel, A being for those on the exterior side of the truss, B for those next to the outside, etc.

Side braces to be numbered to correspond to the panel points to which they belong, and to be marked R. or L.

Iron hand railing requires no marks except E on the end posts and panels, if these be different in any respect from the others.

Floor beams to be numbered to correspond to the panel points.

Roller and bed plates to be marked R. or L., if there be any difference.

Name plates require no marks.

Separate plates to be numbered so as to correspond to the panel points to which they belong, and to be marked R. or L., if necessary.

Lower chord pins to be marked L. 0, L. 1, L. 2, etc.; the numbers corresponding to those of the panel points.

Upper chord pins to be marked U. 1, U. 2, U. 3, etc.; the numbers corresponding to those of the panel points.

Portal diagonal pins to be marked P.

Vibration-rod pins to be marked V.

Pins at middle of posts to be marked M. 1, M. 2, M. 3, etc.; the numbers corresponding to those of the posts.

Lower lateral-rod pins not to be marked, for they should be shipped attached to the jaws.

Bolts need no mark, but should be boxed before shipment.

Brackets to be marked P. or I. (portal or intermediate), also R. or L.

Ornamental work to be marked R. or L.

Beam hangers to be numbered so as to correspond to the panel points to which they belong.

Rollers need no marks.

Fillers to be marked the same as the pins to which they belong.

Turn buckles and sleeve nuts, being attached to the rods before shipment, require no marks.

Jaws to be numbered to correspond to the panel points.

Washers need no marks: they should be boxed, or strung on bolts, before shipment.

Rivets need no marks, but should be boxed.

Pilot nuts need no marks, as there are so few of them required.

In addition to these marks, there should be others for those members which are to be riveted together in the field, and which are assembled in the shop when the rivet holes previously punched are reamed. These marks should be punched into the iron with a steel point, and should consist of one, two, three, or four dots upon each of the pieces so assembled, in order that no piece during erection will be put into the wrong place.

CHAPTER XX.

ERECTION AND MAINTENANCE.

The number of men required to erect an iron highway-bridge will vary from half a dozen to sixty, or even more, according to the length of span, width of roadway, location, and the time to be occupied in erection.

For any one bridge, there is a certain number of men which will be more economical than any other number; and it is only experience which will enable one to tell beforehand what this number is.

If there are too few hands, the work will lag, and difficulty will be experienced in handling heavy pieces: on the other hand, if there are too many men, the travelling expenses, and the time spent in travelling by the extra men, will be wasted, and the total amount of effective work done by each man per day will be less.

If, for any reason, there be need for haste, it will be economical to have a large force of men, notwithstanding the last-mentioned consideration. For raising ordinary county bridges, the author would recommend the following numbers of men in a gang: for pony truss-bridges, six men; for through-spans not exceeding eighty feet, seven men; from eighty to one hundred feet, eight men; from one hundred to one hundred and twenty-five feet, nine or ten men; from one hundred and twenty-five to one hundred and fifty feet, eleven or twelve men; from one hundred and fifty to one hundred and seventy-five feet, thirteen or fourteen men; from one hundred and seventy-five to two hundred feet, fifteen or sixteen men; from two hundred to two hundred and fifty feet, from sixteen to twenty-four men; and, from two hundred and fifty to three hundred feet, from twenty-four to thirty-six men. The long spans require a proportionately greater number of men, on account of the heavy sections. For the same reason, the numbers given should be increased, if the bridge be wider than the ordinary size. For city bridges, which are proportioned for heavy loads and for smaller intensities of working-stresses, the numbers should be increased from ten to twenty per cent. When great haste is necessary, the numbers should be doubled.

The most economical number of men will depend, too, upon their skill; for green hands work at a great disadvantage in bridge-raising. They do not know how to use their strength, and require the foreman to stand over them to show them how to do their work; besides, they are often so light-headed as to be unable to work aloft. Sailors make excellent bridge-men on account of both their agility and their training, which has taught them to do in a few minutes many a difficult little piece of work that ordinary hands would puzzle over for hours.

It is necessary to have a few experienced men in every gang; the more of them, the better, provided that their travelling expenses, and wages when travelling, do not render their employment too expensive.

The cost of raising a bridge depends more upon the foreman than upon the men. The best men will fail to do their full quota of work if the foreman be not energetic. Nor does it suffice to have simply a good worker for a foreman: he must know how to keep the gang busy, or they will stand by and look on, while he does all the work. He should also have their good will, or the progress of the work will be unsatisfactory.

The outfit for a gang to raise ordinary county bridges should be as follows: —

1 forge, 2 pairs of tongs, 2 button setts for each size of rivets, 5 drift-pins of each necessary size, 2 handle cold chisels, 1 handle drift pin, 12 cape chisels, 6 plain chisels, 3 wrenches for $\frac{5}{8}''$ nuts, 3 wrenches for $\frac{3}{4}''$ nuts, 2 riveting-hammers, 1 light sledge, 1 heavy sledge, 4 hand lines $\frac{3}{4}''$ diameter, 4 guy lines 1″ diameter by 130′ long, 2 fall lines 1″ diameter by 130′ long, 6 to 10 rope slings, 2 sets 8″ blocks, 2 snatch blocks, 5 steel

crowbars, 3 cross-cut saws, 2 augers 1″ diameter, 2 augers $\frac{3}{4}$″ diameter, 4 augers $\frac{5}{8}$″ diameter, 3 axes, 2 adzes, 8 timber trucks, 4 monkey wrenches, 4 chains, 2 crabs, 2 holding-on bars, 3 jack screws, several large wrenches for pins, and, if necessary, a pile-driver with its appurtenances. The ordinary weight of a pile-driver hammer varies from sixteen hundred to two thousand pounds; and the height of the driver is about thirty feet. The cost for such an apparatus complete is about two hundred or two hundred and twenty-five dollars.

If the gang be a large one, or if the span exceed one hundred and fifty feet in length, the numbers of some of the tools on the list will have to be increased; for instance, those of the bars, ropes, and timber trucks.

Bridge carpenters generally carry tools of their own: so, if there be much timber work in connection with the bridge, it will be sufficient to employ more carpenters, and not to purchase a larger outfit of carpenters' tools.

In getting ready to erect a bridge, the first step is to prepare the ground in the neighborhood of the site, so that there will be room to store the material and for the men to work. When the iron is received at the site, it should be checked, and any pieces from which the marks have been obliterated should be re-marked. The iron should be piled systematically, similar parts being grouped; and no iron should be allowed to lie upon the ground. It should be piled so that there will be no trouble in getting at any piece which may be required; and the parts to be used first should be placed nearest the bridge site.

The piers and abutments will be supposed to be erected, as this work does not aim to treat of foundations.

The next step is to put the falsework in place. If the bed of the stream be dry, or nearly so, the bottom hard, the distance from the bed to the lower chord less than eighteen feet; and if there be no danger of a sudden rise of water with a swift current, the floor and joists can be used for falsework.

If the distance from the bed of the stream to the bottom chord be greater than eighteen feet, and the other conditions be the same, timber bents on mud-sills will be required. The size of a mud-sill should vary from 6″ by 6″ to 12″ by 12″, according to the hardness of the ground, the weight upon the sill, and the height of the falsework. It is not necessary that the timbers be square. For ground not especially hard, wide timbers laid on their flats are preferable, because they distribute the pressure better.

If there be but one tier per bent, two posts will be enough, when the width of roadway does not exceed sixteen feet. These posts should batter about one inch to the foot, and should be covered by a cap about 6″ by 6″ or 8″ by 8″, long enough to project two feet beyond each truss. The upper ends of the posts should lie directly under the trusses, and the caps should be drift-bolted thereto. If the roadway exceed sixteen feet, there should be an intermediate vertical post. The bent should be braced by diagonal flat timbers, say from 2″ by 6″ to 3″ by 8″, according to their length, running in opposite directions, one on each side of the bent, and bolted or spiked to the posts and cap.

If there be two tiers in a bent, the inclined posts should batter two inches to the foot (or three inches if there be danger of high wind), and there should be a vertical post under each truss. Each tier should be braced with diagonal timbers, as before. The greater the danger of high wind, the more effectively should each bent be braced. Alternate consecutive bents should also be braced diagonally on their outer faces, and all consecutive bents should be connected by longitudinal horizontal planks well spiked to the caps. These planks will be useful, in fact often necessary, for the workmen in passing from bent to bent. If there be more than two tiers per bent, the batter of the inclined posts should be three inches to the foot. A good height for each tier is sixteen feet.

Where the bottom is soft, or where the water is deep and rapid, piles will be required to rest the bents upon. There should be from two to five piles per bent, according to the width of the latter; a pile being placed below each vertical and inclined post. These piles should be braced in the direction of the stream by flat timbers bolted thereto. Any bracing that may be given them transversely to the stream should be at such a distance above high-water level as to cause no obstruction to boats, trees, ice, or other floating objects.

If the bottom be bare rock, incapable of holding piles, the mud-sills must again be resorted to. They should be weighted so that they may be sunk into place, then drift-bolted to the rock. This can be done without the aid of a diver. Of course the sills must be firmly attached to the lower tier before being put down.

The tops of all piles should be cut off to an exact level, so that, when the bents are erected, the upper surfaces of the upper caps will lie in the same horizontal plane.

On these caps should be placed timber-beams stretching from one bent to the next, and lying immediately under the trusses: joists will answer the purpose. It is generally customary to place the bents under the panel points; but the author prefers to put them two feet to one side, so that the floor beams may be swung into place without taking down the falsework. This method may, and probably will, require an extra bent at one end of the span; so, if the bents be expensive, it is better to put one under each panel point, and remove the upper tiers before swinging the floor-beams. The level of the top of the longitudinal beams should be at least six inches below the feet of the posts, so as to permit of the use of camber blocks, like those shown on Plate VII. The angle which the contiguous faces make with the horizontal (less, of course, than the angle of friction of the wood) enables the under block to be easily knocked out when the span is to be swung.

The timbers for the caps and posts of the falsework are generally square, and the sizes for the latter are to be found from Table XXXIX., after the stresses in them have been ascertained as follows: —

Let

W_1 = weight per foot of the iron-work of the bridge,

W_2 = average weight per foot in height of one bent of falsework and the timbers whose weight it supports,

p = wind pressure per square foot,

A = area per lineal foot which the two trusses present to the wind (it is generally about five or six square feet),

A' = the average area subject to wind pressure per foot in height on one bent, and its share of longitudinal bracing,

l = panel length,

c_1, c_2, c_3, etc. = horizontal distance between centre lines of inclined posts measured along the caps,

d = depth of truss,

d_1, d_2, d_3, etc. = heights of the different tiers commencing at the top,

h = vertical distance between centre of chord and upper cap of bent,

and

θ = the angle which the inclined posts make with the vertical;

then

pAl = pressure on trusses at each panel point,

$pA'd_1$ = pressure on upper tier,

$pA'd_2$ = pressure on second tier from top,

$pA'd_3$ = pressure on third tier from top,

and the stresses F_1, F_2, F_3, etc., in the inclined posts of the first, second, and third tiers respectively, will be given by the equations,

$$F_1 = \frac{p \sec\theta}{c_2}\left[Al\left(\frac{d}{2} + h + d_1\right) + \frac{A'd_1^{\,2}}{2}\right] + \left(\frac{W_1 l}{2} + \frac{W_2 d_1}{4}\right)\sec\theta,$$

$$F_2 = \frac{p \sec\theta}{c_3}\left[Al\left(\frac{d}{2} + h + d_1 + d_2\right) + \frac{A'(d_1 + d_2)^2}{2}\right] + \left[\frac{W_1 l}{2} + \frac{W_2}{2}\left(d_1 + \frac{d_2}{2}\right)\right]\sec\theta,$$

$$F_3 = \frac{p \sec\theta}{c_4}\left[Al\left(\frac{d}{2} + h + d_1 + d_2 + d_3\right) + \frac{A'(d_1 + d_2 + d_3)^2}{2}\right] + \left[\frac{W_1 l}{2} + \frac{W_2}{2}\left(d_1 + d_2 + \frac{d_3}{2}\right)\right]\sec\theta,$$

$$F_4 = \text{\&c.} + \text{\&c.}$$

These formulas are obtained under the supposition that the inclined posts are not aided by the vertical ones, which supposition is necessary in order to avoid ambiguity: it would be correct, were the falsework on the verge of overturning. If the timber be green, the error thus made upon the side of safety is advantageous; but, if the timber be dry and of good quality, it is permissible to make a slight reduction in the size given by Table XXXIX. In applying the table, find the size of square timber required for a stress F_1 and length $d_1 \sec \theta$, that for a stress F_2 and length $d_2 \sec \theta$, etc., then take the greatest of these sizes.

The vertical posts should be strong enough to withstand a working-stress given by the equation,

$$S = \frac{W_1 l}{2} + \frac{W_2}{2}\left(d_1 + d_2 + \&c. + d_{n-1} + \frac{d_n}{2}\right),$$

where n is the number of the tier considered, and S the stress in the corresponding vertical post.

One dimension of the vertical posts should be the same as the side of the square which is the section of the inclined posts; so that the diagonal braces may be flush with the entire faces of the bents, and be bolted to the verticals without the intervention of filling-pieces.

These equations seem very long, and no doubt many practical bridge foremen would look upon them with disdain: nevertheless, if the falsework is to be designed by any other method than that of guessing, this is the way in which it should be done. The more elevated the bridge, the more important does it be ome to properly proportion the falsework. The values of W_2 and A' will have to be assumed, or roughly calculated, before applying the equations. The other quantities are, or should be, known. The value of p may be taken from ten to fifteen pounds per square foot, unless the situation be more than ordinarily exposed, when it may be taken at twenty pounds. Bridge companies can afford to risk the chance of a hurricane striking the bridge before it is swung.

The sections of the caps are generally made the same as those of the inclined posts. The caps should be dapped to receive both upper and lower ends of vertical and inclined posts. The vertical posts should be drift-bolted through the caps, the bolt being long enough to project five or six inches into each post; and the inclined posts should be held in place by wooden splice pieces, one on each side of the bent, projecting above and below the cap, and fastened at each end by a bolt passing through the two splice pieces and the post. This attachment may be used for the vertical posts instead of the drift bolts, if it be preferred. For additional security against slipping, a third bolt may be put through the splice pieces and the cap; or cleats may be nailed to the latter above and below, at the toe of each inclined post.

All bolt holes in timber should be accurately located and bored before the falsework is erected. On this account the bents should be all built after one pattern, so that the parts may be interchangeable. If the bents be of different heights, the variation may be effected in the lowest tiers. Bolts are always preferable to spikes for connecting timbers, especially when the falsework has to be taken down, and re-erected for another span. Care should be taken to avoid any unnecessary injury to the timber, in order that it may not be sold at too great a loss after the work is finished.

There should be at least two plank walks on top of the lower falsework, exterior to the trusses, and a runway midway between, formed of several joists set on edge for the purpose of bringing out the material thereon upon timber trucks.

The posts of the upper falsework should rest on the caps of the lower falsework, a few inches inside of the trusses, unless the bents are placed beneath the panel points, in which case they should be placed two feet to one side: they should be attached to the caps by splice timbers and cleats. The height of the upper falsework should be such that the upper surface of the cap will be at least six inches below the under sides of the upper chord sections, so as to permit of the use of camber blocks between.

The author would suggest that the end bents of upper falsework be made three or four feet higher than the others, and the use of four posts instead of two (one on the inside, and one on the outside, of each truss), in order to aid in raising and holding in place the heavy batter braces. After the latter are put in position, a horizontal timber may be firmly bolted to the bent at the level of the other bent caps, for the temporary flooring to rest upon. Stout beams stretching from bent to bent will be required as fulcra for the levers by which the chord sections are handled. The upper falsework should be braced by diagonal timbers, both longitudinally and transversely. The sizes of the posts should generally be about 6″ by 6″: when the trusses are high and the chord sections heavy, it might be well to increase the size to 7″ by 7″. The caps of the upper falsework should be deeper than their breadth; because they have to act as beams, and may be subjected to considerable shock when the chord sections are being put in place. The method of bracing shown on Plate VII. is specially advantageous in this respect.

In both upper and lower falsework, the diagonal bracing in planes parallel to the axis of the bridge should, for economy's sake, be placed between alternate pairs of bents; that is, every other space between bents should be braced. The end spaces should, however, be braced in any case.

Plate VII. gives an illustration of how the working-drawings for falsework should be made. For economy of space, the scale has been taken at an eighth of an inch to the foot; but it should, if intended for an actual case of framing, be four times as great. A drawing of this kind should be accompanied by bills of lumber and iron, prepared in a similar manner to that given in Chapter XIV. for the span. Measurements of distances between bolt holes should be both calculated and scaled. Those on Plate VII. were simply scaled, as the plate is intended for illustration only.

The foreman of the work should be provided with a blue print of the working-drawings for the bridge, unless the type of structure be one with which he is perfectly familiar. He must also be provided with a "Raising Bill," which should consist of a skeleton diagram of one truss, with the following information written thereon:—

Size of each truss strut, and tie, and mark for same, also number of pieces of same in a panel of one truss.

Diameters and lengths S. to S. of truss pins, with their marks.

Diameters, lengths, and marks of fillers for same.

Sizes and marks of all separate plates belonging to the trusses, each in its proper position.

A diagram for the lower lateral system, giving the following information: —

Sizes and marks of rods.
Positions of same, showing which eyes are to go next the trusses.
Sections, lengths, and marks of lateral struts.
Diameters and lengths of lateral pins, if any.
Diameters and lengths of fillers for same.
Sizes and marks of jaws, if there be any difference between them.

A diagram for the upper lateral system and portal bracing, giving the following information: —

Sizes and marks of rods.
Positions of same, showing which eyes are to go next the trusses.
Sections and marks of lateral and portal struts.
Diameters and lengths of portal pins.
Diameters and lengths of fillers for same.
Diameter and length under head of portal strut attaching bolts.

He should also be provided with a plan of the bottom chord packing (the transverse dimensions being exaggerated, so that the size of each piece may be written thereon), a bill of bolts, giving the number and position of each kind, and a clear statement of the system of marking the iron.

Before starting to erect the bridge, the foreman should study carefully all the plans, so that he will have a clear picture of the bridge in his mind's eye, and will not have to be continually referring to the drawings during the erection. On a work of any magnitude, there should be kept on hand a few standard nuts of each size ordinarily used, so that the loss of a nut or two will cause no delay: for the same reason there should be a few extra bolts of each size.

The material, as a general rule, is all piled on one side of the stream: the raising should therefore be commenced at the other side, so that the passage of the material will not interfere with the work. If there be no objection, the far end of the bridge should be the fixed one, so as to start from something permanent; but this is not absolutely necessary.

To illustrate the method of raising, take, for example, the bridge treated in Chapter XVI., and assume that the foundations, with their anchor bolts and falsework, are in place. The first thing to be done is to lay out the centre line of the bridge upon the falsework caps, marking it with a small-headed tack on each cap, then the centre lines for the trusses in the same way. This can be done either with a transit, or with a carpenter's chalk-line; care being taken to make the transverse measurements to the outer lines exactly perpendicular to the central line. A test of the accuracy of the perpendiculars can be made by the three, four, and five method, using a tape-line. Next, mark the exact positions of the panel points upon the longitudinal beams under the trusses, and place the camber blocks, levelling over them so as to make the lines joining the central points of their upper surfaces parallel to the curve of the chords. It is better to have the blocks a trifle high, say, an eighth of an inch near the centre, and a sixteenth of an inch near the ends.

Four small nails will hold each pair of camber blocks from slipping during the work, and they can be left so as to be easily extracted before swinging the bridge. Next transfer the centre lines of the trusses to the tops of the camber blocks, and mark accurately the first panel points from the fixed end, then, starting there, pack the chord bars of both chords. It might be convenient to have a few hard-wood pins to fit the holes pretty tightly, so as to aid in getting the bars properly placed longitudinally.

After the chord packing has made some progress, run out the two batter braces, and hoist them into place by means of pulleys attached to the cap of the first bent of falsework, which bent should have been previously guyed and braced so that it cannot possibly be disturbed by the effect of the pulleys. As soon as each batter brace is raised, and the anchor bolts pass through the holes in the shoe plate, the nuts should be tightly screwed down in order to aid in holding the batter brace in position.

It will not do, however, to rely solely on these, for the threads of the end bolts might be stripped: consequently a hard-wood supporting block must be strongly bolted to the two adjoining posts of the bent of the upper falsework. This block should have a bevelled edge, the angle of bevel being equal to the slope of the batter brace, so that the iron-work will not rest on a sharp edge of wood. If the lattice bars interfere with the bearing, as they are liable to do, rough notches can be cut in a minute on the bevelled face so as to bring the bearing upon the channels.

Meanwhile the end lower lateral strut, the portal struts, and the portal and end lower lateral rods, having been run out, the three struts are to be put into place; the upper ones being retained there by their connecting bolts, and the lower one by the end pins, which should also pass through the chord bars, fillers, and end lateral rods.

Such small portions of the structure as pins, fillers, and beam hangers, should not be brought out upon the falsework until required for use, for fear of their being lost overboard. Nothing more will be said about running out these and other small portions, but it will be assumed that they will be at hand when wanted. It should be an understood thing between the foreman and the men, that any one who drops any portion of the bridge into the water forfeits a certain amount of his wages. Such an arrangement will make green hands a little more careful than they are apt to be generally.

As the portal rods are adjusted by turn buckles with single tap ends, they may be omitted until after the portal struts are riveted to the batter-braces, because the riveters can then work to better advantage. They can be left upon the abutment until required.

Next run out, and hoist upon the falsework, by means of pulleys attached thereto and timbers used as levers, the end sections of the top chords, working them into place by the levers, and attaching them temporarily at the hips by bolts, putting in at the same time the end diagonals, but omitting the hip verticals and fillers, so that room may be left for the holding-on bars. The other ends of the chord sections rest on the camber blocks.

Next run out, and hoist into place, the first vertical posts, letting the upper ends lie in the open ends of the chord sections.

Now start the rivet gang at work on the portal, and let them follow up the work as it progresses, not leaving the portal until they have made the hip attachment, connected the portal struts, and put the brackets and ornamental work in place.

Next bring out the second sections of the top chords and the second set of diagonals. Raise the chord sections into place, as before, with pulleys and beam levers, holding them there until temporary bolts are put into a few holes through the connecting-plates, filling-plates, and channel webs, and until the pins are run through the posts, diagonals, and fillers. The latter, in this case, will not interfere with the riveting.

Next run out and put into place, as before, the second pair of posts; then bring on the third sections of the chords, the third set of main diagonals, and the first set of counters, putting all three into place as before, and so on until the end of the bridge is reached. Meanwhile the wooden lower lateral struts should have been framed, and the jaws attached to their ends.

Just before the riveters complete the riveting of the portal, the first upper lateral and intermediate struts should be run out, and bolted into place; but the upper lateral and vibration rods should be omitted, as they would be in the way of the riveters, and can be readily inserted afterwards.

About the time that one-half the span is erected, commence running out the lower lateral struts and rods, putting them into place, inserting the hip verticals and fillers, and coupling the lower chords into their final position, leaving the beam hangers lying horizontally, so that, when the longitudinal supporting timbers are removed, they will drop into their proper places.

A little before the riveters reach the end of the span, the upper lateral and vibration rods should be put into place, and screwed up *about* the right amount.

When the end of the bridge is reached by the riveters, and as soon as they have riveted the hip connection, and attached the main diagonals and hip verticals, the last couplings of the bottom chords can be made at the pedestals.

The shoes rest upon the rollers, which should have been put in *exactly* transverse to the direction of the bridge, and blocked so that they cannot move.

The last connection for each truss can easily be made by raising the hip either with levers or by jack-screws, and either pressing against the shoe with jack-screws abutting against blocks chained to the roller plate, or by attaching a pair of blocks to the pedestal and first panel-point lower chord pin.

After the final coupling has been made, and the riveting is finished, knock out the upper chord camber blocks, so as to bring all the weight of the upper part of the bridge upon the posts; then take down the upper falsework.

Next knock out the camber blocks of the lower chords, lowering them together gradually so as to bring no shock upon the bridge.

Next run out the first floor beam to the end of the bridge, and remove the runway of the second panel, in order that the beam may be dropped between the lateral struts and lateral rods, and swung into place, lowering it beneath the ends of the hangers, then raising it up, inserting the filling-plates, putting on the hanger plates, and screwing up the nuts. In this way attach all the floor beams, seeing that the hanger nuts are screwed up firmly, but not to such an extent as to endanger stripping the threads. Then bolt all the wooden lateral struts to the beams through the holes previously bored, which holes should be at least a quarter of an inch greater than the diameter of the bolts.

Next screw up every adjustable rod to the proper tension, which can be ascertained by the sound they make when tapped with a hammer.

Next wash off any mud or other impurity that there may be on the iron-work, and give it two good coats of paint wherever the brush will reach. The best kinds of paint to use are lead paints, when they can be obtained unadulterated; but they are at the same time the most expensive of all the paints used for iron-work. Iron oxide is a good paint, but requires more frequent renewal. The color should be such as to readily show any sign of rust: various shades of gray are efficient in this respect, and are at the same time pleasing to the eye.

There remains nothing now to be done except to put on the joists, floor, hand railing, and felly plank, a matter so simple that it is unnecessary to describe it here; the only point worthy of attention being, that the joists should be dapped one inch on to the lateral struts, and that they should go on so hard that it will be necessary to drive them into place. This can be accomplished by cutting each dap a sixteenth of an inch short, and bevelling the end of one dap slightly, in order to give the joists a start when they are being driven down. When they come to their bearings, they should be spiked to the lateral struts by a five-inch spike at each end, driven obliquely.

In regard to the flooring, Mr. James Owen, C.E., in a paper read before the American Society of Civil Engineers, specifies as follows: "Lay no plank wider than nine inches. This prevents wide joints in shrinkage. Bore all holes for the spikes to prevent splitting, and put no spike nearer than four inches to the end of the planking."

In long bridges of several spans, it may be economical to dispense with the upper falsework by using a travelling derrick, running upon wooden stringers, for the purpose of handling the heavy sections. Under these circumstances, the whole of the portal might be connected while lying upon the falsework, then hoisted into place in one piece, and supported there by shore timbers from the first bent of falsework. The bridge should be completed as the traveller retreats: otherwise there will be difficulty in carrying the members past the traveller. The material should be brought on cars within reach of the derrick.

The last thing to be done is to take down the falsework, and draw the piles from the bed of the stream. The latter is easily accomplished by a crab on the bridge; the rope being attached to the head of the pile, which is vibrated transversely in all directions while being lifted by the tension of the rope.

There is no reason why a well-designed iron highway-bridge, when properly cared for, should not last forever. Under loads which are light and slowly moving, compared to those of railroad-bridges, the iron cannot possibly wear out; and, when properly protected from the weather, it cannot rust. Of course the wooden parts of the structure must be replaced from time to time as they wear out or decay.

When knots begin to project above the surface of the floor, they should be adzed off, both for the comfort of those driving over the bridge, and to prevent vibration. After half an inch has been worn off one side of the planks, they should be turned over; and when another half-inch has been worn off, or before then if the wood show signs of weakness or decay, they should be replaced.

It would be well for county commissioners to buy all the lumber needed for renewal a year before required for use, so that it may be well seasoned.

Iron bridges should be thoroughly inspected for rust spots at least once a year; and, if any be found, the bridge should be repainted. One or two spots in places where something might have rubbed off the paint may be touched up with a brush; but, generally speaking, when rust spots begin to appear, it shows that two good coats of paint are required immediately.

The adjustable members should be tested occasionally by tapping with a hammer. This duty should not be intrusted to an ignorant workman, who will turn away on the nuts until he shears the thread, or breaks the rod. Whenever, in driving over a bridge, any of the iron-work rattles, it shows that something is out of adjustment. Generally speaking, a well-proportioned iron bridge will not get out of adjustment unless some one meddles with the nuts or turn buckles. With combination bridges it is a different matter, for the shrinkage of the wood may loosen the counters.

APPENDICES.

APPENDIX I.

A NEGLECTED CONSIDERATION IN HIGHWAY-BRIDGE DESIGNING.

SPECIFICATIONS for highway-bridges generally call for strength to resist a wind pressure of at least thirty pounds per square foot of exposed surface; but there are many such structures in the United States whose trusses would not, unaided, withstand this pressure. Granting that the lateral rods are large enough, that the upper lateral and portal struts have sufficient strength to resist both direct thrust and bending, and even that the lower lateral rod connection is all that could be desired, still the bridge may be far from fulfilling the requirements, as the following investigation will show: —

Let

$p =$ the assumed pressure per square foot,

and

$A =$ the area in square feet per lineal foot of the vertical projection of that part of the structure lying below a horizontal plane, which passes midway between the chords of a through Pratt-truss bridge (the windward truss and hand-rail are not supposed to shelter the leeward ones);

then

$pA = W =$ wind load per lineal foot for the lower lateral system when the bridge is empty.

Let

$A_1 =$ the total area of bridge per lineal foot exposed to the wind pressure,

$h =$ the vertical distance of the centre of pressure above the level of the bed-plate,

$l =$ the panel length,

$b =$ clear width between trusses,
$c =$ width of one truss,
$d =$ depth of trusses,
$W_1 =$ dead load per lineal foot for one truss,

and

$W_2 =$ reduced dead load per lineal foot for the windward truss;

then the overturning moment of the wind per lineal foot is pA_1h, and it has the same effect as that of a couple of lever-arm $b + c$, and force,

$$\frac{pA_1h}{b+c};$$

that is, the weight per foot on the leeward truss is increased, and that on the windward truss is decreased, by this amount, which gives the equation,

$$W_2 = W_1 - \frac{pA_1h}{b+c}.$$

Let

$n =$ number of panels in the bridge,

and

$n_1 =$ number of any panel, counting from the nearest end of the span;

then

$Wl =$ panel wind load,

and

$W_2l =$ reduced panel dead load.

The compression on the windward bottom chord of the n_1^{th} panel will be

$$\frac{n_1}{2}(n - n_1) W \frac{l^2}{b},$$

if we consider that the inclination of a lateral rod to a line perpendicular to the planes of the trusses is $\tan^{-1}\frac{l}{b}$. The tension in the same panel, due to the reduced dead load alone, is

$$(n_1 - 1)\left(\frac{n - n_1 + 1}{2}\right) W_2 \frac{l^2}{d},$$

except in the case of the first panel, to find the stress for which n_1 must be made equal to two.

Now, if this tension be less than the compression just found, the chord at the panel considered, if not a compression member, or if it be not externally aided, will buckle; for flat bars cannot be relied upon, when acting separately, to resist compression.

The following inequality should, therefore, hold true:—

$$(n_1 - 1)(n - n_1 + 1)\frac{W_2}{d} > n_1(n - n_1)\frac{W}{b}.$$

By inspecting the chord stresses in a few Pratt truss through bridges, it can be readily seen, that, if this inequality hold true for the second panel, it will hold true for all the others.

The three following cases are fair samples of bridges with which the author has met in his practice. The wind pressure assumed is thirty pounds per square foot.

(1) A 140′ span of 12′ clear roadway is 23′ deep, consists of seven panels, weighs 460 pounds per lineal foot, presents to the wind about six square feet of surface below the middle horizontal plane for every lineal foot, and about eight and a half square feet above and below. The centre of pressure is about 8 feet above the shoe plate, and the width of the truss is 1 foot.

These data give $W_2 = 73$, and, for the second panel,

$$(n_1 - 1)(n - n_1 + 1)\frac{W_2}{d} = 19,$$

and

$$n_1(n - n_1)\frac{W}{b} = 150.$$

(2) A 150′ span of 14′ clear roadway is 24′ deep, consists of eight panels, weighs 540 pounds per lineal foot, presents to the wind about six and a half square feet of surface per lineal foot for the lower lateral system, and about nine square feet above and below. The centre of pressure is about $8\frac{1}{2}$ feet above the shoe plate, and the width of the truss is about 1 foot.

These data give $W_2 = 117$, and, for the second panel,

$$(n_1 - 1)(n - n_1 + 1)\frac{W_2}{d} = 34.1,$$

and

$$n_1(n - n_1)\frac{W}{b} = 167.1.$$

(3) A 120′ span of 16′ clear roadway is 22′ deep, consists of six panels, weighs 530 pounds per lineal foot, and presents the same surface per lineal foot as in the last case. The centre of pressure is about $7\frac{1}{2}$ feet above the shoe plate, and the width of the truss is 1 foot.

These data give $W_2 = 146$, and, for the second panel,

$$(n_1 - 1)(n - n_1 + 1)\frac{W_2}{d} = 33.2,$$

and

$$n_1(n - n_1)\frac{W}{b} = 97.5.$$

In all these cases

$$(n_1 - 1)(n - n_1 + 1)\frac{W_2}{d} < n_1(n - n_1)\frac{W}{b}.$$

How is it, then, that more bridges do not fail by the buckling of the bottom chord under wind pressure? For two reasons. First, the probability of a bridge ever being subjected to a pressure of thirty pounds per square foot over its whole length is very small; and, second, that in a well-built bridge, where the joists are dapped to the floor beams, the joists would take up the compression that would tend to buckle any panel of the chord except the first.

In view of the fact of the small chance that a bridge has of ever being subjected to the assumed pressure, it would be legitimate to trust somewhat to the stiffness of the joists in cases where

$$(n_1 - 1)(n - n_1 + 1)\frac{W_2}{d} < n_1(n - n_1)\frac{W}{b},$$

and not to make the chords stiffened throughout, except in short spans.

But, as the joists cannot stiffen the end panels, the chords in these panels should be proportioned to resist the compression due to the difference between the longitudinal component of the greatest stress in the end lateral rod, including the initial tension, and the *reduced* dead-load stress, whenever the former is in excess of the latter. It is often well, for the sake of both rigidity and appearance, to stiffen the chords in the second panels when those in the first panels are stiffened.

APPENDIX II.

DEMONSTRATION OF FORMULA FOR FLOOR BEAMS.

LET the notation be the same as given on p. 19, viz.:—

A = area of bottom flange in square inches,
A' = area of web in square inches,
A'' = area lost by a rivet hole in square inches,
W = the uniformly distributed load in tons,
L = length of beam in feet between centres of supports,
D = depth in feet between centres of gravity of flanges,

and

T = intensity of working tensile stress in tons.

The moment at the centre of the beam is $\frac{WL}{8}$. Let us take the centre of moments at the middle of the web, which will correspond with the neutral surface, if we assume, which is nearly true, that the upper and lower flanges are of the same area, and are subjected to numerically equal stresses.

The moment of the load is resisted by the sum of the moments of the flange stresses and those of the web stresses. The sum of the moments of the flange stresses is

$$2(A - A'')T \times \frac{D}{2} = (A - A'')TD.$$

The resisting-moment of the web stresses is found as follows:—

The resisting-intensity of stress on the fibre most remote from the neutral surface may be taken equal to T; then that for any fibre at the distance x will be, by the common theory of flexure, $\frac{2Tx}{D'}$. The stress on an elementary area at this distance will then be $\frac{2Tx}{D'} \cdot bdx$, where b and D' are respectively the width and depth of the web. The moment of this stress is $\frac{2Tx^2}{D'} \cdot bdx$, integrating which between the limits

$$x = +\frac{D'}{2} \text{ and } -\frac{D'}{2}$$

gives, for the total resisting-moment of the web,

$$\frac{2bT}{D'}\int_{-\frac{D'}{2}}^{+\frac{D'}{2}} x^2 dx = \frac{2bT}{3D'}\left[\left(\frac{D'}{2}\right)^3 - \left(-\frac{D'}{2}\right)^3\right] = \frac{bTD'^2}{6} = \tfrac{1}{6}A'TD'.$$

Equating-moments gives

$$\frac{WL}{8} = (A - A'')TD + \tfrac{1}{6}A'TD'.$$

If we put D for D', we will commit a small error on the side of safety; then will

$$\frac{WL}{8} = TD(A - A'' + \tfrac{1}{6}A'),$$

and therefore

$$A = \frac{WL}{8DT} - \tfrac{1}{6}A' + A''. \qquad \text{Q. E. D.}$$

If M be the moment at any section of the beam, and R the intensity of stress on the fibre most remote from the neutral surface at the same section when the beam is fully loaded, we can write the equation

$$M = ARD + \tfrac{1}{6}A'RD,$$

where A is the area of the top flange, and D' is, as before assumed, equal to D; from which we have

$$R = \frac{M}{D(A + \frac{1}{6}A')},$$

and

$$S = RA = \frac{M}{D\left(1 + \frac{A'}{6A}\right)},$$

where S is the stress on the upper flange at the section considered. This last formula is useful in determining the rivet spacing in the flanges of built floor beams and plate girders.

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$\frac{bTD'^2}{6} = \frac{1}{6}A'TD'$.

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APPENDIX III.

METHOD OF FINDING THE LENGTH OF THE LONG DIAGONALS IN A DOUBLE-INTERSECTION BRIDGE.

Let

l = panel length of bottom chord = GD or DB in the accompanying diagram,

c = half increase of panel length in top chord,

d = depth of truss between centres of chords = AB,

α = angle between radial line at panel point and perpendicular to lower chord;

then

$$\alpha = \sin^{-1}\frac{c}{d},$$

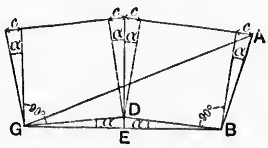
and

$$DE : c :: l : d,$$

or

$$DE = \frac{cl}{d}.$$

$$BG = 2GE = 2\sqrt{l^2 - \frac{c^2l^2}{d^2}} = 2l\sqrt{\frac{d^2 - c^2}{d^2}}.$$



When the camber is small, BG can be taken equal to $2GD$.

In triangle ABG, AB and BG are known, also angle

$$ABG = 90° + 2\alpha.$$

$$AB + BG : AB - BG :: \tan \tfrac{1}{2}[180^\circ - (90^\circ + 2\alpha)] : \tan \tfrac{1}{2}[BAG - BGA];$$

$$\therefore \quad BAG - BGA = 2 \tan^{-1}\left[\frac{AB - BG}{AB + BG}\tan\left(45^\circ - \alpha\right)\right].$$

Again :

$$(BAG - BGA) + (BAG + BGA) = 2BAG = (BAG - BGA) + (90^\circ - 2\alpha),$$ which gives BAG;

also

$$BGA = 180^\circ - (BAG + 90^\circ + 2\alpha) = 90^\circ - (BAG + 2\alpha):$$

finally,

$$AG = AB \cos BAG + BG \cos BGA =$$ length of diagonal required.

ADDENDA.

ADDENDA.

IN an otherwise very favorable review of this treatise by "The American Engineer," there was pointed out a serious objection to the usual attachment of a floor beam by four hangers.

In the words of the review, "the inner loop will take nearly if not quite all the load at the panel point, when the bridge is first adjusted; and this not only becomes constrained itself, but also overstrains the inner tension brace.* The number of inner hangers which are constantly working loose, presumably by stretching, in railroad-bridges in which this detail is used, demonstrates its unsatisfactory character."

The author has long recognized the inequality of distribution of floor-beam load between the inner and outer hangers, but considered that the low intensity of working-stress on these members would compensate for the objectionable inequality. Such has been also, in all probability, the opinion of most American engineers; for beams, when not riveted to the posts, are nearly always suspended by four hangers. The fact of the inner hangers working loose can have been only lately discovered. It shows, however, that this detail needs improvement; and as the aim of this treatise is to design structures not only equal, but in some respects superior, to the best American bridges, it becomes necessary to correct the newly discovered fault.

The most simple manner of so doing is to use single beam

* Main diagonal.

hangers; but this method will not always work, owing to the great bending-movements which they produce upon the pins. For instance, take the case of a 20′ panel of a Class A bridge having a 24′ clear roadway, and two 6′ sidewalks. The weight supported by each single hanger would be about 23 tons, and the distance between centres of main diagonals would not be far from ten inches. These data give a vertical bending-moment upon the pin equal to 57.5 inch-tons, which *alone* would call for an iron pin $4\frac{3}{8}''$ in diameter; but, when combined with the horizontal moment, it would require a much larger pin than a practical designer would care to employ.

The double hangers in such a case are a necessity, but the connection must be such as to distribute the load *equally* upon them. Such a distribution can be assured by using the following detail:—

On the under side of the beam at each end is attached by four rivets a plate about five-eighths of an inch thick, six inches long, and as wide as, or a little wider than, the beam flange. This plate is placed symmetrically to the plane of the truss; and the middle of the under side is grooved so as to receive one-sixth of the surface of a pin about two inches in diameter, which rests in a similar groove on the top of the beam-hanger plate. The lateral dimensions of this plate will be slightly greater than usual, but the thickness need not exceed three-quarters of an inch. To prevent the plate from rupture by bending, there are attached to the under side by countersunk rivets two angle irons, or plates bent into the form of angle irons, the vertical legs being connected by countersunk rivets, which in the neighborhood of the pin pass as nearly as may be through the neutral surface of the T beam, and elsewhere near the lower edges of the angles.

As the axis of the pin is parallel to the length of the bridge, the vertical legs must be transverse thereto. This detail will be readily understood by referring to Plate VIII.

To illustrate how to find the sizes of the stiffening plates, number of rivets required, etc., let us design a beam-hanger plate, when the total weight upon the four hangers is forty tons. The centres of the beam-hanger holes may be assumed to be situated on the corners of a six-inch square. This would make the bending-moment on the plate thirty inch-tons, which would be resisted by the T-shaped section of the two bent plates combined with the uncut portion of the beam-hanger plate below the pin. Assuming the latter thickness $\frac{1}{2}''$, and the plate stiffeners to be of $6'' \times 6'' \times \frac{1}{2}''$ angle iron, would make the T about $12'' \times 6\frac{1}{2}'' \times 1''$, the centre of gravity of which is about $5''$ above the bottom. The moment of inertia is, therefore,

$$\tfrac{1}{12} \times 12 \times (1.0)^3 + 12 \times (1.0)^2 + \tfrac{1}{12} \times 1 \times (5.5)^3 + 5.5 \times (2.25)^2 = 54+.$$

The resisting moment is given by the equation

$$M = \frac{RI}{d_1}:$$

so taking $R = 4$ tons,

$$M = \frac{4 \times 54}{5} = 43.2.$$

As the bending-moment was thirty inch-tons, the sizes assumed are ample.

It will be well to use three-quarter-inch countersunk rivets (the largest possible), so as to make the different portions of the T head act together.

There is a tendency to bend the plate in a direction at right angles to the one considered, the moment for which is fifteen inch-tons on each side of the pin. This will be resisted by a couple whose forces act as compression on the top plate of the T, and tension on the rivets near the bottom of the angles. Taking the centre of moments at the middle of the top plate, and the distance therefrom to the horizontal centre line of the rivet holes as $4\frac{3}{8}$ inches, will make the tension on the rivets $\frac{15}{4.375} = 3.42$ tons. Using an intensity of only two and a half tons, because of the initial tension on the rivets, will make the section required 1.37 square inches: consequently two $1''$ rivets will be sufficient.

The difference in the total weight of iron per lineal foot caused by the use of this detail will be from six to ten pounds, which weight should be added to those given in Tables I., II., and III., under the heading "Floor System," whenever this style of beam-hanger plate is to be employed.

It will be noticed in the diagram on Plate VIII., that the floor-beam stiffeners at the support are placed close together so as to take up the vertical reaction of the hangers transferred by the auxiliary pin. The sectional area of these stiffeners should be about equal to that of the hangers.

Plate VIII. illustrates a detail by which the foot of a post may be riveted or bolted to the floor beam, for the purpose of aiding the distribution of lower lateral rod stresses among the chord bars. This is accomplished by means of a jaw plate between the post channels, and by turning up the ends of the exterior re-enforcing plates, so as to permit the passage of rivets or bolts. The latter may be considered preferable; as they need not be screwed up very hard, but should fit with great accuracy in the holes. Their object is to prevent torsion of the post, but not to aid the beam hangers in resisting tension.

When this detail is employed, the lower chord pin at the first panel point should pass through holes bored in bent plates, which are to be well riveted to the floor beam.

It is probable that most bridge designers will consider this arrangement to possess too much refinement for highway-bridges, preferring to trust to the rigidity of the joists as suggested on p. 218.

There is no doubt, though, about its being a detail that could be advantageously employed in railroad-bridges.

When vertical sway bracing is used, the detail for the upper lateral strut connection, shown on Plate VIII., will be foundt o be an improvement on the one previously described, in that it obviates field riveting.

It consists in the use of a double jaw on the end of the lateral strut, and two nuts of different diameters, the pin being doubly shouldered. The office of the inner and larger nut is to press the end of the strut against the chord; and that of the outer one, to take up the pull of the bent eyes, the injurious effect of which is mitigated by the inner jaw plate.

When no vertical sway bracing is used, the detail described on p. 98 will probably be preferable; because it involves the spreading apart of the lateral strut channels, and thus furnishes a greater resistance to bending the strut. The use of the improved detail will not affect the sizes of the lateral strut channels as given in Table XXV.

Plate VIII. illustrates also an improved connection for the portal struts, avoiding the necessity for field riveting. The increased depth of the jaw plate at the pin hole is an important feature, its object being to resist the bending effect of that component of the stress in the portal rods which is parallel to the length of the batter brace.

Whenever the portal rods exceed $1\frac{1}{4}''$ in diameter, this improved shape of jaw plate should be employed.

GLOSSARY OF TERMS.

GLOSSARY OF TERMS.

Adjustable Member. — A member of a bridge the length of which can be increased or diminished at will.

Angle Iron. — Iron rolled into the shape shown in section on Plate II., Fig. 3.

Apex. — The intersection of a brace with a chord or flange; called also a panel point.

Axis of Symmetry. — A line dividing an area into two parts equal and similar to each other, and similarly disposed to the line.

Bar. — A piece of iron flat or square in section.

Batter. — Slope, or inclination, to the vertical; usually measured by the tangent of the angle, or so many inches to the foot.

Batter Brace. — The inclined end post of a bridge. (Plate I.)

Beam. — A member intended to resist bending.

Beam Hanger. — A rod or square bar supporting a floor beam from a chord pin. (Plate I. and Plate II., Fig. 13.)

Beam-hanger Nuts. — Nuts on the ends of beam hangers, serving to press the floor beam against the feet of the posts or against the chord heads. (Plate II., Fig. 13.)

Beam-hanger Plate. — A plate placed beneath the end of a floor beam for the beam-hanger nuts to rest against. (Plate II., Fig. 13.)

Beam-trussing Posts. — Posts for trussing beams. (Plate II., Fig. 16.)

Beam-trussing Rods. — Diagonal rods for trussing beams. (Plate II., Fig. 16.)

Bearing. — A resting-place, usually for a pin or rivet.

Bearing-Pressure. — The pressure on a bearing.

Bed Plate. — A plate to distribute pressure upon masonry. (Plate III.)

Bending-Moment. — The moment of a force or forces which bend or tend to bend a piece.

Bending-Stress. — The stress produced in a piece by bending.

Bent. — A frame of timber or iron, usually the former, as a bent of false-work.

Bent Eye. — An eye on the end of a bar, the plane of which makes an angle with the direction of the length of the bar.

Bevel. — The slope on the end of a piece.

Bill of Material. — A list of various portions of material giving dimensions and weights, or other quantitative measurements.

Block. — A system of one or more pulleys or sheaves, so arranged in a frame or shell as to multiply the power of the rope passing around them, or to change its direction.

Board Measure. — The measure of timber, the unit being a piece one foot square and one inch thick. Timber is sold at so much per thousand feet board measure, usually written, per M. b. m.

Bolt. — An iron rod with a square head at one end, and a thread and nut at the other.

Brace. — Generally a strut, but sometimes the term is applied to a tie.

Bracket. — A knee or knee brace to connect a post or batter brace to an overhead strut. (Plate I. or Plate II., Fig. 12.)

Built-Beam. — A beam made up of plates and angles riveted together. (Plate II., Fig. 13.)

Burr. — A rough edge or ridge left by a tool in cutting metal. The term is sometimes used for a nut.

Button Sett. — A tool for forming the heads of rivets.

Camber. — The upward curvature of a truss. It is measured by the height of the middle point of the centre line of the lower chord above the line joining the centres of end pins.

Camber Blocks. — Blocks of wood used in erection, so placed as to be easily removed (Plate VII.)

Cape Chisel. — A tool for cutting iron. It consists of a rounded edge on the end of a short rod. The edge is very obtuse, so as not to break easily.

Centre of Gravity. — That point of a body about which the weights of all the different portions balance.

Channel, or Channel Bar. — Iron rolled into the shape shown in section on Plate II., Fig. 1.

Check Nut, or Lock Nut. — A contrivance to prevent a nut from turning when subjected to shock.

Chord. — The upper or lower part of a truss, usually horizontal, resisting compression or tension. (Plate I.)

Chord Bar. — A member of the chord which is subjected to tension. (Plate I.)

Chord Head. — The enlarged end of a chord bar, through which the pin passes.

Chord Packing. — The arrangement of the bottom chord of a truss.

Clear Headway. — The vertical distance from the upper surface of the floor to the lowest part of the overhead bracing. It is a measure of the height of the highest vehicle that could pass through the bridge.

Clear Roadway. — The horizontal distance, measured perpendicularly to the planes of the trusses, between the inner edges of the batter braces. It is a measure of the width of the widest vehicle that could pass through the bridge.

Cleat. — A narrow strip of wood nailed to something for the purpose of keeping a piece of work in its proper place.

Co-efficient of Friction. — A numerical quantity, which, multiplied into the normal pressure, gives the frictional resistance. It is equal to the natural tangent of the angle of repose.

Cold Chisel. — A tool for cutting iron.

Column. — A pillar or strut; a long member which resists compression.

Component. — One of the parts into which a stress may be resolved or divided.

Compression. — A stress which tends to shorten the member which is subjected to it.

Concentrated Load. — A load which is, or may be considered, collected at one or more points.

Connecting Chord Heads. — Chord heads used to connect bottom chord channels to pins. (Plate II., Fig. 10.)

Connecting-Plate. — A plate used for connecting two pieces.

Continuous Spans. — Consecutive spans connected over the points of support.

Counter. — An adjustable diagonal which is not subjected to stress by a uniformly distributed load covering the bridge. (Plate I.)

Countersunk Rivets. — Rivets, the heads of which are let into one or both of the plates which they connect, so as to leave a flush surface or surfaces.

Couple. — Two equal and parallel forces not acting in the same line.

Cover Plate. — A plate used to cover a joint, or to connect two pieces of the top chord plate. (Plate II., Figs. 11 and 12.)

Crab. — A slow-motion machine, worked by a crank for the purpose of winding a rope upon a drum, thereby raising a heavy weight.

Dap. — To notch timber onto its bearing.

Dead Load. — The weight of all the parts of the bridge itself, and any thing that may remain upon it for any length of time.

Deck Bridge. — A bridge in which the passing loads come upon the upper chords or the upper ends of the posts.

Deflection. — Motion laterally, or at right angles to the length of the piece. It is also used for the amount of motion, and is generally expressed in inches.

Depth of Truss. — The vertical distance between the centre lines of upper and lower chords.

Diagonal. — A member running obliquely across a panel. In this work all the diagonals except the batter braces are tension members.

Diagram of Stresses. — A skeleton drawing of a truss, upon which are written the stresses in the different members. (Plate V.)

Double Intersection. — The style of truss where the diagonals cross the posts at the middle of their length, as in the bridge shown on Plate I.

Double-riveted Lacing. — Lacing in which each bar is connected by two rivets at each end. (Plate II., Fig. 13.)

Drift Bolt. — A round or square piece of iron, usually from one to three feet long, without head or nut, used to connect timbers.

Drift Pin. — A slightly tapering rod of hard steel, used for making rivet holes coincide. Its use is more convenient than advisable.

Effective Area. — The gross area of a section, less that lost by rivet or pin holes; the net area.

Elastic Limit. — That intensity of stress at which the ratio of stress over strain commences to show a decided change. . For wrought-iron it is from twelve to fifteen tons.

Erecting-Bill. — A bill of material for a bridge, so arranged as to facilitate the finding and placing of members during erection.

Expansion Joint. — The connection of pedestal to bed-plate, shown on Plate III.

Expansion Rollers. — A set of half a dozen or more turned rods of exactly the same diameter, placed under the shoe plate at one end of a truss to permit of expansion and contraction. (Plate II., Fig. 9.)

Extension Plate. — A plate riveted to the end of a strut channel, and projecting beyond it, to permit of the passage of a pin. (Plate II., Fig. 12.)

Eye. — A hole in the end of a member to permit of the passage of a pin.

Eye Bar. — A bar with an eye at each or one end.

Factor, or Factor of Safety. — The ratio of ultimate load to greatest allowable working-load. This term is getting out of favor among engineers, as its use has been somewhat abused. There is no such thing as a factor of safety for a well-proportioned bridge, for each member should have an intensity of working-stress proportionate to the character and amount of work which it has to perform.

Fall Line. — A rope used in erection for raising and lowering weights.

Falsework. — Temporary timber work to support a bridge during erection.

Felly Plank. — A guard rail so placed as to catch the felly of a wheel, and thus prevent the vehicle from striking the truss. (Plate II., Fig. 13.) In wide bridges a felly plank is often placed midway between the trusses, to prevent vehicles passing from one side of the bridge to the other.

Field Riveting. — Riveting done in the field, or during erection. It is the poorest and most expensive kind of riveting.

Fixed End. — An end of a strut so firmly connected as to prevent all motion of the strut in the neighborhood of the end.

Filling-Plate. — A plate the function of which is to make flush two surfaces (Plate II., Fig. 12.)

Filler. — A small ring of iron or piece of pipe placed on a pin in order to keep in position the members coupled thereon.

Fixed Load. — A load remaining permanently, or for a considerable length of time, upon a structure or portion of a structure.

Flange. — The upper or lower chord of a beam. It is the principal part for resisting either compression or tension.

Flexure. — Bending.

Floor, or Flooring. — That part of the bridge which directly receives the travel. (Plate II., Fig. 13.)

Floor Beam. — A beam to support a portion of the floor and its load. (Plate I. and Plate II., Fig. 13.)

Forge. — An apparatus for heating iron.

Framing. — The carpenter work on timber.

Giasticutus Rods. — A term (perhaps unauthorized, but in common use among bridge builders) to denote a small horizontal rod connecting the middle points of two adjacent posts of the same truss, for the professed purpose of fixing or holding the posts at the middle in order that they may be figured for half-length. The benefit derived therefrom is more imaginary than real.

Girder. — Any structure to cross a chasm or opening. The term is generally applied to short structures for places where it is not advisable to use trusses; for instance, a plate girder, or a rolled girder.

Guard Rail. — See felly plank.

Guys, or Guy Lines. — Lines for bracing the top of a pole, derrick, or any similar apparatus.

Gyration. — See radius of gyration.

Hammered Head. — A head formed on the end of a bar by hammering.

Hand Lines. — Small ropes used in erection.

Hand Rail, or Hand Railing. — An iron or wooden frame placed on or near the outside of a bridge in order to prevent persons or animals from falling therefrom. (Plate IV., or Plate II., Fig. 13.)

Hand-rail Cap. — The upper longitudinal timber or timbers of a wooden hand-railing. (Plate II., Fig. 13.)

Hand-rail Post. — Post for supporting a hand railing. (Plate II., Fig. 13; Plate IV.)

Headway. — See clear headway.

Hinged End. — An end of a strut connected only by a pin.

Hip. — The place at which the top chord meets the batter brace.

Hip Joint. — The joint of the top chord and batter brace.

Hip Vertical. — A rod hung from the pin at the hip for the purpose of suspending the floor beam.

Holding-on Bar. — A lever to hold against one end of a rivet while the head at the other end is being formed with a button sett.

Hub Plank. — A plank to protect the iron-work of the truss from being struck by the hubs of passing wheels. (Plate II., Fig. 13.)

I-Beam. — A piece of rolled iron of the section shown on Plate II., Fig. 2.

Initial Tension. — The tension caused in any adjustable member by screwing up the adjusting apparatus.

Intensity. — The intensity of a stress is the amount of stress upon a square inch of section.

Intermediate Strut. — An overhead strut in high bridges, attached to the posts of opposite trusses, and lying between the upper lateral strut and the floor. In deck bridges, if used at all, it would lie between the upper and lower lateral struts. (Plate I.)

Jaw. — A connection on the end of a strut similar to that shown on Plate II., Fig. 13.

Joint. — A place where two abutting or lapping pieces are connected.

Joist. — A timber beam that supports part of the floor and its load. (Plate I. and Plate II., Fig. 13.)

Knee, or Knee Brace. — See bracket.

Lacing. — A system of bars, not intersecting each other at the middle, used to connect the two channels of a strut in order to make them act as one member. (Plate II., Fig. 12.)

Lacing-Bar. — A bar belonging to a system of lacing.

Lateral Rod. — A tension diagonal of a lateral system. (Plate I.)

Lateral Strut. — A compression member of a lateral system. (Plate I.)

Lateral System. — A system of tension and compression members forming the web of a horizontal truss connecting the opposite chords of a bridge. Its purposes are to transmit wind pressure to the piers or abutments, and to prevent undue vibration from passing loads.

Latticing. — A system of bars crossing each other at the middle of their lengths, used to connect the two channels of a strut in order to make them act as one member. (Plate II., Fig. 12.)

Lattice Bar. — A bar belonging to a system of latticing.

Leg. — One of the two portions of an angle iron separated from each other by the bend.

Lever Arm. — The perpendicular from the centre of moments to the line of action of a force. The lever arm of a couple is the perpendicular distance between the lines of action of the two equal and parallel forces.

Live Load. — The moving or passing load upon a structure.

Linville Truss (also called "Double Quadrangular," "Whipple," and "Double System Pratt" truss). — A truss with vertical posts and diagonal ties spanning two panels. It is the truss represented on Plate I.

Lock Nut. — See check nut.

Loop Eye. — An eye on the end of a rod or square bar, elongated into the form of a loop, as shown on Plate II., Fig. 16.

Lower Falsework. — The falsework below the level of the lower chords.

Main Diagonal. — A tension member of a truss, sloping upward towards the nearer end of the span. Main diagonals in iron bridges are not adjustable.

Moment. — The product of a force by its lever arm.

Moment of Inertia. — Represented by the equation, $I = A\rho^2 = \Sigma r^2 dA$, where A is the area of the section considered, ρ the radius of gyration, and r the distance of any point from an assumed line lying either in the surface or outside of it: in other words, the moment of inertia of a surface about any axis is the product of the area by the square of the radius of gyration; or it is the summation of the products of each differential of the area by the square of its distance from the axis. If the axis lie in the surface, the moment of inertia is called a surface moment of inertia; while, if the axis be perpendicular to the surface, the moment of inertia is called a polar moment of inertia.

Monkey Wrench. — A wrench capable of being adjusted so as to fit nuts of different sizes.

Moving Load. — See live load.

Mud-Sill. — A timber, usually from 6″ by 6″ to 12″ by 12″, at the bottom of a bent. It is laid horizontally in a trench, and the posts of the bent rest upon it.

Name Plate. — A plate of iron placed in a conspicuous position on a bridge. containing the name of the maker or designer of the structure.

Negative Rotation. — Rotation in a direction opposite to that of the hands of a watch.

Net Section. — See effective area.

Neutral Surface. — That part of a member subjected to bending, which is neither extended nor compressed. In symmetrical wrought-iron beams, with equal or nearly equal flanges, it is taken to be at the centre line of the web.

Nut. — A small piece of iron with a threaded core to fit on the screw end of a bolt, rod, or bar. (Plate II., Fig. 6.)

Order Bill. — A form of bill used in ordering material from the manufacturers.

Ornamental Work. — Fancy work at the portals of a bridge to give it architectural effect. (Plates I. and VI.)

Overhead Bracing. — The upper lateral or vertical sway bracing in through bridges. The term is usually applied to the vertical sway bracing, if there be any; if not, to the upper lateral bracing.

Packing. — See chord packing.

Panel. — That portion of a truss between adjacent posts or struts in Pratt truss bridges; called also a bay.

Panel Length. — The distance between two adjacent panel points of the same chord.

Panel Point. — See apex.

Pedestal. — The foot of a batter brace or end post. (Plate II., Fig. 9.)

Permanent Set. — The alteration in length of a piece of material which has been subjected to stress, remaining after the stress has been removed.

Pillar. — See column.

Pilot Nut, or Pin Pilot. — A nut, one end of which is a truncated cone, used to protect the thread on the end of a pin when the latter is being driven into place. (Plate II., Fig. 5.)

Pin. — A cylindrical piece of iron used to connect bridge members. (Plate II., Fig. 5.)

Pitch. — The distance between centres of consecutive rivets of the same row.

Plane of Symmetry. — A plane dividing a body into two equal and symmetrical parts similarly disposed in reference to the plane.

Plant. — Tools and apparatus used in construction.

Plate. — A piece of flat iron wider than a bar. The common distinction between the two is that a plate is attached to something else, and acts with it, while a bar is an independent member.

Plate Girder. — A beam, built of plates and angles, used to span a small opening, generally less than forty feet.

Pony Truss. — A truss so shallow as not to permit the use of overhead bracing.

Portal. — The space between the batter braces at one end of a bridge. Sometimes the term is applied to the portal bracing, though incorrectly.

Portal Bracing. — The combination of struts and ties in the plane of the batter braces at a portal, which transfers the wind pressure from the upper lateral system to the abutment or pier.

Portal Strut. — A strut belonging to the portal bracing. (Plate I.)

Positive Rotation. — Rotation in the direction of the hands of a watch.

Post. — A vertical strut. (Plate I.)

Pratt Truss (called also the "Murphy-Whipple," or "Quadrangular" truss). — A single-intersection truss with vertical struts and diagonal ties.

Quadrangular Truss. — See Pratt truss.

Radius of Gyration. — The radius of gyration of any surface in reference to an axis is the distance from the axis to that point of the surface in which, if the whole area were concentrated, the moment of inertia in reference to the axis would be unchanged. It is therefore equal to the square root of the ratio of the moment of inertia over the area.

Ream. — To enlarge a rivet hole.

Reamer. — A tool for enlarging rivet holes.

Re-enforcing Plate. — A plate used for the purpose of providing additional pin bearing, or strength, to compensate for material cut away. (Plate II., Figs. 11 and 13.)

Resolve. — To divide a force into component parts.

Rivet. — A short piece of round iron tightly connecting two or more thicknesses of metal, and having, when in place, a head at each end.

Roadway. — The passage-way of a bridge for vehicles; usually means clear roadway, *q. v.*

Rod. — A piece of round iron.

Rolled Beam. — An I-beam. (Plate II., Fig. 2.)

Roller. — See expansion roller.

Roller Frame. — A light frame of iron for holding the rollers in position. (Plate II., Fig. 9.)

Roller Plate. — The plate upon which the rollers rest, and which itself rests upon the masonry.

Rope Sling. — See sling.

Run. — A line, or string; as, a run of joists.

Set. — The extension or compression of a piece of material under stress.

Shear, or Shearing-Stress. — The resistance which a body offers to the passage, or to the tendency to passage, of one section along the next consecutive section.

Shipping-Bill. — A list of portions of a bridge, arranged in a manner to facilitate counting and checking when the material is received after shipment.

Shoe. — Another term for pedestal, *q. v.*

Shoe Plate. — The plate on the under side of the shoe, resting on the rollers, bed-plate, or masonry.

Side Bracing. — A bracing for pony trusses to attach the panels of the top chord to the floor beams prolonged, in order to fix the panel points of the top chord. (Plate III.)

Sidewalks. — Roadways at the sides of a bridge for foot-passengers only.

Single Intersection. — That style of truss in which the diagonals do not cross the posts. It is represented in skeleton on Plate V.

Skeleton Drawing. — A drawing which shows only the centre lines of members, such as a diagram of stresses. (Plate V.)

Skew Bridge. — A bridge in which the horizontal lines joining corresponding panel points of the opposite trusses are oblique to the planes of the trusses.

Sledge. — A heavy hammer, or mallet.

Sleeve Nut. — An elongated nut, the core at one end having a right-hand thread, and that at the other a left-hand thread. Its office is to lengthen or shorten a tension member. (Plate II., Fig. 16.)

Sling. — A loop of rope, very useful in erection for making a hasty attachment.

Slope. — Inclination to a horizontal plane.

Snatch Block. — A block with one side capable of being opened for the insertion of the rope. Its office is to change the direction of the rope.

Span. — The length of a bridge from centre to centre of end pins or bearings.

Spikes. — Large nails for timber work. (Plate II., Fig. 13.)

Splay. — To spread at one end the two main portions of a member.

Splice. — A joint connected by means of plates.

Splice Plate. — A connecting plate at a joint. (Plate II., Fig. 12.)

Spread. — The distance apart laterally.

Staggered Rivets. — Rivets are said to be staggered when each rivet of one row is opposite to the middle of the space between two rivets of the next row.

Static Load. — Dead load, *q. v.*

Stay Plate. — A plate always used at the end of a system of lacing or latticing. (Plate II., Fig. 12.)

Stiffening-Angle. — An angle iron used to stiffen the web of a beam. (Plate II., Fig. 13.)

Stiffener. — A piece of iron used to stiffen the web of a beam: it may be of angle or tee section. (Plate II., Fig. 13.)

Strain. — The extension or compression of a piece of material which is or has been under stress.

Stress. — The internal resisting force of a piece of material which is strained.

Strut. — A member which resists compression.

Sub-Punching. — The punching of rivet holes which have to be afterwards enlarged by reaming.

Sway Bracing. — Bracing transverse to the planes of the trusses. Its objects are to resist wind pressure, and to prevent undue vibration from passing loads. (Plate I.)

Table of Data. — A list of the known circumstances that affect the designing of a structure.

Tap. — A screw for cutting a thread in a nut.

Tee or T iron. — A piece of rolled iron of the section shown on Plate II., Fig. 4.

Tension. — A stress tending to elongate a body.

Thread. — The spiral part of a screw or nut.

Through Bridge. — A bridge with overhead bracing.

Tie. — A tension member; generally refers to a main truss.

Timber Truck. — A small, strong wooden frame, with an iron roller set entirely below the upper surface. It is used in bridge erection for moving large timbers and heavy weights along a runway.

Tongs. — Part of a riveting outfit; used for holding and carrying heated rivets.

Transverse Component. — A component in a transverse direction; generally intended for a component perpendicular to the planes of the trusses.

Truss. — An assemblage of tension and compression members so arranged as to transmit loads from intermediate points to the ends.

Trussing. — A poor substitute for lacing or latticing. (Plate II., Fig. 8, Plate VI.)

Turn Buckle. — Similar to a sleeve nut, and for the same purpose. The sides are open, so that a crowbar may be inserted for the purpose of screwing up. Turn buckles are used for larger bars or rods than are sleeve nuts. (Plate II., Fig. 16.)

Ultimate Strength. — The greatest load that a portion of material can bear.

Uniform Load. — A load so distributed over an entire structure, that equal lengths everywhere receive equal portions.

U-nut. — A piece of iron, in the shape of the letter U, through which passes the threaded end of a rod, and which affords a bearing for the nut, with room to screw up the latter. Its use is not permissible in first-class bridge construction.

Upper Falsework. — The falsework that lies above the level of the lower chords.

Upset End. — An end of a rod or bar enlarged for the cutting thereon of a screw-thread.

Vibration Rod. — A tension member for vertical or portal sway bracing. (Plate I.)

Washer. — A piece of cast or wrought iron to distribute the pressure of a bolt-head or nut over timber. (Plate II., Fig. 6.)

Web. — The portion of a truss or beam between the flanges. Its office is principally to resist shear.

Welded Heads. — Heads first worked into shape, then welded on the bars.

Whipple Truss. — See Linville truss.

Wind Shakes. — Cracks in timber caused by the wind while the tree was living.

Working-Drawings. — Drawings containing all the measurements necessary for construction.

Working-Stress. — The stress, usually the greatest stress, to which a piece of material is or should be subjected. Sometimes incorrectly employed for intensity of working-stress.

Wrench. — A tool for screwing up nuts.

INDEX.

INDEX.

INDEX TO ADDENDA.

A.

TABLE I

OT OF IR

CLASS A.

Span	18′ Roadway.				24′ Roadway.					Span.
	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	
40	20	58	266	559	168	21	87	339	600	40
50	20	68	268	616	209	22	105	341	664	50
60	30	86	257	640	215	30	136	327	696	60
70	57	69	266	662	210	67	105	339	710	70
80	53	65	276	681	222	62	113	352	739	80
90	49	64	291	722	253	55	110	376	784	90
100	49	61	336	807	293	53	107	432	876	100
110	55	63	314	834	336	60	113	410	911	110
120	51	62	336	890	374	58	110	432	966	120
130	57	64	314	919	426	66	115	410	1009	130
140	54	63	336	936	426	64	114	432	1028	140
150	61	66	314	966	476	71	117	410	1067	150
160	77	64	336	1035	502	96	116	432	1139	160
170	78	67	314	1060	546	99	120	410	1168	170
180	79	64	336	1108	576	98	119	432	1218	180
190	81	69	314	1132	621	101	122	410	1247	190
200	81	67	336	1189	662	99	121	432	1307	200
210	83	71	314	1173	662	102	124	410	1291	210
220	83	68	336	1232	696	101	123	432	1345	220
230	85	72	314	1263	752	104	125	410	1384	230
240	84	69	336	1324	802	102	124	432	1453	240
250	86	73	336	1380	859	105	129	432	1518	250
260	86	69	336	1417	904	103	126	432	1558	260
270	89	72	336	1478	959	106	130	432	1621	270
280	87	71	336	1520	1002	105	129	432	1662	280
290	90	72	336	1580	1062	107	132	432	1727	290
300	89	71	336	1623	1111	106	131	432	1774	300

TABLE I.

TABLE OF WEIGHTS PER LINEAL FOOT OF IRON

CLASS A.

Span.	12′ Roadway.					14′ Roadway.				
	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.
40	143	20	26	193	368	145	20	39	217	407
50	168	20	30	194	400	175	20	41	218	442
60	154	30	44	187	404	164	30	57	210	450
70	161	49	30	193	423	170	52	44	217	473
80	164	45	28	200	428	182	48	40	225	486
90	187	43	29	219	469	205	45	40	241	522
100	207	43	30	241	513	221	45	39	269	566
110	220	42	33	230	518	240	50	41	258	582
120	241	44	32	241	551	268	46	39	269	615
130	268	52	35	230	578	298	53	42	258	644
140	285	49	34	241	602	320	49	41	269	672
150						354	53	44	258	703
160						370	66	43	269	742
170						374	74	45	258	745
180						395	72	43	269	773
190						424	75	45	258	796
200										
210										
220										
230										
240										
250										
260										
270										
280										
290										
300										

Span.	16′ Roadway.					18′ Roadway.			
	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.
40	147	20	48	242	443	153	20	58	266
50	182	20	54	244	488	192	20	68	268
60	174	30	74	233	500	184	30	86	257
70	186	54	54	242	526	199	57	69	266
80	200	50	50	250	541	214	53	65	276
90	221	47	50	266	575	232	49	64	291
100	242	47	47	308	636	249	49	61	336
110	266	53	50	286	648	279	55	63	314
120	295	48	48	308	692	317	51	62	336
130	330	54	51	286	714	346	57	64	314
140	354	49	50	308	754	369	54	63	336
150	390	55	54	286	779	374	61	66	314
160	389	69	51	308	811	399	77	64	336
170	413	74	55	286	822	422	78	67	314
180	426	76	53	308	857	440	79	64	336
190	461	80	55	286	876	470	81	69	314
200	491	76	53	308	922	509	81	67	336
210	489	79	55	286	903	507	83	71	314
220	514	76	54	308	946	549	83	68	336
230	548	79	55	286	962	587	85	72	314
240	578	78	54	308	1012	621	84	69	336
250	615	81	55	308	1053	661	86	73	336
260	641	80	54	308	1077	693	86	69	336
270						739	89	72	336
280						780	87	71	336
290						826	92	72	336
300						872	89	71	336

TABLE I.

L FOOT OF IRON PRATT TRUSS HIGHWAY-BRIDGES.

CLASS A.

18′ Roadway.					20′ Roadway.					22′ Roadway.					24′ Roadway.					Span.
russes.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	
153	20	58	266	482	161	20	72	290	528	159	21	79	315	559	168	21	87	339	600	40
192	20	68	268	535	201	21	81	292	582	200	22	90	317	616	209	22	105	341	664	50
184	30	86	257	545	198	30	100	280	596	202	30	116	304	640	215	30	136	327	696	60
199	57	69	266	580	207	60	82	290	628	203	63	92	315	662	210	67	105	339	710	70
214	53	65	276	598	221	56	78	301	646	216	59	89	327	681	222	62	113	352	739	80
232	49	64	291	626	243	51	74	325	683	242	53	86	351	722	253	55	110	376	784	90
249	49	61	336	686	267	50	71	364	743	274	52	86	404	807	293	53	107	432	876	100
279	55	63	314	703	300	56	74	342	704	313	58	89	382	834	336	60	113	410	911	110
317	51	62	336	758	335	53	72	364	806	350	56	88	404	890	374	58	110	432	966	120
346	57	64	314	773	372	60	75	342	841	391	63	91	382	919	426	66	115	410	1009	130
369	54	63	336	814	397	59	72	364	884	388	62	90	404	936	426	64	114	432	1028	140
374	61	66	314	808	400	66	75	342	876	429	69	93	382	966	476	71	117	410	1067	150
399	77	64	336	869	418	85	73	364	933	457	90	91	404	1035	502	96	116	432	1139	160
422	78	67	314	874	457	88	77	342	957	497	93	95	382	1060	546	99	120	410	1168	170
440	79	64	336	912	482	87	74	364	1000	524	92	95	404	1108	576	98	119	432	1218	180
470	81	69	314	927	519	90	79	342	1023	564	95	98	382	1132	621	101	122	410	1247	190
509	81	67	336	986	554	88	77	364	1076	603	93	96	404	1189	662	99	121	432	1307	200
507	83	71	314	968	555	91	80	342	1061	605	96	97	382	1173	662	102	124	410	1291	210
549	83	68	336	1029	589	90	77	364	1113	644	95	96	404	1232	696	101	123	432	1345	220
587	85	72	314	1051	633	93	80	342	1141	689	98	101	382	1263	752	104	125	410	1384	230
621	84	69	336	1103	673	91	78	364	1199	733	96	98	404	1324	802	102	124	432	1453	240
661	86	73	336	1149	718	94	81	364	1250	782	99	102	404	1380	859	105	129	432	1518	250
693	86	69	336	1177	752	93	80	364	1282	821	98	101	404	1417	904	103	126	432	1558	260
739	89	72	336	1230	798	96	83	364	1335	874	101	105	404	1478	959	106	130	432	1621	270
780	87	71	336	1268	841	95	81	364	1375	919	100	103	404	1520	1002	105	129	432	1662	280
826	92	72	336	1318	892	98	84	364	1432	973	103	106	404	1580	1062	107	132	432	1727	290
872	89	71	336	1362	936	96	82	364	1472	1020	101	104	404	1623	1111	106	131	432	1774	300

E II.

F IRON

S B.

dway.

Sp

R M.	LUMBER.	D. I.
	266	47
	268	51
	257	51
	266	55
	276	56
	291	59
1	336	65
1	314	66
1	336	71
1	314	72
1	336	76
1	314	78
1	336	81
1	314	82
1	336	86
1	314	87
2	336	91
2	314	90
2	336	94
2	314	96
2	336	100
2	336	104
2	336	107
2	336	111
2	336	114
2	336	118
3	339	122

E II.

F IRON

S B.

...lway.			24′ Roadway.					Span.
...r ...m.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	
	266	473.3	143	21	78	339	567	40
	268	518.9	184	22	94	341	629	50
	257	516.4	184	30	122	327	652	60
	266	552.3	179	67	94	339	669	70
	276	568.4	194	62	101	352	700	80
	291	598.7	226	55	98	376	746	90
1	336	657.0	200	53	96	432	833	100
1	314	669.1	207	60	101	410	861	110
1	336	714.1	327	58	99	432	909	120
1	314	729.5	368	66	103	410	940	130
1	336	760.5	397	64	102	432	988	140
1	314	780.0	409	71	105	410	989	150
1	336	816.2	431	96	104	432	1057	160
1	314	828.2	468	99	107	410	1078	170
1	336	862.5	492	98	106	432	1122	180
1	314	874.3	530	101	109	410	1144	190
2	336	916.3	563	99	108	432	1166	200
2	314	907.7	564	102	111	410	1181	210
2	336	942.8	596	101	110	432	1233	220
2	314	985.1	641	104	112	410	1261	230
2	336	1008.5	680	102	111	432	1310	240
2	336	1045.2	725	105	115	432	[illegible]	250
2	336	1078.2	761	103	113	432	1403	260
2	336	1113.2	807	106	116	432	1450	270
2	336	1146.4	842	105	115	432	1489	280
2	336	1186.5	891	107	118	432	1513	290
3	336	1224.8	929	106	117	432	1580	300

TABLE II.

TABLE OF WEIGHTS PER LINEAL FOOT OF IRON

CLASS B.

Span.	12′ Roadway. Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	14′ Roadway. Trusses.	Lateral System.	Floor System.	Lumber.	D. L.
40	143	20	25	193	368	143	20	36	217	403
50	161	20	29	194	393	161	20	38	218	426
60	144	30	42	187	393	148	30	52	210	430
70	150	49	27	193	410	156	52	40	217	456
80	151	45	25	200	413	163	48	37	225	465
90	169	43	26	219	449	183	45	37	241	498
100	185	43	27	241	489	198	45	36	260	541
110	197	48	30	230	499	215	50	38	258	555
120	216	44	29	241	524	239	46	36	260	584
130	239	52	32	230	547	266	53	39	258	610
140	252	49	31	241	567	284	49	38	260	634
150						311	53	40	258	657
160						327	66	39	260	696
170						349	74	41	258	717
180						349	72	39	260	724
190						374	75	41	258	743
200										
210										
220										
230										
240										
250										
260										
270										
280										
290										
300										

Span.	16′ Roadway. Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	18′ Roadway. Trusses.	Lateral System.	Floor System.	Lumber.	D. L.
40	143	20	44	242	436	147	20	53	266	47[illegible]
50	170	20	49	244	472	179	20	62	268	517
60	156	30	68	233	477	164	30	78	257	51[illegible]
70	165	54	49	242	501	175	57	63	266	55[illegible]
80	178	50	46	250	516	190	53	59	276	56[illegible]
90	198	47	46	266	549	208	49	57	291	59[illegible]
100	217	47	43	308	608	222	49	53	336	65[illegible]
110	237	53	46	286	616	249	55	56	314	667
120	263	48	44	308	657	282	51	54	336	71[illegible]
130	293	54	47	286	674	308	57	57	314	72[illegible]
140	314	49	46	308	711	330	54	55	336	76[illegible]
150	345	55	49	286	730	358	61	58	314	78[illegible]
160	365	69	47	308	784	354	77	56	336	81[illegible]
170	365	74	50	286	770	377	78	59	314	82[illegible]
180	377	76	48	308	804	397	79	57	336	86[illegible]
190	406	80	50	286	817	422	81	60	314	87[illegible]
200	432	76	48	308	859	449	81	59	336	91[illegible]
210	429	79	50	286	839	447	83	62	314	90[illegible]
220	451	76	49	308	879	472	83	60	336	94[illegible]
230	481	79	50	286	891	504	85	63	314	96[illegible]
240	507	78	49	308	937	533	84	61	336	100[illegible]
250	538	81	50	308	972	568	86	64	336	104[illegible]
260	560	80	49	308	992	594	86	61	336	107[illegible]
270						631	89	64	336	111[illegible]
280						663	87	62	336	114[illegible]
290						701	90	64	336	118[illegible]
300						738	89	62	336	122[illegible]

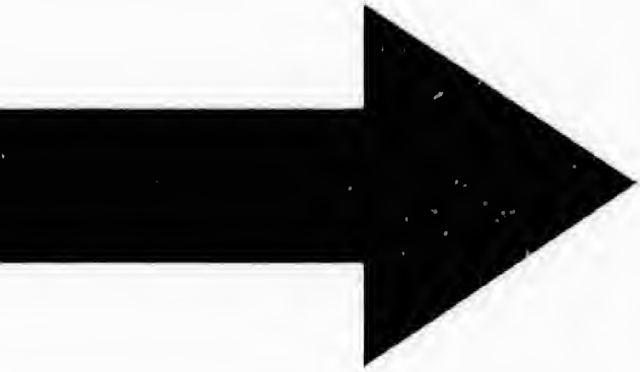
TABLE II.

…FOOT OF IRON PRATT TRUSS HIGHWAY-BRIDGES.

CLASS B.

18′ Roadway.				20′ Roadway.					22′ Roadway.					24′ Roadway.					Span.
Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	
20	53	266	472	149	20	65	290		141	21	70	315	533	143	21	78	339	567	40
20	62	268	517	187	21	73	292		181	22	81	317	589	184	22	94	341	629	50
30	78	257	518	179	30	90	280	568	177	30	104	304	604	184	30	122	327	652	60
57	63	266	551	178	60	74	290	592	172	63	83	315	623	179	67	94	339	669	70
53	59	276	569	193	56	70	301	611	187	59	80	327	644	194	62	101	352	700	80
49	57	291	596	217	51	67	325	651	215	53	77	357	687	226	55	98	376	746	90
49	53	336	652	238	50	63	364	707	245	52	77	404	770	260	53	96	432	833	100
55	56	314	667	267	56	66	342	724	278	58	80	382	791	297	60	101	410	861	110
51	54	336	716	290	53	64	364	773	309	56	79	404	841	327	58	99	432	909	120
57	57	314	729	331	60	67	342	793	345	63	82	382	865	368	66	103	410	940	130
54	55	336	768	351	59	64	364	831	365	62	81	404	905	397	64	102	432	988	140
61	58	314	785	378	66	67	342	847	371	60	84	382	900	409	71	105	410	989	150
77	56	336	817	369	85	65	364	877	392	90	82	404	962	431	96	104	432	1057	160
78	59	314	822	399	88	68	342	891	428	93	85	382	982	468	99	107	410	1078	170
79	57	336	863	419	87	66	364	930	450	92	85	404	1025	492	98	106	432	1122	180
81	60	314	871	450	90	70	342	946	484	95	88	382	1043	530	101	109	410	1144	190
81	59	336	919	479	88	68	364	993	516	93	86	404	1093	563	99	108	432	1196	200
83	62	314	900	478	91	71	342	976	516	96	89	382	1077	564	102	111	410	1181	210
83	60	336	945	505	90	68	364	1021	547	95	88	404	1128	596	101	110	432	1233	220
85	63	314	960	541	93	71	342	1041	586	98	91	382	1151	641	104	112	410	1261	230
84	61	336	1008	574	91	69	364	1092	622	96	89	404	1205	680	102	111	432	1319	240
86	64	336	1048	612	94	72	364	1136	663	99	92	404	1252	725	105	115	432	1371	250
86	61	336	1071	639	93	71	364	1161	695	98	91	404	1282	761	103	113	432	1403	260
89	64	336	1115	677	96	74	364	1206	738	101	94	404	1332	807	106	116	432	1456	270
87	62	336	1143	712	95	72	364	1238	773	100	92	404	1364	842	105	115	432	1489	280
90	64	336	1186	754	98	75	364	1286	818	103	95	404	1415	891	107	118	432	1543	290
89	62	336	1221	790	96	73	364	1319	854	101	93	404	1448	929	106	117	432	1580	300

[redacted]



ABI

OT (

CLA

18′ R

Spar

	TERAL STEM.	Fı Sy
40	20	
50	20	
60	30	
70	57	
80	53	
90	19	
100	49	
110	55	
120	51	
130	57	
140	54	
150	51	
160	77	
170	78	
180	79	
190	31	
200	31	
210	33	
220	33	
230	35	
240	34	
250	36	
260	36	
270	39	
280	37	
290	30	
300	39	

ABI

OT (

CLA

Span	18′ Ro… Lateral System	Fl. Sy… / Lumber	D. L.	24′ Roadway. Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	Span.
40	20	315	536	139	21	71	339	557	40
50	20	301	565	172	22	85	325	594	50
60	30	304	593	172	30	110	327	629	60
70	57	315	623	172	67	86	339	656	70
80	53	327	644	183	62	92	351	681	80
90	49	333	665	210	55	89	358	705	90
100	49	357	711	238	53	87	385	757	100
110	55	341	736	272	60	92	366	785	110
120	51	357	774	300	58	90	385	828	120
130	57	341	801	337	66	94	366	858	130
140	54	357	832	363	64	93	385	899	140
150	…1	341	862	374	71	96	366	901	150
160	77	357	884	392	96	94	385	961	160
170	…8	341	904	425	99	97	366	982	170
180	…9	357	933	442	98	96	385	1016	180
190	…1	341	958	481	101	99	366	1042	190
200	…1	357	998	511	99	98	385	1088	200
210	…3	341	984	508	102	101	366	1072	210
220	…3	357	1023	536	101	100	385	1117	220
230	…5	341	1050	577	104	102	366	1144	230
240	…4	357	1094	612	102	101	385	1195	240
250	…6	357	1138	653	105	104	385	1242	250
260	…6	357	1162	685	103	102	385	1271	260
270	…9	357	1207	727	106	105	385	1319	270
280	…7	357	1235	758	105	104	385	1346	280
290	…0	357	1283	804	107	107	385	1399	290
300	…9	357	1351	838	106	106	385	1431	300

TABLE III.

TABLE OF WEIGHTS PER LINEAL FOOT OF IRON

CLASS C.

Span.	12′ Roadway.					14′ Roadway.					16′ Roadway.					18′ Roadway.			
	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.
40	143	20	25	193	368	143	20	33	217	410	143	20	40	241	431	143	20	49	266
50	161	20	29	194	494	161	20	35	208	414	161	20	45	232	448	167	20	57	255
60	140	30	42	187	389	144	30	47	210	421	148	30	62	234	464	158	30	72	257
70	148	49	27	193	409	153	52	37	217	451	158	54	45	241	490	167	57	58	266
80	149	45	24	198	409	158	48	34	222	455	166	50	42	247	498	179	53	54	271
90	159	43	24	198	418	170	45	34	223	465	181	47	42	249	512	194	49	52	274
100	169	43	23	217	446	181	45	32	245	497	193	47	39	272	545	206	49	49	301
110	180	48	26	206	455	197	50	34	232	508	213	53	42	257	560	231	55	52	290
120	198	44	25	217	479	219	46	32	245	537	239	48	40	272	594	256	51	50	301
130	219	52	28	206	500	244	53	35	232	559	268	54	43	257	617	284	57	53	290
140	232	49	27	217	520	260	49	34	245	583	289	49	42	272	647	301	54	51	301
150						285	53	36	232	601	316	55	45	257	668	327	61	54	290
160						298	66	35	245	639	331	69	43	272	710	344	77	52	301
170						318	74	37	232	656	331	74	46	257	703	343	78	55	290
180						318	72	35	245	665	342	76	44	272	729	356	79	53	301
190						340	75	37	232	679	370	80	46	257	748	386	81	56	290
200											390	76	44	272	767	409	81	54	301
210											386	79	46	257	763	405	83	57	290
220											405	76	45	272	793	426	83	55	301
230											433	79	46	257	810	457	85	58	290
240											456	78	45	272	846	483	84	56	301
250											484	81	46	272	878	515	86	59	301
260											505	80	45	272	897	538	86	56	301
270																574	89	59	301
280																603	87	57	301
290																638	90	59	301
300																668	89	57	301

TABLE III.

FOOT OF IRON PRATT TRUSS HIGHWAY-BRIDGES.

CLASS C.

18′ Roadway.					20′ Roadway.					22′ Roadway.					24′ Roadway.					Span.
usses.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	Trusses.	Lateral System.	Floor System.	Lumber.	D L.	Trusses.	Lateral System.	Floor System.	Lumber.	D. L.	
143	20	49	266	465	146	20	60	290	503	148	21	64	315	536	139	21	71	339	557	40
167	20	57	255	489	174	21	67	278	530	178	22	74	301	565	172	22	85	325	594	50
158	30	72	257	507	168	30	83	280	551	175	30	94	304	593	172	30	110	327	629	60
167	57	58	266	540	172	60	68	290	582	177	63	76	315	623	172	67	86	339	656	70
179	53	54	271	550	183	56	64	295	591	190	59	75	327	644	183	62	92	351	681	80
194	49	52	274	562	204	51	62	308	618	213	53	73	333	665	210	55	89	358	705	90
206	49	49	301	599	225	50	58	329	656	235	52	70	357	711	238	53	87	385	757	100
231	55	52	290	623	254	56	61	316	682	269	58	73	341	736	272	60	92	366	785	110
256	51	50	301	653	277	53	59	329	713	294	56	72	357	774	300	58	90	385	828	120
284	57	53	290	679	305	60	62	316	738	327	63	75	341	801	337	66	94	366	858	130
301	54	51	301	702	322	59	59	329	764	344	62	74	357	832	363	64	93	385	899	140
327	61	54	290	727	348	66	62	316	787	380	69	77	341	862	374	71	96	366	901	150
344	77	52	301	769	337	85	60	329	806	367	90	75	357	884	392	96	94	385	961	160
343	78	55	290	761	365	88	63	316	827	397	93	78	341	904	425	99	97	366	982	170
356	79	53	301	784	379	87	61	329	851	412	92	77	357	933	442	98	96	385	1016	180
386	81	56	290	808	411	90	64	316	876	447	95	80	341	958	481	101	99	366	1042	190
409	81	54	301	840	436	88	62	329	910	475	93	78	357	998	511	99	98	385	1088	200
405	83	57	290	830	432	91	65	316	899	471	96	81	341	984	508	102	101	366	1072	210
426	83	55	301	860	455	90	62	329	931	496	95	80	357	1023	536	101	100	385	1117	220
457	85	58	290	885	489	93	65	316	958	533	98	83	341	1050	577	104	102	366	1144	230
483	84	56	301	919	518	91	63	329	996	565	96	81	357	1094	612	102	101	385	1195	240
515	86	59	301	956	553	94	66	329	1037	603	99	84	357	1138	653	105	104	385	1242	250
538	86	56	301	976	577	93	64	329	1059	630	98	82	357	1162	685	103	102	385	1271	260
574	89	59	301	1019	612	96	67	329	1100	669	101	85	357	1207	727	106	105	385	1319	270
603	87	57	301	1044	644	95	65	329	1129	700	100	83	357	1235	758	105	104	385	1346	280
638	90	59	301	1084	682	98	68	329	1173	742	103	86	357	1283	804	107	107	385	1399	290
668	89	57	301	1111	713	96	66	329	1200	774	101	84	357	1351	838	106	106	385	1431	300

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TABLE IV.

ECONOMIC DEPTHS AND PANEL LENGTHS.

Span.	No. of Panels.	Depth.	
		SINGLE INTERSECTION.	DOUBLE INTERSECTION.
80′	5	16.5′	
90′	5	18′	
100′	5	20′	
110′	6	21′	
120′	6	21′	
130′	7	22′	
140′	7	23′	26′
150′	8	23′	26′
160′	8	24′	27′
170′	9	26′	29′
180′	9	27′	30′
190′	10		32′
200′	10		33′
210′	11		34′
220′	11		35′
230′	12		36′
240′	12		38′
250′	13		39′
260′	13		40′
270′	14		41′
280′	14		42′
290′	15		43′
300′	15		44′

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260'
270'
280'
290'
300'

TABLE V.

ECONOMIC DEPTHS AND PANEL LENGTHS.

Span.	No. of Panels.	Depth.	
		SINGLE INTERSECTION.	DOUBLE INTERSECTION.
80′	5	16.5′	
90′	5	18′	
100′	5	20′	
110′	5	21′	
120′	5	22′	
130′	6	22′	
140′	6	23′	26′
150′	7	24′	27′
160′	7	25′	28′
170′	8	27′	30′
180′	8	28′	32′
190′	8		34′
200′	9		35′
210′	9		36′
220′	10		37′
230′	10		38′
240′	10		40′
250′	11		41′
260′	11		42′
270′	12		43′
280′	12		44′
290′	13		45′
300′	13		46′

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Panel Length.	12 Road
10′	2 $\frac{3}{4}''$ 0.91
11′	2 $\frac{3}{4}''$ 1.00
12′	2 $\frac{3}{4}''$ 1.09
13′	2 $\frac{13}{16}$ 1.18
14′	2 $\frac{13}{16}$ 1.27
15′	2 $\frac{7}{8}''$ 1.36
16′	2 $\frac{7}{8}''$ 1.45
17′	2 $\frac{7}{8}''$ 1.54
18′	2 $\frac{15}{16}$ 1.64
19′	2 $\frac{15}{16}$ 1.73
20′	2 1″ 1.84
21′	2 1″ 1.95
22′	2 $1\frac{1}{16}$ 2.07
23′	2 $1\frac{1}{16}$ 2.18
24′	2 $1\frac{1}{8}$ 2.29

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TABLE VI.

TABLE OF SIZES OF HIP VERTICALS FOR BRIDGES OF CLASS A,

In which the live load is one hundred pounds per square foot of floor, and the working-stress on the verticals is four tons to the square inch.

The upper figures give the sizes of the hip verticals; the lower ones, the sections required.

Panel Length.	12′ Roadway.	14′ Roadway.	16′ Roadway.	18′ Roadway.	20′ Roadway.	22′ Roadway.	24′ Roadway.	Panel Length.
10′	2 3/4″ □ 0.91 □″	2 3/4″ □ 1.06 □″	2 13/16″ □ 1.21 □″	2 7/8″ □ 1.38 □″	2 7/8″ □ 1.51 □″	2 15/16″ □ 1.67 □″	2 1″ □ 1.84 □″	10′
11′	2 3/4″ □ 1.00 □″	2 13/16″ □ 1.16 □″	2 13/16″ □ 1.33 □″	2 7/8″ □ 1.52 □″	2 15/16″ □ 1.66 □″	2 1″ □ 1.84 □″	2 1″ □ 2.03 □″	11′
12′	2 3/4″ □ 1.09 □″	2 13/16″ □ 1.26 □″	2 7/8″ □ 1.45 □″	2 15/16″ □ 1.64 □″	2 1″ □ 1.81 □″	2 1″ □ 2.00 □″	2 1 1/16″ □ 2.19 □″	12′
13′	2 13/16″ □ 1.18 □″	2 7/8″ □ 1.37 □″	2 15/16″ □ 1.56 □″	2 15/16″ □ 1.77 □″	2 1″ □ 1.95 □″	2 1 1/16″ □ 2.16 □″	2 1 1/8″ □ 2.36 □″	13′
14′	2 13/16″ □ 1.27 □″	2 7/8″ □ 1.48 □″	2 15/16″ □ 1.68 □″	2 1″ □ 1.90 □″	2 1 1/16″ □ 2.10 □″	2 1 1/8″ □ 2.32 □″	2 1 1/8″ □ 2.53 □″	14′
15′	2 7/8″ □ 1.36 □″	2 15/16″ □ 1.59 □″	2 1″ □ 1.81 □″	2 1 1/16″ □ 2.04 □″	2 1 1/16″ □ 2.26 □″	2 1 1/8″ □ 2.49 □″	2 1 3/16″ □ 2.72 □″	15′
16′	2 7/8″ □ 1.45 □″	2 15/16″ □ 1.69 □″	2 1″ □ 1.93 □″	2 1 1/16″ □ 2.18 □″	2 1 1/8″ □ 2.41 □″	2 1 3/16″ □ 2.66 □″	2 1 1/4″ □ 2.91 □″	16′
17′	2 7/8″ □ 1.54 □″	2 1″ □ 1.80 □″	2 1 1/16″ □ 2.05 □″	2 1 1/8″ □ 2.32 □″	2 1 1/8″ □ 2.50 □″	2 1 3/16″ □ 2.83 □″	2 1 1/4″ □ 3.09 □″	17′
18′	2 15/16″ □ 1.64 □″	2 1″ □ 1.91 □″	2 1 1/16″ □ 2.17 □″	2 1 1/8″ □ 2.46 □″	2 1 3/16″ □ 2.71 □″	2 1 1/4″ □ 2.99 □″	2 1 5/16″ □ 3.27 □″	18′
19′	2 15/16″ □ 1.73 □″	2 1″ □ 2.02 □″	2 1 1/8″ □ 2.29 □″	2 1 3/16″ □ 2.61 □″	2 1 1/4″ □ 2.86 □″	2 1 5/16″ □ 3.16 □″	2 1 5/16″ □ 3.40 □″	19′
20′	2 1″ □ 1.84 □″	2 1 1/16″ □ 2.14 □″	2 1 1/8″ □ 2.43 □″	2 1 3/16″ □ 2.70 □″	2 1 1/4″ □ 3.03 □″	2 1 5/16″ □ 3.31 □″	2 1 3/8″ □ 3.66 □″	20′
21′	2 1″ □ 1.95 □″	2 1 1/8″ □ 2.27 □″	2 1 3/16″ □ 2.58 □″	2 1 1/4″ □ 2.92 □″	2 1 5/16″ □ 3.20 □″	2 1 3/8″ □ 3.54 □″	2 1 7/16″ □ 3.87 □″	21′
22′	2 1 1/16″ □ 2.07 □″	2 1 1/8″ □ 2.40 □″	2 1 3/16″ □ 2.73 □″	2 1 1/4″ □ 3.00 □″	2 1 5/16″ □ 3.30 □″	2 1 3/8″ □ 3.74 □″	2 1 7/16″ □ 4.09 □″	22′
23′	2 1 1/16″ □ 2.18 □″	2 1 1/8″ □ 2.53 □″	2 1 1/4″ □ 2.87 □″	2 1 5/16″ □ 3.27 □″	2 1 3/8″ □ 3.57 □″	2 1 7/16″ □ 3.95 □″	2 1 1/2″ □ 4.33 □″	23′
24′	2 1 1/8″ □ 2.29 □″	2 1 3/16″ □ 2.65 □″	2 1 1/4″ □ 3.01 □″	2 1 5/16″ □ 3.45 □″	2 1 3/8″ □ 3.74 □″	2 1 7/16″ □ 4.16 □″	2 1 1/2″ □ 4.56 □″	24′

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TABLE VII.

TABLE OF SIZES OF HIP VERTICALS FOR BRIDGES OF CLASS B,

In which the live load is one hundred pounds per square foot of floor, and the working-stress on the verticals is five tons to the square inch.

The upper figures give the sizes of the hip verticals; the lower ones, the sections required.

Panel Length.	12′ Roadway.	14′ Roadway.	16′ Roadway.	18′ Roadway.	20′ Roadway.	22′ Roadway.	24′ Roadway.	Panel Length.
10′	2 3/4″ □ 0.72 □″	2 3/4″ □ 0.84 □″	2 3/4″ □ 0.96 □″	2 3/4″ □ 1.09 □″	2 13/16″ □ 1.22 □″	2 7/8″ □ 1.34 □″	2 7/8″ □ 1.45 □″	10′
11′	2 3/4″ □ 0.79 □″	2 3/4″ □ 0.92 □″	2 3/4″ □ 1.06 □″	2 13/16″ □ 1.20 □″	2 7/8″ □ 1.33 □″	2 7/8″ □ 1.47 □″	2 15/16″ □ 1.60 □″	11′
12′	2 3/4″ □ 0.86 □″	2 3/4″ □ 1.00 □″	2 13/16″ □ 1.15 □″	2 13/16″ □ 1.31 □″	2 7/8″ □ 1.44 □″	2 15/16″ □ 1.59 □″	2 15/16″ □ 1.74 □″	12′
13′	2 3/4″ □ 0.94 □″	2 3/4″ □ 1.08 □″	2 13/16″ □ 1.24 □″	2 7/8″ □ 1.41 □″	2 15/16″ □ 1.55 □″	2 15/16″ □ 1.72 □″	2 1″ □ 1.88 □″	13′
14′	2 3/4″ □ 1.01 □″	2 13/16″ □ 1.17 □″	2 7/8″ □ 1.34 □″	2 7/8″ □ 1.52 □″	2 15/16″ □ 1.67 □″	2 1″ □ 1.85 □″	2 1″ □ 2.02 □″	14′
15′	2 3/4″ □ 1.08 □″	2 13/16″ □ 1.26 □″	2 7/8″ □ 1.44 □″	2 15/16″ □ 1.62 □″	2 1″ □ 1.80 □″	2 1″ □ 1.98 □″	2 1 1/16″ □ 2.16 □″	15′
16′	2 13/16″ □ 1.15 □″	2 7/8″ □ 1.35 □″	2 7/8″ □ 1.54 □″	2 15/16″ □ 1.73 □″	2 1″ □ 1.93 □″	2 1 1/16″ □ 2.12 □″	2 1 1/8″ □ 2.31 □″	16′
17′	2 13/16″ □ 1.23 □″	2 7/8″ □ 1.43 □″	2 15/16″ □ 1.64 □″	2 1″ □ 1.85 □″	2 1 1/16″ □ 2.05 □″	2 1 1/16″ □ 2.26 □″	2 1 1/8″ □ 2.46 □″	17′
18′	2 13/16″ □ 1.30 □″	2 7/8″ □ 1.52 □″	2 15/16″ □ 1.73 □″	2 1″ □ 1.96 □″	2 1 1/16″ □ 2.17 □″	2 1 1/8″ □ 2.39 □″	2 1 3/16″ □ 2.61 □″	18′
19′	2 7/8″ □ 1.38 □″	2 15/16″ □ 1.61 □″	2 1″ □ 1.83 □″	2 1 1/16″ □ 2.07 □″	2 1 1/8″ □ 2.29 □″	2 1 1/8″ □ 2.53 □″	2 1 3/16″ □ 2.76 □″	19′
20′	2 7/8″ □ 1.46 □″	2 15/16″ □ 1.71 □″	2 1″ □ 1.94 □″	2 1 1/16″ □ 2.19 □″	2 1 1/8″ □ 2.42 □″	2 1 3/16″ □ 2.67 □″	2 1 1/4″ □ 2.92 □″	20′
21′	2 15/16″ □ 1.55 □″	2 1″ □ 1.81 □″	2 1 1/16″ □ 2.06 □″	2 1 1/8″ □ 2.32 □″	2 1 3/16″ □ 2.57 □″	2 1 1/4″ □ 2.83 □″	2 1 1/4″ □ 3.00 □″	21′
22′	2 15/16″ □ 1.65 □″	2 1″ □ 1.91 □″	2 1 1/16″ □ 2.18 □″	2 1 1/8″ □ 2.46 □″	2 1 3/16″ □ 2.72 □″	2 1 1/4″ □ 3.00 □″	2 1 5/16″ □ 3.28 □″	22′
23′	2 15/16″ □ 1.74 □″	2 1″ □ 2.01 □″	2 1 1/8″ □ 2.29 □″	2 1 3/16″ □ 2.60 □″	2 1 1/4″ □ 2.86 □″	2 1 5/16″ □ 3.17 □″	2 1 5/16″ □ 3.47 □″	23′
24′	2 1″ □ 1.83 □″	2 1 1/16″ □ 2.11 □″	2 1 1/8″ □ 2.41 □″	2 1 3/16″ □ 2.74 □″	2 1 1/4″ □ 3.[illegible] □″	2 1 5/16″ □ 3.33 □″	2 1 3/8″ □ 3.65 □″	24′

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TABLE C

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stress on the
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tions required.

Panel Length.	12′ Roadw
10′	2 $\frac{3}{4}''$ 0.60 □
11′	2 $\frac{3}{4}''$ 0.65 □
12′	2 $\frac{3}{4}''$ 0.71 □
13′	2 $\frac{3}{4}''$ 0.77 □
14′	2 $\frac{3}{4}''$ 0.83 □
15′	2 $\frac{3}{4}''$ 0.89 □
16′	2 $\frac{3}{4}''$ 0.95 □
17′	2 $\frac{3}{4}''$ 1.01 □
18′	2 $\frac{3}{4}''$ 1.07 □
19′	2 $\frac{3}{4}''$ 1.13 □
20′	2 $\frac{13}{16}''$ 1.21
21′	2 $\frac{13}{16}''$ 1.28
22′	2 $\frac{7}{8}''$ 1.36 □
23′	2 $\frac{7}{8}''$ 1.44
24′	2 $\frac{7}{8}''$ 1.55

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TABLE VIII.

TABLE OF SIZES OF HIP VERTICALS FOR BRIDGES OF CLASS C,

In which the live load is eighty pounds per square foot of floor, and the working-stress on the verticals is five tons to the square inch.

The upper figures give the sizes of the hip verticals; the lower ones, the sections required.

Panel Length.	12′ Roadway.	14′ Roadway.	16′ Roadway.	18′ Roadway.	20′ Roadway.	22′ Roadway.	24′ Roadway.	Panel Length.
10′	2 $\frac{3}{4}$″ □ 0.60 □″	2 $\frac{3}{4}$″ □ 0.69 □″	2 $\frac{3}{4}$″ □ 0.79 □″	2 $\frac{3}{4}$″ □ 0.89 □″	2 $\frac{3}{4}$″ □ 1.00 □″	2 $\frac{3}{4}$″ □ 1.12 □″	2 $\frac{13}{16}$″ □ 1.21 □″	10′
11′	2 $\frac{3}{4}$″ □ 0.65 □″	2 $\frac{3}{4}$″ □ 0.75 □″	2 $\frac{3}{4}$″ □ 0.87 □″	2 $\frac{3}{4}$″ □ 0.98 □″	2 $\frac{3}{4}$″ □ 1.10 □″	2 $\frac{13}{16}$″ □ 1.22 □″	2 $\frac{7}{8}$″ □ 1.33 □″	11′
12′	2 $\frac{3}{4}$″ □ 0.71 □″	2 $\frac{3}{4}$″ □ 0.82 □″	2 $\frac{3}{4}$″ □ 0.95 □″	2 $\frac{3}{4}$″ □ 1.07 □″	2 $\frac{13}{16}$″ □ 1.20 □″	2 $\frac{13}{16}$″ □ 1.33 □″	2 $\frac{7}{8}$″ □ 1.45 □″	12′
13′	2 $\frac{3}{4}$″ □ 0.77 □″	2 $\frac{3}{4}$″ □ 0.89 □″	2 $\frac{3}{4}$″ □ 1.03 □″	2 $\frac{13}{16}$″ □ 1.16 □″	2 $\frac{13}{16}$″ □ [illegible] □″	2 $\frac{7}{8}$″ □ 1.43 □″	2 $\frac{15}{16}$″ □ 1.57 □″	13′
14′	2 $\frac{3}{4}$″ □ 0.83 □″	2 $\frac{3}{4}$″ □ 0.97 □″	2 $\frac{3}{4}$″ □ 1.11 □″	2 $\frac{13}{16}$″ □ 1.25 □″	2 $\frac{7}{8}$″ □ 1.40 □″	2 $\frac{7}{8}$″ □ 1.54 □″	2 $\frac{15}{16}$″ □ 1.69 □″	14′
15′	2 $\frac{3}{4}$″ □ 0.89 □″	2 $\frac{3}{4}$″ □ 1.04 □″	2 $\frac{13}{16}$″ □ 1.19 □″	2 $\frac{7}{8}$″ □ 1.34 □″	2 $\frac{7}{8}$″ □ 1.50 □″	2 $\frac{15}{16}$″ □ 1.65 □″	2 1″ □ 1.81 □″	15′
16′	2 $\frac{3}{4}$″ □ 0.95 □″	2 $\frac{3}{4}$″ □ 1.11 □″	2 $\frac{13}{16}$″ □ 1.27 □″	2 $\frac{7}{8}$″ □ 1.42 □″	2 $\frac{15}{16}$″ □ 1.60 □″	2 $\frac{15}{16}$″ □ 1.76 □″	2 1″ □ 1.92 □″	16′
17′	2 $\frac{3}{4}$″ □ 1.01 □″	2 $\frac{13}{16}$″ □ 1.18 □″	2 $\frac{7}{8}$″ □ 1.35 □″	2 $\frac{7}{8}$″ □ 1.51 □″	2 $\frac{15}{16}$″ □ 1.70 □″	2 1″ □ 1.87 □″	2 1$\frac{1}{16}$″ □ 2.04 □″	17′
18′	2 $\frac{3}{4}$″ □ 1.07 □″	2 $\frac{13}{16}$″ □ 1.25 □″	2 $\frac{7}{8}$″ □ 1.43 □″	2 $\frac{15}{16}$″ □ 1.60 □″	2 1″ □ 1.81 □″	2 1″ □ 1.99 □″	2 1$\frac{1}{16}$″ □ 2.16 □″	18′
19′	2 $\frac{3}{4}$″ □ 1.13 □″	2 $\frac{13}{16}$″ □ 1.32 □″	2 $\frac{7}{8}$″ □ 1.51 □″	2 $\frac{15}{16}$″ □ 1.70 □″	2 1″ □ 1.92 □″	2 1$\frac{1}{16}$″ □ 2.11 □″	2 1$\frac{1}{8}$″ □ 2.30 □″	19′
20′	2 $\frac{13}{16}$″ □ 1.21 □″	2 $\frac{7}{8}$″ □ 1.41 □″	2 $\frac{15}{16}$″ □ 1.61 □″	2 1″ □ 1.80 □″	2 1$\frac{1}{16}$″ □ 2.03 □″	2 1$\frac{1}{16}$″ □ 2.23 □″	2 1$\frac{1}{8}$″ □ 2.43 □″	20′
21′	2 $\frac{13}{16}$″ □ 1.28 □″	2 $\frac{7}{8}$″ □ 1.50 □″	2 $\frac{15}{16}$″ □ 1.71 □″	2 1″ □ 1.91 □″	2 1$\frac{1}{16}$″ □ [illegible] □″	2 1$\frac{1}{8}$″ □ 2.36 □″	2 1$\frac{3}{16}$″ □ 2.58 □″	21′
22′	2 $\frac{7}{8}$″ □ 1.36 □″	2 $\frac{15}{16}$″ □ 1.58 □″	2 1″ □ 1.81 □″	2 1″ □ 2.02 □″	2 1$\frac{1}{8}$″ □ 2.28 □″	2 1$\frac{1}{8}$″ □ 2.50 □″	2 1$\frac{3}{16}$″ □ 2.73 □″	22′
23′	2 $\frac{7}{8}$″ □ 1.41 □″	2 $\frac{15}{16}$″ □ 1.67 □″	2 1″ □ 1.90 □″	2 1$\frac{1}{16}$″ □ 2.13 □″	2 1$\frac{1}{8}$″ □ [illegible] □″	2 1$\frac{3}{16}$″ □ 2.64 □″	2 1$\frac{1}{4}$″ □ 2.88 □″	23′
24′	2 $\frac{7}{8}$″ □ 1.55 □″	2 $\frac{15}{16}$″ □ 1.75 □″	2 1″ □ 2.00 □″	2 1$\frac{1}{16}$″ □ 2.24 □″	2 1$\frac{1}{8}$″ □ 2.52 □″	2 1$\frac{3}{16}$″ □ 2.78 □″	2 1$\frac{1}{4}$″ □ 3.03 □″	24′

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Panel Length
10′
11′
12′
13′
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15′
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22′
23′
24′

TABLE C

In which the
stress on the
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tions required.

Panel Length.	12' Roadw
10'	2 ¾" 0.60 □
11'	2 ¾" 0.65 □
12'	2 ¾" 0.71 □
13'	2 ¾" 0.77 □
14'	2 ¾" 0.83 □
15'	2 ¾" 0.89 □
16'	2 ¾" 0.95 □
17'	2 ¾" 1.01 □
18'	2 ¾" 1.07 □
19'	2 ¾" 1.13 □
20'	2 13/16" 1.21 □
21'	2 13/16" 1.28
22'	2 ⅞" 1.36 □
23'	2 ⅞" 1.44 □
24'	2 ⅞" 1.55 □

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Panel Length.
10′
11′
12′
13′
14′
15′
16′
17′
18′
19′
20′
21′
22′
23′
24′

TABLE VIII.

TABLE OF SIZES OF HIP VERTICALS FOR BRIDGES OF CLASS C.

In which the live load is eighty pounds per square foot of floor, and the working-stress on the verticals is five tons to the square inch.

The upper figures give the sizes of the hip verticals; the lower ones, the sections required.

Panel Length.	12′ Roadway.	14′ Roadway.	16′ Roadway.	18′ Roadway.	20′ Roadway.	22′ Roadway.	24′ Roadway.	Panel Length.
10′	2 3/4″ □ 0.60 □″	2 3/4″ □ 0.69 □″	2 3/4″ □ 0.79 □″	2 3/4″ □ 0.89 □″	2 3/4″ □ 1.00 □″	2 3/4″ □ 1.12 □″	2 13/16″ □ 1.21 □″	10′
11′	2 3/4″ □ 0.65 □″	2 3/4″ □ 0.75 □″	2 3/4″ □ 0.87 □″	2 3/4″ □ 0.98 □″	2 3/4″ □ 1.10 □″	2 13/16″ □ 1.22 □″	2 7/8″ □ 1.33 □″	11′
12′	2 3/4″ □ 0.71 □″	2 3/4″ □ 0.82 □″	2 3/4″ □ 0.95 □″	2 3/4″ □ 1.07 □″	2 13/16″ □ 1.20 □″	2 13/16″ □ 1.33 □″	2 7/8″ □ 1.45 □″	12′
13′	2 3/4″ □ 0.77 □″	2 3/4″ □ 0.89 □″	2 3/4″ □ 1.03 □″	2 13/16″ □ 1.16 □″	2 13/16″ □ 1.3[illegible] □″	2 7/8″ □ 1.43 □″	2 15/16″ □ 1.57 □″	13′
14′	2 3/4″ □ 0.83 □″	2 3/4″ □ 0.97 □″	2 3/4″ □ 1.11 □″	2 13/16″ □ 1.25 □″	2 7/8″ □ 1.40 □″	2 7/8″ □ 1.54 □″	2 15/16″ □ 1.69 □″	14′
15′	2 3/4″ □ 0.89 □″	2 3/4″ □ 1.04 □″	2 13/16″ □ 1.19 □″	2 7/8″ □ 1.34 □″	2 7/8″ □ 1.50 □″	2 15/16″ □ 1.65 □″	2 1″ □ 1.81 □″	15′
16′	2 3/4″ □ 0.95 □″	2 3/4″ □ 1.11 □″	2 13/16″ □ 1.27 □″	2 7/8″ □ 1.42 □″	2 15/16″ □ 1.60 □″	2 15/16″ □ 1.76 □″	2 1″ □ 1.92 □″	16′
17′	2 3/4″ □ 1.01 □″	2 13/16″ □ 1.18 □″	2 7/8″ □ 1.35 □″	2 7/8″ □ 1.51 □″	2 15/16″ □ 1.70 □″	2 1″ □ 1.87 □″	2 1 1/16″ □ 2.04 □″	17′
18′	2 3/4″ □ 1.07 □″	2 13/16″ □ 1.25 □″	2 7/8″ □ 1.43 □″	2 15/16″ □ 1.60 □″	2 1″ □ 1.81 □″	2 1″ □ 1.99 □″	2 1 1/16″ □ 2.16 □″	18′
19′	2 3/4″ □ 1.13 □″	2 13/16″ □ 1.32 □″	2 7/8″ □ 1.51 □″	2 15/16″ □ 1.70 □″	2 1″ □ 1.92 □″	2 1 1/16″ □ 2.11 □″	2 1 1/8″ □ 2.30 □″	19′
20′	2 13/16″ □ 1.21 □″	2 7/8″ □ 1.41 □″	2 15/16″ □ 1.61 □″	2 1″ □ 1.80 □″	2 1 1/16″ □ 2.03 □″	2 1 1/16″ □ 2.23 □″	2 1 1/8″ □ 2.43 □″	20′
21′	2 13/16″ □ 1.28 □″	2 7/8″ □ 1.50 □″	2 15/16″ □ 1.71 □″	2 1″ □ 1.91 □″	2 1 1/16″ □ 2.19 □″	2 1 1/8″ □ 2.36 □″	2 1 3/16″ □ 2.58 □″	21′
22′	2 7/8″ □ 1.36 □″	2 15/16″ □ 1.58 □″	2 1″ □ 1.81 □″	2 1″ □ 2.02 □″	2 1 1/8″ □ 2.28 □″	2 1 1/8″ □ 2.50 □″	2 1 3/16″ □ 2.73 □″	22′
23′	2 7/8″ □ 1.44 □″	2 15/16″ □ 1.67 □″	2 1″ □ 1.90 □″	2 1 1/16″ □ 2.13 □″	2 1 1/8″ □ 2.40 □″	2 1 3/16″ □ 2.64 □″	2 1 1/4″ □ 2.88 □″	23′
24′	2 7/8″ □ 1.55 □″	2 15/16″ □ 1.75 □″	2 1″ □ 2.00 □″	2 1 1/16″ □ 2.24 □″	2 1 1/8″ □ 2.52 □″	2 1 3/16″ □ 2.78 □″	2 1 1/4″ □ 3.03 □″	24′

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Diameter.	Intensity of Stress =
	○
$\frac{3}{4}''$	1.268
$\frac{13}{16}''$	1.451
$\frac{7}{8}''$	1.650
$\frac{15}{16}''$	1.885
$1''$	2.140
$1\frac{1}{16}''$	2.423
$1\frac{1}{8}''$	2.726
$1\frac{3}{16}''$	3.057
$1\frac{1}{4}''$	3.408
$1\frac{5}{16}''$	3.787
$1\frac{3}{8}''$	4.190
$1\frac{7}{16}''$	4.617
$1\frac{1}{2}''$	5.068
$1\frac{9}{16}''$	5.547
$1\frac{5}{8}''$	6.040
$1\frac{11}{16}''$	6.573
$1\frac{3}{4}''$	7.120
$1\frac{13}{16}''$	7.695
$1\frac{7}{8}''$	8.294
$1\frac{15}{16}''$	8.917
$2''$	9.568
$2\frac{1}{16}''$	10.239
$2\frac{1}{8}''$	10.938
$2\frac{3}{16}''$	11.657
$2\frac{1}{4}''$	12.404
$2\frac{5}{16}''$	13.175
$2\frac{3}{8}''$	13.970
$2\frac{7}{16}''$	14.793
$2\frac{1}{2}''$	15.636

TABLE IX.

TABLE OF GREATEST WORKING STRESSES,

In tons of two thousand (2,000) pounds, on adjustable round and square rods, exclusive of the initial tensions; also the initial tensions.

Diameter.	Intensity of Working Stress = 4 tons.		Intensity of Working Stress = 5 tons.		Intensity of Working Stress = 7.5 tons.		Initial Tensions.		Diameter.
	●	■	●	■	●	■	●	■	
$\frac{3}{4}''$	1.268	1.517	1.710	2.178	2.815	3.574	0.500	0.635	$\frac{3}{4}''$
$\frac{13}{16}''$	1.451	1.846	1.970	2.507	3.261	4.157	0.625	0.794	$\frac{13}{16}''$
$\frac{7}{8}''$	1.650	2.111	2.257	2.875	3.760	4.789	0.750	0.953	$\frac{7}{8}''$
$\frac{15}{16}''$	1.885	2.405	2.576	3.284	4.303	5.481	0.875	1.111	$\frac{15}{16}''$
$1''$	2.140	2.730	2.927	3.730	4.890	6.230	1.000	1.270	$1''$
$1\frac{1}{16}''$	2.423	3.087	3.308	4.216	5.525	7.038	1.125	1.429	$1\frac{1}{16}''$
$1\frac{1}{8}''$	2.726	3.476	3.620	4.740	6.205	7.904	1.250	1.588	$1\frac{1}{8}''$
$1\frac{3}{16}''$	3.057	3.894	4.163	5.305	6.931	8.830	1.375	1.746	$1\frac{3}{16}''$
$1\frac{1}{4}''$	3.408	4.347	4.636	5.908	7.704	9.814	1.500	1.905	$1\frac{1}{4}''$
$1\frac{5}{16}''$	3.787	4.828	5.140	6.550	8.523	10.856	1.625	2.064	$1\frac{5}{16}''$
$1\frac{3}{8}''$	4.190	5.341	5.675	7.230	9.387	11.956	1.750	2.223	$1\frac{3}{8}''$
$1\frac{7}{16}''$	4.617	5.883	6.240	7.951	10.298	13.117	1.875	2.381	$1\frac{7}{16}''$
$1\frac{1}{2}''$	5.068	6.469	6.836	8.710	11.253	14.335	2.000	2.540	$1\frac{1}{2}''$
$1\frac{9}{16}''$	5.547	7.065	7.463	9.508	12.256	15.611	2.125	2.699	$1\frac{9}{16}''$
$1\frac{5}{8}''$	6.046	7.706	8.119	10.345	13.304	16.947	2.250	2.858	$1\frac{5}{8}''$
$1\frac{11}{16}''$	6.573	8.376	8.808	11.223	14.399	18.342	2.375	3.016	$1\frac{11}{16}''$
$1\frac{3}{4}''$	7.120	9.077	9.527	12.138	15.540	19.794	2.500	3.175	$1\frac{3}{4}''$
$1\frac{13}{16}''$	7.695	9.810	10.276	13.092	16.726	21.305	2.625	3.334	$1\frac{13}{16}''$
$1\frac{7}{8}''$	8.294	10.571	11.056	14.085	17.959	22.874	2.750	3.493	$1\frac{7}{8}''$
$1\frac{15}{16}''$	8.917	11.365	11.867	15.118	19.237	24.503	2.875	3.651	$1\frac{15}{16}''$
$2''$	9.568	12.190	12.708	16.190	20.562	26.190	3.000	3.810	$2''$
$2\frac{1}{16}''$	10.239	13.047	13.580	17.300	21.933	27.935	3.125	3.969	$2\frac{1}{16}''$
$2\frac{1}{8}''$	10.938	13.936	14.483	18.450	23.349	29.739	3.250	4.128	$2\frac{1}{8}''$
$2\frac{3}{16}''$	11.657	14.854	15.417	19.640	24.812	31.603	3.375	4.286	$2\frac{3}{16}''$
$2\frac{1}{4}''$	12.404	15.807	16.380	20.867	26.321	33.524	3.500	4.445	$2\frac{1}{4}''$
$2\frac{5}{16}''$	13.175	16.788	17.375	22.135	28.875	35.504	3.625	4.604	$2\frac{5}{16}''$
$2\frac{3}{8}''$	13.970	17.801	18.400	23.440	29.470	37.541	3.750	4.763	$2\frac{3}{8}''$
$2\frac{7}{16}''$	14.793	18.843	19.457	24.786	31.123	39.640	3.875	4.921	$2\frac{7}{16}''$
$2\frac{1}{2}''$	15.636	19.920	20.544	26.170	32.815	41.795	4.000	5.080	$2\frac{1}{2}''$

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INTANNEL STRUTS.

Ratio, L to D.	■■	●●	Ratio, L to D.	■■	■●	●●
10	4.20	1.419	64	1.569	1.130	0.832
10½	4.17	1.398	64½	1.553	1.116	0.820
11	4.14	1.377	65	1.538	1.102	0.809
11½	4.114	1.356	65½	1.523	1.088	0.797
12	4.08	1.335	66	1.508	1.075	0.786
12½	4.05	1.315	66½	1.493	1.062	0.775
13	4.02	1.296	67	1.479	1.049	0.765
13½	3.99	1.276	67½	1.464	1.036	0.754
14	3.96	1.256	68	1.450	1.024	0.744
14½	3.93	1.238	68½	1.435	1.011	0.734
15	3.901	1.219	69	1.421	0.999	0.724
15½	3.87	1.200	69½	1.407	0.987	0.714
16	3.841	1.182	70	1.393	0.975	0.704
16½	3.811	1.165	70½	1.379	0.963	0.694
17	3.781	1.148	71	1.366	0.951	0.685
17½	3.751	1.130	71½	1.353	0.939	0.676
18	3.721	1.113	72	1.340	0.928	0.667
18½	3.69	1.097	72½	1.327	0.917	0.658
19	3.66	1.081	73	1.314	0.906	0.649
19½	3.63	1.064	73½	1.301	0.895	0.640
20	3.60	1.048	74	1.288	0.885	0.632
20½	3.57	1.033	74½	1.275	0.874	0.624
21	3.54	1.018	75	1.263	0.864	0.616
21½	3.51	1.003	75½	1.251	0.854	0.608
22	3.48	0.988	76	1.238	0.844	0.600
22½	3.45	0.974	76½	1.226	0.834	0.592
23	3.42	0.960	77	1.214	0.824	0.584
23½	3.39	0.946	77½	1.202	0.814	0.576
24	3.36	0.933	78	1.191	0.805	0.569
24½	3.34	0.920	78½	1.179	0.795	0.562
25	3.31	0.907	79	1.168	0.786	0.555
25½	3.28	0.894	79½	1.157	0.777	0.548
26	3.25	0.881	80	1.146	0.768	0.541
26½	3.22	0.868	80½	1.135	0.759	0.534
27	3.19	0.856	81	1.124	0.750	0.527
27½	3.17	0.844	81½	1.114	0.741	0.520

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Ratio, L to D.	■ ■	■ ●	● ●
10	4.205	4.140	3.990
10½	4.175	4.103	3.947
11	4.145	4.066	3.904
11½	4.114	4.029	3.862
12	4.085	3.993	3.819
12½	4.053	3.956	3.775
13	4.023	3.919	3.732
13½	3.993	3.882	3.688
14	3.962	3.845	3.645
14½	3.932	3.807	3.601
15	3.901	3.770	3.557
15½	3.872	3.732	3.514
16	3.841	3.695	3.470
16½	3.811	3.657	3.424
17	3.781	3.620	3.382
17½	3.751	3.583	3.341
18	3.721	3.546	3.295
18½	3.692	3.508	3.252
19	3.662	3.471	3.209
19½	3.632	3.434	3.166
20	3.602	3.397	3.123
20½	3.573	3.360	3.080
21	3.543	3.323	3.038
21½	3.514	3.286	2.996
22	3.485	3.250	2.953
22½	3.456	3.214	2.912
23	3.426	3.178	2.871
23½	3.397	3.142	2.830
24	3.369	3.106	2.790
24½	3.340	3.070	2.750
25	3.311	3.035	2.710
25½	3.282	2.999	2.670
26	3.254	2.964	2.630
26½	3.226	2.929	2.591
27	3.198	2.895	2.553
27½	3.170	2.860	2.515

TABLE X.

INTENSITIES OF WORKING-STRESS FOR CHANNEL STRUTS.

CLASS A.

Ratio, L to D.	■■	■●	●●	Ratio, L to D.	■■	■●	●●	Ratio, L to D.	■■	■●	●●	Ratio, L to D.	■■	■●	●●
10	4.205	4.140	3.990	28	3.142	2.826	2.477	46	2.241	1.792	1.419	64	1.569	1.130	0.832
10½	4.175	4.103	3.947	28½	3.114	2.792	2.441	46½	2.219	1.769	1.398	64½	1.553	1.116	0.820
11	4.145	4.066	3.904	29	3.086	2.759	2.404	47	2.198	1.746	1.377	65	1.538	1.102	0.809
11½	4.114	4.029	3.862	29½	3.059	2.725	2.368	47½	2.176	1.723	1.356	65½	1.523	1.088	0.797
12	4.085	3.993	3.819	30	3.031	2.692	2.332	48	2.155	1.701	1.335	66	1.508	1.075	0.786
12½	4.053	3.956	3.775	30½	3.005	2.659	2.297	48½	2.134	1.679	1.315	66½	1.493	1.062	0.775
13	4.023	3.919	3.732	31	2.976	2.627	2.262	49	2.113	1.658	1.296	67	1.479	1.049	0.765
13½	3.993	3.882	3.688	31½	2.950	2.595	2.227	49½	2.092	1.636	1.276	67½	1.464	1.036	0.754
14	3.962	3.845	3.645	32	2.923	2.563	2.193	50	2.072	1.615	1.256	68	1.450	1.024	0.744
14½	3.932	3.807	3.601	32½	2.897	2.531	2.160	50½	2.051	1.594	1.238	68½	1.435	1.011	0.734
15	3.901	3.770	3.557	33	2.870	2.500	2.127	51	2.031	1.574	1.219	69	1.421	0.999	0.724
15½	3.872	3.732	3.514	33½	2.844	2.469	2.094	51½	2.011	1.554	1.200	69½	1.407	0.987	0.714
16	3.841	3.695	3.470	34	2.818	2.438	2.061	52	1.991	1.534	1.182	70	1.393	0.975	0.704
16½	3.811	3.657	3.424	34½	2.791	2.408	2.030	52½	1.971	1.514	1.165	70½	1.379	0.963	0.694
17	3.781	3.620	3.382	35	2.766	2.378	1.999	53	1.952	1.495	1.148	71	1.366	0.951	0.685
17½	3.751	3.583	3.341	35½	2.740	2.348	1.968	53½	1.933	1.476	1.130	71½	1.353	0.939	0.676
18	3.721	3.546	3.295	36	2.715	2.318	1.937	54	1.914	1.457	1.113	72	1.340	0.928	0.667
18½	3.692	3.508	3.252	36½	2.689	2.289	1.908	54½	1.895	1.438	1.097	72½	1.327	0.917	0.658
19	3.662	3.471	3.209	37	2.664	2.260	1.878	55	1.876	1.420	1.081	73	1.314	0.906	0.649
19½	3.632	3.434	3.166	37½	2.639	2.231	1.849	55½	1.857	1.402	1.064	73½	1.301	0.895	0.640
20	3.602	3.397	3.123	38	2.614	2.203	1.820	56	1.839	1.384	1.048	74	1.288	0.885	0.632
20½	3.573	3.360	3.080	38½	2.589	2.175	1.793	56½	1.821	1.366	1.033	74½	1.275	0.874	0.624
21	3.543	3.323	3.038	39	2.565	2.147	1.765	57	1.803	1.349	1.018	75	1.263	0.864	0.616
21½	3.514	3.286	2.996	39½	2.540	2.119	1.738	57½	1.785	1.332	1.003	75½	1.251	0.854	0.608
22	3.485	3.250	2.953	40	2.516	2.092	1.710	58	1.768	1.315	0.988	76	1.238	0.844	0.600
22½	3.456	3.214	2.912	40½	2.492	2.065	1.684	58½	1.751	1.298	0.974	76½	1.226	0.834	0.592
23	3.426	3.178	2.871	41	2.469	2.039	1.658	59	1.734	1.282	0.960	77	1.214	0.824	0.584
23½	3.397	3.142	2.830	41½	2.445	2.013	1.633	59½	1.717	1.266	0.946	77½	1.202	0.814	0.576
24	3.369	3.106	2.790	42	2.422	1.987	1.607	60	1.700	1.250	0.933	78	1.191	0.805	0.569
24½	3.340	3.070	2.750	42½	2.399	1.962	1.583	60½	1.683	1.235	0.920	78½	1.179	0.795	0.562
25	3.311	3.035	2.710	43	2.375	1.937	1.558	61	1.666	1.219	0.907	79	1.168	0.786	0.555
25½	3.282	2.999	2.670	43½	2.352	1.912	1.533	61½	1.649	1.203	0.894	79½	1.157	0.777	0.548
26	3.254	2.964	2.630	44	2.329	1.887	1.509	62	1.632	1.188	0.881	80	1.146	0.768	0.541
26½	3.226	2.929	2.591	44½	2.307	1.863	1.486	62½	1.616	1.173	0.868	80½	1.135	0.759	0.534
27	3.198	2.895	2.553	45	2.285	1.839	1.464	63	1.600	1.159	0.856	81	1.124	0.750	0.527
27½	3.170	2.860	2.515	45½	2.263	1.815	1.442	63½	1.584	1.145	0.844	81½	1.114	0.741	0.520

Ratio L to …
10
10½
11
11½
12
12½
13
13½
14
14½
15
15½
16
16½
17
17½
18
18½
19
19½
20
20½
21
21½
22
22½
23
23½
24
24½
25
25½
26
26½
27
27½

INTEANNEL STRUTS.

Ratio, L to D.	■ ■	● ●	Ratio, L to D.	■ ■	■ ●	● ●
10	4.367	1.616	64	1.842	1.327	0.976
10½	4.343	1.593	64½	1.826	1.312	0.963
11	4.319	1.570	65	1.809	1.297	0.951
11½	4.294	1.547	65½	1.792	1.282	0.939
12	4.270	1.525	66	1.776	1.267	0.926
12½	4.245	1.503	66½	1.760	1.253	0.915
13	4.220	1.482	67	1.744	1.238	0.903
13½	4.195	1.461	67½	1.728	1.223	0.891
14	4.169	1.440	68	1.711	1.209	0.879
14½	4.144	1.420	68½	1.696	1.195	0.867
15	4.118	1.400	69	1.681	1.182	0.856
15½	4.098	1.380	69½	1.665	1.168	0.845
16	4.068	1.360	70	1.650	1.154	0.834
16½	4.042	1.342	70½	1.636	1.140	0.824
17	4.015	1.323	71	1.622	1.127	0.813
17½	3.990	1.305	71½	1.607	1.115	0.803
18	3.964	1.286	72	1.591	1.103	0.792
18½	3.938	1.268	72½	1.577	1.090	0.782
19	3.912	1.250	73	1.563	1.078	0.772
19½	3.886	1.233	73½	1.549	1.066	0.762
20	3.860	1.215	74	1.534	1.054	0.752
20½	3.833	1.199	74½	1.521	1.042	0.743
21	3.807	1.182	75	1.507	1.031	0.734
21½	3.781	1.166	75½	1.493	1.019	0.725
22	3.755	1.149	76	1.479	1.007	0.716
22½	3.728	1.134	76½	1.466	0.996	0.707
23	3.702	1.118	77	1.453	0.985	0.698
23½	3.676	1.103	77½	1.440	0.974	0.689
24	3.649	1.088	78	1.426	0.963	0.681
24½	3.623	1.073	78½	1.413	0.953	0.673
25	3.596	1.059	79	1.400	0.943	0.665
25½	3.570	1.045	79½	1.388	0.933	0.657
26	3.544	1.031	80	1.376	0.922	0.649
26½	3.518	1.017	80½	1.363	0.912	0.641
27	3.491	1.003	81	1.351	0.902	0.633
27½	3.465	0.990	81½	1.338	0.891	0.624

INTENSITIES

Ratio, L to D.			
10	4.367	4.299	4.143
10½	4.343	4.268	4.105
11	4.319	4.237	4.067
11½	4.294	4.206	4.029
12	4.270	4.175	3.992
12½	4.245	4.142	3.950
13	4.220	4.110	3.914
13½	4.195	4.077	3.875
14	4.169	4.045	3.836
14½	4.144	4.012	3.795
15	4.118	3.979	3.755
15½	4.098	3.946	3.714
16	4.068	3.913	3.674
16½	4.042	3.879	3.633
17	4.015	3.845	3.592
17½	3.990	3.811	3.551
18	3.964	3.777	3.510
18½	3.938	3.743	3.469
19	3.912	3.708	3.428
19½	3.886	3.674	3.387
20	3.860	3.640	3.346
20½	3.833	3.606	3.305
21	3.807	3.571	3.264
21½	3.781	3.537	3.223
22	3.755	3.502	3.182
22½	3.728	3.468	3.142
23	3.702	3.433	3.102
23½	3.676	3.398	3.062
24	3.649	3.364	3.023
24½	3.623	3.329	2.983
25	3.596	3.295	2.943
25½	3.570	3.261	2.903
26	3.544	3.227	2.864
26½	3.518	3.194	2.826
27	3.491	3.160	2.788
27½	3.465	3.127	2.750

TABLE XI.

INTENSITIES OF WORKING-STRESS FOR CHANNEL STRUTS.

CLASSES B AND C.

Ratio, L to D.	■■	■●	●●	Ratio, L to D.	■■	■●	●●	Ratio, L to D.	■■	■●	●●	Ratio, L to D.	■■	■●	●●
10	4.367	4.299	4.143	28	3.439	3.094	2.712	46	2.551	2.040	1.616	64	1.842	1.327	0.976
10½	4.343	4.268	4.105	28½	3.412	3.061	2.674	46½	2.529	2.016	1.593	64½	1.826	1.312	0.963
11	4.319	4.237	4.067	29	3.386	3.028	2.638	47	2.507	1.992	1.570	65	1.809	1.297	0.951
11½	4.294	4.206	4.029	29½	3.360	2.995	2.602	47½	2.485	1.968	1.547	65½	1.792	1.282	0.939
12	4.270	4.175	3.992	30	3.334	2.962	2.565	48	2.463	1.944	1.525	66	1.776	1.267	0.926
12½	4.245	4.142	3.950	30½	3.308	2.930	2.529	48½	2.440	1.921	1.503	66½	1.760	1.253	0.915
13	4.220	4.110	3.914	31	3.282	2.897	2.494	49	2.418	1.898	1.482	67	1.744	1.238	0.903
13½	4.195	4.077	3.875	31½	3.257	2.864	2.459	49½	2.396	1.875	1.461	67½	1.728	1.223	0.891
14	4.169	4.045	3.836	32	3.231	2.832	2.424	50	2.374	1.853	1.440	68	1.711	1.209	0.879
14½	4.144	4.012	3.795	32½	3.205	2.800	2.390	50½	2.353	1.831	1.420	68½	1.696	1.195	0.867
15	4.118	3.979	3.755	33	3.180	2.769	2.356	51	2.333	1.809	1.400	69	1.681	1.182	0.856
15½	4.098	3.946	3.714	33½	3.154	2.738	2.323	51½	2.312	1.787	1.380	69½	1.665	1.168	0.845
16	4.068	3.913	3.674	34	3.129	2.707	2.289	52	2.292	1.766	1.360	70	1.650	1.154	0.834
16½	4.042	3.879	3.633	34½	3.103	2.676	2.257	52½	2.272	1.745	1.342	70½	1.636	1.140	0.824
17	4.015	3.845	3.592	35	3.078	2.646	2.225	53	2.251	1.724	1.323	71	1.622	1.127	0.813
17½	3.990	3.811	3.551	35½	3.052	2.615	2.193	53½	2.231	1.703	1.305	71½	1.607	1.115	0.803
18	3.964	3.777	3.510	36	3.028	2.585	2.161	54	2.211	1.683	1.286	72	1.591	1.103	0.792
18½	3.938	3.743	3.469	36½	3.003	2.556	2.130	54½	2.191	1.665	1.268	72½	1.577	1.090	0.782
19	3.912	3.708	3.428	37	2.978	2.526	2.100	55	2.171	1.648	1.250	73	1.563	1.078	0.772
19½	3.886	3.674	3.387	37½	2.953	2.496	2.069	55½	2.151	1.626	1.233	73½	1.549	1.066	0.762
20	3.860	3.640	3.346	38	2.928	2.467	2.039	56	2.132	1.604	1.215	74	1.534	1.054	0.752
20½	3.833	3.606	3.305	38½	2.904	2.439	2.010	56½	2.113	1.585	1.190	74½	1.521	1.042	0.743
21	3.807	3.571	3.264	39	2.879	2.412	1.981	57	2.094	1.566	1.182	75	1.507	1.031	0.734
21½	3.781	3.537	3.223	39½	2.855	2.385	1.952	57½	2.075	1.547	1.166	75½	1.493	1.019	0.725
22	3.755	3.502	3.182	40	2.831	2.358	1.924	58	2.056	1.529	1.149	76	1.479	1.007	0.716
22½	3.728	3.468	3.142	40½	2.807	2.329	1.896	58½	2.037	1.511	1.134	76½	1.466	0.996	0.707
23	3.702	3.433	3.102	41	2.783	2.301	1.869	59	2.019	1.493	1.118	77	1.453	0.985	0.698
23½	3.676	3.398	3.062	41½	2.755	2.273	1.842	59½	2.000	1.475	1.103	77½	1.440	0.974	0.689
24	3.649	3.364	3.023	42	2.730	2.245	1.815	60	1.982	1.458	1.088	78	1.426	0.963	0.681
24½	3.623	3.329	2.983	42½	2.712	2.218	1.789	60½	1.964	1.441	1.073	78½	1.413	0.953	0.673
25	3.596	3.295	2.943	43	2.689	2.192	1.763	61	1.946	1.424	1.059	79	1.400	0.943	0.665
25½	3.570	3.261	2.903	43½	2.665	2.166	1.738	61½	1.928	1.407	1.045	79½	1.388	0.933	0.657
26	3.544	3.227	2.864	44	2.642	2.140	1.712	62	1.910	1.391	1.031	80	1.376	0.922	0.649
26½	3.518	3.194	2.826	44½	2.619	2.115	1.688	62½	1.893	1.375	1.017	80½	1.363	0.912	0.641
27	3.491	3.160	2.788	45	2.596	2.090	1.664	63	1.876	1.359	1.003	81	1.351	0.902	0.633
27½	3.465	3.127	2.750	45½	2.573	2.065	1.640	63½	1.859	1.343	0.990	81½	1.338	0.891	0.624

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TABLE OF
AND ST
STRESS

Diameter.	Class A
1 1/4"	—
1 3/8"	—
1 1/2"	—
1 5/8"	
1 3/4"	
1 7/8"	
2"	5.9
2 1/8"	7.1
2 1/4"	8.4
2 3/8"	9.9
2 1/2"	11.5
2 5/8"	13.3
2 3/4"	15.3
2 7/8"	17.5
3"	19.9
3 1/8"	22.5
3 1/4"	25.3
3 3/8"	28.3
3 1/2"	31.6
3 5/8"	35.1
3 3/4"	38.8
3 7/8"	42.8
4"	47.1
4 1/8"	51.7
4 1/4"	56.5
4 3/8"	61.7
4 1/2"	67.1
4 5/8"	72.8
4 3/4"	78.9
4 7/8"	85.3
5"	92.0
5 1/8"	99.1
5 1/4"	106.5
5 3/8"	114.3
5 1/2"	122.5
5 5/8"	131.0
5 3/4"	140.0
5 7/8"	149.3
6"	159.0

TABLE XII.

TABLE OF WORKING BENDING-MOMENTS FOR IRON AND STEEL PINS, AND OF WORKING SHEARING-STRESSES FOR STEEL PINS.

Diameter.	Resisting Moments for Bending.					Resisting Shearing-Stresses.		Diameter.
	Iron.			Steel.		Steel.		
	Class A.	Classes B and C.	Lateral System.	Class A.	Classes B and C.	Class A.	Classes B and C.	
$1\frac{1}{4}''$			2.1					$1\frac{1}{4}''$
$1\frac{3}{8}''$			2.9					$1\frac{3}{8}''$
$1\frac{1}{2}''$		3.1	3.7		[illegible]		8.5	$1\frac{1}{2}''$
$1\frac{5}{8}''$		4.0	4.8		6.4		10.0	$1\frac{5}{8}''$
$1\frac{3}{4}''$		4.9	5.9		7.8		11.6	$1\frac{3}{4}''$
$1\frac{7}{8}''$		6.1	7.2		9.8		13.3	$1\frac{7}{8}''$
$2''$	5.9	7.4	8.8	9.4	11.8	12.1	15.2	$2''$
$2\frac{1}{8}''$	7.1	8.9	10.6	11.4	14.2	13.7	17.2	$2\frac{1}{8}''$
$2\frac{1}{4}''$	8.4	10.5	12.6	13.4	[illegible]	15.4	19.2	$2\frac{1}{4}''$
$2\frac{3}{8}''$	9.9	12.3	14.8	15.8	19.7	17.1	21.3	$2\frac{3}{8}''$
$2\frac{1}{2}''$	11.5	14.4	17.2	18.4	23.0	18.9	23.6	$2\frac{1}{2}''$
$2\frac{5}{8}''$	13.3	16.6	19.9	21.3	26.6	20.8	26.1	$2\frac{5}{8}''$
$2\frac{3}{4}''$	15.3	19.1	22.9	24.5	30.6	22.9	28.7	$2\frac{3}{4}''$
$2\frac{7}{8}''$	17.5	21.8	26.2	28.0	34.9	25.1	31.4	$2\frac{7}{8}''$
$3''$	19.9	24.8	29.8	31.8	39.7	27.3	34.2	$3''$
$3\frac{1}{8}''$	22.5	28.1	33.7	36.0	45.0	29.6	37.1	$3\frac{1}{8}''$
$3\frac{1}{4}''$	25.3	31.6	37.9	40.5	50.6	32.0	40.1	$3\frac{1}{4}''$
$3\frac{3}{8}''$	28.3	35.4	42.4	45.3	56.6	34.5	43.2	$3\frac{3}{8}''$
$3\frac{1}{2}''$	31.6	39.5	47.4	50.6	63.2	37.1	46.5	$3\frac{1}{2}''$
$3\frac{5}{8}''$	35.1	43.8	52.6	56.2	70.1	39.8	49.9	$3\frac{5}{8}''$
$3\frac{3}{4}''$	38.8	48.5	58.2	62.1	77.6	42.7	53.4	$3\frac{3}{4}''$
$3\frac{7}{8}''$	42.8	53.5	64.2	68.5	85.6	45.6	57.0	$3\frac{7}{8}''$
$4''$	47.1	58.9	70.6	75.4	94.2	48.6	60.7	$4''$
$4\frac{1}{8}''$	51.7	64.6		82.7	103.6	51.7	64.6	$4\frac{1}{8}''$
$4\frac{1}{4}''$	56.5	70.6		90.4	113.0	54.8	68.6	$4\frac{1}{4}''$
$4\frac{3}{8}''$	61.7	77.1		98.7	123.4	58.1	72.7	$4\frac{3}{8}''$
$4\frac{1}{2}''$	67.1	83.8		107.4	134.1	61.5	76.9	$4\frac{1}{2}''$
$4\frac{5}{8}''$	72.8	91.0		116.5	145.6	65.0	81.2	$4\frac{5}{8}''$
$4\frac{3}{4}''$	78.9	98.6		126.2	157.8	68.5	85.6	$4\frac{3}{4}''$
$4\frac{7}{8}''$	85.3	106.6		136.5	170.6	72.1	90.1	$4\frac{7}{8}''$
$5''$	92.0	115.0		147.2	184.0	75.8	94.8	$5''$
$5\frac{1}{8}''$	99.1	123.8		158.6	198.1	79.7	99.6	$5\frac{1}{8}''$
$5\frac{1}{4}''$	106.5	133.1		170.4	213.0	83.7	104.6	$5\frac{1}{4}''$
$5\frac{3}{8}''$	114.3	142.9		[illegible]	228.7	87.8	109.7	$5\frac{3}{8}''$
$5\frac{1}{2}''$	122.5	153.1		[illegible]	[illegible]	91.9	114.9	$5\frac{1}{2}''$
$5\frac{5}{8}''$	131.0			209.6		96.1		$5\frac{5}{8}''$
$5\frac{3}{4}''$	140.0			224.0		100.4		$5\frac{3}{4}''$
$5\frac{7}{8}''$	149.3			238.9		104.8		$5\frac{7}{8}''$
$6''$	159.0			254.4		109.3		$6''$

I

$W' =$

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FOADS THAT CAN BE

TTrautwine's formula, $W = \dfrac{d^3}{16l^2}$, where

$W =$ lo:

Team, and in right or left hand vertical line for :r, and a horizontal line through the latter, n

	21″	22″	23″	24″	
8′	9.044	10.398	11.882	13.500	8′
9′	7.146	8.216	9.388	10.667	9′
10′	5.788	6.655	7.604	8.640	10′
11′	4.784	5.500	6.285	7.141	11′
12′	4.020	4.622	5.281	6.000	12′
13′	3.425	3.988	4.500	5.112	13′
14′	2.953	3.396	3.880	4.408	14′
15′	2.573	2.958	3.380	3.840	15′
16′	2.261	2.600	2.971	3.375	16′
17′	2.003	2.303	2.631	2.989	17′
18′	1.787	2.054	2.347	2.667	18′
19′	1.603	1.843	2.106	2.393	19′
20′	1.447	1.664	1.901	2.160	20′
21′	1.313	1.509	1.724	1.959	21′
22′	1.196	1.375	1.571	1.785	22′
23′	1.094	1.258	1.438	1.633	23′
24′	1.005	1.156	1.320	1.500	24′
25′	0.926	1.065	1.217	1.382	25′
26′	0.856	0.985	1.125	1.278	26′
27′	0.794	0.913	1.043	1.185	27′
28′	0.738	0.849	0.970	1.102	28′
29′	0.688	0.791	0.904	1.027	29′
30′	0.643	0.740	0.845	0.960	30′

FOR FINDING THE S

The loads being those which will p

W = load in tons, d = depth of beam in

To find the safe distributed load fo line for length of span. The number fo latter, multiplied by the width of beam in

	8″	9″	10″	11″	1.
8′	0.500	0.712	0.977	1.300	1.6
9′	0.395	0.563	0.772	1.027	1.3
10′	0.320	0.456	0.625	0.832	1.0
11′	0.264	0.377	0.517	0.688	0.8
12′	0.222	0.317	0.434	0.578	0.7
13′	0.189	0.270	0.370	0.492	0.6
14′	0.163	0.233	0.319	0.425	0.5
15′	0.142	0.202	0.278	0.370	0.
16′	0.125	0.178	0.244	0.325	0.
17′	0.111	0.158	0.215	0.288	0.
18′	0.099	0.141	0.193	0.257	0.
19′	0.089	0.126	0.173	0.230	0.
20′	0.080	0.114	0.156	0.208	0.
21′	0.073	0.103	0.142	0.189	0.
22′	0.066	0.094	0.129	0.172	0.
23′		0.086	0.118	0.157	0.
24′		0.079	0.109	0.145	0.
25′			0.100	0.133	0.
26′			0.093	0.123	0.
27′				0.114	0.
28′				0.106	0.
29′					0.
30′					0.

TABLE XIII.

FINDING THE SAFE UNIFORMLY DISTRIBUTED LOADS THAT CAN BE BORNE BY PINE BEAMS.

ls being those which will produce a deflection of only $\frac{1}{480}$ of the span, calculated by Trautwine's formula, $W = \frac{d^3}{16l^2}$, where
ons, d = depth of beam in inches, and l = length of span in feet.
the safe distributed load for any span, look in the upper horizontal line for depth of beam, and in right or left hand vertical
of span. The number found at the intersection of a vertical line through the former, and a horizontal line through the
ied by the width of beam in inches, will give the load required.

9″	10″	11″	12″	13″	14″	15″	16″	17″	18″	19″	20″	21″	22″	23″	24″	
0.712	0.977	1.300	1.688	2.146	2.680	3.296	4.000	4.798	5.695	6.698	7.813	9.044	10.398	11.882	13.500	8′
0.563	0.772	1.027	1.333	1.695	2.117	2.604	3.160	3.791	4.500	5.292	6.173	7.146	8.216	9.388	10.667	9′
0.456	0.625	0.832	1.080	1.373	1.715	2.109	2.560	3.071	3.645	4.287	5.000	5.788	6.655	7.604	8.640	10′
0.377	0.517	0.688	0.893	1.135	1.417	1.743	2.116	2.538	3.012	3.543	4.132	4.784	5.500	6.285	7.141	11′
0.317	0.434	0.578	0.750	0.954	1.191	1.465	1.778	2.133	2.531	2.977	3.472	4.020	4.622	5.281	6.000	12′
0.270	0.370	0.492	0.639	0.813	1.015	1.249	1.515	1.817	2.157	2.537	2.959	3.425	3.988	4.500	5.112	13′
0.233	0.319	0.425	0.551	0.701	0.875	1.076	1.306	1.567	1.860	2.187	2.551	2.953	3.396	3.880	4.408	14′
0.202	0.278	0.370	0.480	0.610	0.762	0.937	1.138	1.365	1.620	1.905	2.222	2.573	2.958	3.380	3.840	15′
0.178	0.244	0.325	0.422	0.537	0.670	0.824	1.000	1.200	1.424	1.675	1.953	2.261	2.600	2.971	3.375	16′
0.158	0.215	0.288	0.374	0.475	0.593	0.730	0.886	1.063	1.261	1.483	1.730	2.003	2.303	2.631	2.989	17′
0.141	0.193	0.257	0.333	0.424	0.529	0.651	0.790	0.948	1.125	1.323	1.543	1.787	2.054	2.347	2.667	18′
0.126	0.173	0.230	0.299	0.380	0.475	0.584	0.709	0.851	1.010	1.188	1.385	1.603	1.843	2.106	2.393	19′
0.114	0.156	0.208	0.270	0.343	0.429	0.527	0.640	0.768	0.911	1.072	1.250	1.447	1.664	1.901	2.160	20′
0.103	0.142	0.189	0.245	0.311	0.389	0.478	0.580	0.696	0.827	0.972	1.134	1.313	1.509	1.724	1.959	21′
0.094	0.129	0.172	0.223	0.284	0.354	0.436	0.529	0.635	0.753	0.886	1.033	1.196	1.375	1.571	1.785	22′
0.086	0.118	0.157	0.204	0.260	0.324	0.399	0.484	0.580	0.689	0.810	0.945	1.094	1.258	1.438	1.633	23′
0.079	0.109	0.145	0.188	0.239	0.298	0.366	0.445	0.533	0.633	0.744	0.868	1.005	1.156	1.320	1.500	24′
	0.100	0.133	0.173	0.220	0.274	0.337	0.410	0.491	0.583	0.686	0.800	0.926	1.065	1.217	1.382	25′
	0.093	0.123	0.160	0.200	0.254	0.312	0.379	0.454	0.539	0.634	0.740	0.856	0.985	1.125	1.278	26′
		0.114	0.148	0.180	0.235	0.289	0.351	0.421	0.500	0.588	0.686	0.794	0.913	1.043	1.185	27′
		0.106	0.138	0.175	0.219	0.269	0.327	0.392	0.465	0.547	0.638	0.738	0.849	0.970	1.102	28′
			0.128	0.163	0.204	0.251	0.304	0.365	0.433	0.510	0.595	0.688	0.791	0.904	1.027	29′
			0.120	0.152	0.191	0.234	0.285	0.341	0.405	0.476	0.556	0.643	0.740	0.845	0.960	30′

$W =$

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D LOADS THAT CAN BE

ed by Trautwine's formula, $W = \frac{d^3}{11.5l^2}$, where

$W =$

pth of beam, and in right or left hand vertical line he former, and a horizontal line through the latte

	20″	21″	22″	23″	24″	
9	10.870	12.583	14.467	16.531	18.783	8′
3	8.588	9.942	11.431	13.062	14.841	9′
4	6.957	8.053	9.259	10.580	12.021	10′
9	5.749	6.655	7.652	8.744	9.935	11′
2	4.831	5.592	6.430	7.347	8.348	12′
9	4.116	4.765	5.479	6.260	7.113	13′
3	3.549	4.109	4.724	5.397	6.133	14′
1	3.092	3.579	4.115	4.702	5.343	15′
9	2.718	3.146	3.617	4.133	4.696	16′
4	2.407	2.786	3.204	3.661	4.159	17′
1	2.147	2.485	2.858	3.265	3.710	18′
2	1.927	2.231	2.565	2.931	3.330	19′
1	1.739	2.013	2.315	2.645	3.005	20′
2	1.577	1.826	2.098	2.399	2.726	21′
2	1.437	1.664	1.913	2.186	2.485	22′
8	1.315	1.522	1.750	2.000	2.274	23′
5	1.208	1.398	1.607	1.837	2.087	24′
4	1.113	1.288	1.481	1.693	1.923	25′
2	1.029	1.191	1.369	1.565	1.778	26′
8	0.954	1.105	1.270	1.451	1.649	27′
1	0.887	1.027	1.181	1.349	1.533	28′
9	0.827	0.958	1.101	1.258	1.429	29′
3	0.773	0.895	1.029	1.175	1.336	30′

FOR FINDING THE

The loads being those which will

W = load in tons, d = depth of beam

To find the safe distributed load

line for length of span. The number

latter, multiplied by the width of beam

	8″	9″	10″	11″
8′	0.695	0.991	1.359	1.809
9′	0.550	0.783	1.074	1.430
10′	0.445	0.634	0.870	1.157
11′	0.368	0.524	0.719	0.957
12′	0.309	0.440	0.604	0.804
13′	0.263	0.375	0.515	0.685
14′	0.227	0.323	0.444	0.591
15′	0.198	0.282	0.386	0.514
16′	0.174	0.248	0.340	0.452
17′	0.154	0.219	0.301	0.400
18′	0.137	0.196	0.268	0.357
19′	0.123	0.176	0.241	0.321
20′	0.111	0.158	0.217	0 289
21′	0.101	0 144	0.197	0.262
22′	0.092	0.131	0.179	0.239
23′		0.120	0.164	0.219
24′		0.110	0.151	0.201
25′			0.139	0.185
26′			0.120	0.171
27′				0.160
28′				0.148
29′				
30′				

TABLE XIV.

FINDING THE SAFE UNIFORMLY DISTRIBUTED LOADS THAT CAN BE BORNE BY OAK BEAMS.

oads being those which will produce a deflection of only $\frac{1}{480}$ of the span, calculated by Trautwine's formula, $W = \frac{d^3}{11.5 l^2}$, where n tons, d = depth of beam in inches, and l = length of span in feet.

nd the safe distributed load for any span, look in the upper horizontal line for depth of beam, and in right or left hand vertical gth of span. The number found at the intersection of a vertical line through the former, and a horizontal line through the plied by the width of beam in inches, will give the load required.

″	9″	10″	11″	12″	13″	14″	15″	16″	17″	18″	19″	20″	21″	22″	23″	24″	
95	0.991	1.359	1.809	2.348	2.985	3.728	4.585	5.565	6.675	7.924	9.319	10.870	12.583	14.467	16.531	18.783	8′
550	0.783	1.074	1.430	1.855	2.359	2.946	3.623	4.397	5.274	6.261	7.363	8.588	9.942	11.431	13.062	14.841	9′
445	0.634	0.870	1.157	1.503	1.911	2.386	2.935	3.562	4.272	5.071	5.964	6.957	8.053	9.259	10.580	12.021	10′
368	0.524	0.719	0.957	1.242	1.570	1.972	2.425	2.944	3.531	4.191	4.929	5.749	6.655	7.652	8.744	9.935	11′
309	0.440	0.604	0.804	1.044	1.327	1.657	2.038	2.474	2.967	3.522	4.142	4.831	5.592	6.430	7.347	8.348	12′
263	0.375	0.515	0.685	0.889	1.130	1.412	1.737	2.108	2.528	3.001	3.529	4.116	4.765	5.479	6.260	7.113	13′
227	0.323	0.444	0.591	0.767	0.975	1.217	1.497	1.817	2.179	2.587	3.043	3.549	4.109	4.724	5.397	6.133	14′
98	0.282	0.386	0.514	0.668	0.849	1.060	1.304	1.583	1.899	2.254	2.651	3.092	3.579	4.115	4.702	5.343	15′
74	0.248	0.340	0.452	0.587	0.746	0.932	1.146	1.391	1.669	1.981	2.329	2.718	3.146	3.617	4.133	4.696	16′
54	0.219	0.301	0.400	0.520	0.661	0.826	1.015	1.232	1.478	1.755	2.064	2.407	2.786	3.204	3.661	4.159	17′
37	0.196	0.268	0.357	0.464	0.589	0.736	0.906	1.099	1.318	1.565	1.841	2.147	2.485	2.858	3.265	3.710	18′
23	0.176	0.241	0.321	0.416	0.529	0.661	0.813	0.987	1.183	1.405	1.652	1.927	2.231	2.565	2.931	3.330	19′
11	0.158	0.217	0.289	0.376	0.477	0.596	0.734	0.891	1.068	1.268	1.491	1.739	2.013	2.315	2.645	3.005	20′
01	0.144	0.197	0.262	0.341	0.433	0.541	0.665	0.808	0.969	1.150	1.352	1.577	1.826	2.098	2.399	2.726	21′
92	0.131	0.179	0.239	0.311	0.395	0.493	0.606	0.736	0.883	1.048	1.232	1.437	1.664	1.913	2.186	2.485	22′
	0.120	0.164	0.219	0.284	0.361	0.451	0.555	0.673	0.808	0.959	1.128	1.315	1.522	1.750	2.000	2.274	23′
	0.110	0.151	0.201	0.261	0.332	0.414	0.509	0.617	0.742	0.881	1.035	1.208	1.398	1.607	1.837	2.087	24′
		0.139	0.185	0.240	0.306	0.382	0.470	0.570	0.684	0.811	0.954	1.113	1.288	1.481	1.693	1.923	25′
		0.129	0.171	0.222	0.282	0.353	0.434	0.527	0.632	0.750	0.882	1.029	1.191	1.369	1.565	1.778	26′
			0.160	0.206	0.262	0.327	0.403	0.489	0.586	0.696	0.818	0.954	1.105	1.270	1.451	1.649	27′
			0.148	0.192	0.244	0.304	0.374	0.454	0.545	0.647	0.761	0.887	1.027	1.181	1.349	1.533	28′
				0.179	0.227	0.284	0.349	0.424	0.508	0.603	0.709	0.827	0.958	1.101	1.258	1.429	29′
				0.167	0.212	0.265	0.326	0.396	0.475	0.563	0.663	0.773	0.895	1.029	1.175	1.336	30′

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OF CLASSES A AND B

12″; and guard rails, 6″×6″.

Pa Ler	Roadway, 24′ clear.	No. of Joists.	Size of Joists.	No. Hand-rail Posts per panel.	Panel Length.
1	1201	13	3″ × 10″	2	10′
1	1382	13	3″ × 10″	4	11′
1	1454	13	3″ × 10″	4	12′
1	1710	13	3″ × 12″	4	13′
1	1782	13	3″ × 12″	4	14′
1	2056	15	3″ × 12″	4	15′
1	2136	13	3″×14″	4	16′
1	2453	15	3″ × 14″	4	17′
1	2588	16	3″ × 14″	4	18′
:	3080	15	4″ × 14″	4	19′
:	3339	17	4″ × 14″	4	20′
:	3511	14	4″ × 16″	6	21′
:	3847	16	4″ × 16″	6	22′
:	4344	18	4″ × 16″	6	23′
:	4672	20	4″ × 16″	6	24′

TABLE OF PINE LUMBE

Flooring 3″ thick ; hand-rail po

Panel Length.	Roadway, 12′ clear.	No. of Joists.	Roadway, 14′ clear.	No. of Joists.	Road 16′ cl
10′	691	7	776	8	80
11′	806	7	902	8	99
12′	842	7	944	8	104
13′	990	7	1110	8	123
14′	1026	7	1152	8	127
15′	1180	8	1318	9	145
16′	1224	7	1376	8	152
17′	1400	8	1565	9	173
18′	1439	8	1670	10	1841
19′	1743	8	1950	9	2157
20′	1872	9	2085	10	2392
21′	1933	7	2177	8	2420
22′	2087	8	2336	9	2585
23′	2364	9	2758	11	3024
24′	2528	10	2928	12	3328

TABLE XV.

OF PINE LUMBER REQUIRED PER PANEL IN BRIDGES OF CLASSES A AND B (including waste material).

oring 3″ thick ; hand-rail posts, 4″×6″×4′ ; hand rail, two pieces, 2″×6″ ; hub rails, 2″×12″ ; and guard rails, 6″×6″.

o. of ists.	Roadway, 14′ clear.	No. of Joists.	Roadway, 16′ clear.	No. of Joists.	Roadway, 18′ clear.	No. of Joists.	Roadway, 20′ clear.	No. of Joists.	Roadway, 22′ clear.	No. of Joists.	Roadway, 24′ clear.	No. of Joists.	Size of Joists.	No. Hand-rail Posts per panel.	Panel Length.
7	776	8	861	9	946	10	1031	11	1116	12	1201	13	3″ × 10″	2	10′
7	902	8	998	9	1094	10	1190	11	1286	12	1382	13	3″ × 10″	4	11′
7	944	8	1046	9	1148	10	1250	11	1352	12	1454	13	3″ × 10″	4	12′
7	1110	8	1230	9	1350	10	1470	11	1590	12	1710	13	3″ × 12″	4	13′
7	1152	8	1278	9	1404	10	1530	11	1656	12	1782	13	3″ × 12″	4	14′
8	1318	9	1456	10	1594	11	1732	12	1918	14	2056	15	3″ × 12″	4	15′
7	1376	8	1528	9	1680	10	1832	11	1984	12	2136	13	3″ × 14″	4	16′
8	1565	9	1730	10	1895	11	2060	12	2288	14	2453	15	3″ × 14″	4	17′
8	1670	10	1841	11	2012	12	2246	14	2417	15	2588	16	3″ × 14″	4	18′
8	1950	9	2157	10	2365	11	2572	12	2873	14	3080	15	4″ × 14″	4	19′
[illegible]	2085	10	2392	12	2605	13	2819	14	3125	16	3339	17	4″ × 14″	4	20′
[illegible]	2177	8	2420	9	2663	10	3024	12	3267	13	3511	14	4″ × 16″	6	21′
[illegible]	2336	9	2585	10	2835	11	3201	13	3451	14	3847	16	4″ × 16″	6	22′
[illegible]	2758	11	3024	12	3290	13	3684	15	3950	16	4344	18	4″ × 16″	6	23′
[illegible]	2928	12	3328	14	3500	15	4000	17	4272	18	4672	20	4″ × 16″	6	24′

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.IDGES OF CLASS C

2″; and guard rails, 6″×6″.

Roadway, 24′ clear.	No. of Joists.	Size of Joists.	No. Hand-rail Posts per panel.	Panel Length.
1201	13	3″ × 10″	2	10′
1382	13	3″ × 10″	4	11′
1454	13	3″ × 10″	4	12′
1619	13	3″ × 10″	4	13′
1782	13	3″ × 12″	4	14′
1960	13	3″ × 12″	4	15′
2128	15	3″ × 12″	4	16′
2327	13	3″ × 14″	4	17′
2462	14	3″ × 14″	4	18′
2730	15	3″ × 14″	4	19′
2965	13	4″ × 14″	4	20′
3393	13	4″ × 16″	6	21′
3465	13	4″ × 16″	6	22′
3832	14	4″ × 16″	6	23′
4160	16	4″ × 16″	6	24′

T

TABLE OF PINE LUMBER REQU
(inclu

Flooring 3″ thick; hand-rail posts, 4″×6″×4′; ha

Panel Length.	Roadway, 12′ clear.	No. of Joists.	Roadway, 14′ clear.	No. of Joists.	Roadway, 16′ clear.	No. of Joists.	Roadwa 18′ clea
10′	691	7	776	8	861	9	946
11′	806	7	902	8	998	9	1094
12′	842	7	944	8	1046	9	1148
13′	941	7	1054	8	1167	9	1280
14′	1026	7	1152	8	1278	9	1404
15′	1132	7	1270	8	1408	9	1546
16′	1216	8	1360	9	1504	10	1648
17′	1337	7	1502	8	1667	9	1832
18′	1373	7	1544	8	1715	9	1886
19′	1556	8	1740	9	1924	10	2178
20′	1685	7	1899	8	2112	9	2325
21′	1933	7	2117	8	2420	9	2663
22′	1966	7	2219	8	2468	9	2717
23′	2236	8	2502	9	2768	10	3034
24′	2400	9	2672	10	2944	11	3341

TABLE XVI.

LUMBER REQUIRED PER PANEL IN BRIDGES OF CLASS C (including waste material).

d-rail posts, 4″×6″×4′; hand rail, two pieces, 2″×6″; hub rails, 2″×12″; and guard rails, 6″×6″.

of ts.	Roadway, 16′ clear.	No. of Joists.	Roadway, 18′ clear.	No. of Joists.	Roadway, 20′ clear.	No. of Joists.	Roadway, 22′ clear.	No. of Joists.	Roadway, 24′ clear.	No. of Joists.	Size of Joists.	No. Hand-rail Posts per panel.	Panel Length.
	861	9	946	10	1031	11	1116	12	1201	13	3″ × 10″	2	10′
	998	9	1094	10	1190	11	1286	12	1382	13	3″ × 10″	4	11′
	1046	9	1148	10	1250	11	1352	12	1454	13	3″ × 10″	4	12′
	1167	9	1280	10	1393	11	1504	12	1619	13	3″ × 10″	4	13′
	1278	9	1404	10	1530	11	1656	12	1782	13	3″ × 12″	4	14′
	1408	9	1546	10	1684	11	1822	12	1960	13	3″ × 12″	4	15′
	1504	10	1648	11	1792	12	1984	14	2128	15	3″ × 12″	4	16′
	1667	9	1832	10	1997	11	2162	12	2327	13	3″ × 14″	4	17′
	1715	9	1886	10	2120	12	2291	13	2462	14	3″ × 14″	4	18′
	1924	10	2178	12	2362	13	2546	14	2730	15	3″ × 14″	4	19′
	2112	9	2325	10	2539	11	2752	12	2965	13	4″ × 14″	4	20′
	2420	9	2663	10	2907	11	3150	12	3393	13	4″ × 16″	6	21′
	2468	9	2717	10	2967	11	3216	12	3465	13	4″ × 16″	6	22′
	2768	10	3034	11	3300	12	3566	13	3832	14	4″ × 16″	6	23′
	2944	11	3314	13	3616	14	3888	15	4160	16	4″ × 16″	6	24′

Joist

an l guar

The

Panel Length.
10′
11′
12′
13′
14′
15′
16′
17′
18′
19′
20′
21′
22′
23′
24′

TA PANRIDGES OF

Joists and flo'×4'; haiieces, 2"×6"; hub rail, 2"×12";
an l guard-rail, 6";

The upper fig waste r

Panel Length.	Roadway, 12' clear.	No. of Joists.	). of ists.	Size of Joists.	No. Hand-rail Posts per panel.	Panel Length.
10'	156 417	12	13	2½"×8"	2	10'
11'	200 488	12	13	2½"×9"	4	11'
12'	200 535	12	13	2½"×10"	4	12'
3'	228 615	12	13	2½"×11"	4	13'
14'	228 665	12	13	2½"×12"	4	14'
15'	256 730	12	13	2½"×12"	4	15'
16'	256 81[illegible]	12	13	3"×12"	4	16'
17'	284 920	12	13	3"×13"	4	17'
18'	284 981	12	13	3"×14"	4	18'
19'	312 1142	12	13	3½"×14"	4	19'
20'	312 1253	12	13	4"×14"	4	20'
21'	356 1400	12	13	4"×15"	6	21'
22'	356 1481	12	13	4"×16"	6	22'
23'	384 1586	12	13	4"×16"	6	23'
24'	384 1744	14	15	4"×16"	6	24'

TABLE OF PINE AND O

Joists and flooring of oak, and other lumber pine
and guard-rail, 6″×6″.

The upper figures in each rectangle are for pin

Panel Length.	Roadway, 12′ clear.	No. of Joists.	Roadway, 14′ clear.	No. of Joists.	Roadway, 16′ clear.	N Jo
10′	156 417	7	156 483	8	156 550	
11′	200 488	7	200 565	8	200 643	
12′	200 535	7	200 620	8	200 705	
13′	228 615	7	228 712	8	228 809	
14′	228 665	7	228 770	8	228 875	
15′	256 730	7	256 845	8	256 960	
16′	256 [illegible]	7	256 944	8	256 1072	
17′	284 920	7	284 1063	8	284 1207	
18′	284 981	7	284 1134	8	284 1287	
19′	312 1142	7	312 1318	8	312 1495	
20′	312 1253	7	312 1447	8	312 1640	
21′	356 1400	7	356 1615	8	356 1830	
22′	356 1451	7	356 1709	8	356 1936	
23′	384 1586	7	384 1829	8	384 2072	
24′	384 1744	8	384 1992	9	384 2368	

TABLE XVII.

OF PINE AND OAK LUMBER REQUIRED PER PANEL IN BRIDGES OF CLASSES A AND B.

oak, and other lumber pine. Flooring $2\frac{1}{2}''$ thick; hand-rail posts, $4'' \times 6'' \times 4'$; hand rail, two pieces, $2'' \times 6''$; hub rail, $2'' \times 12''$;

each rectangle are for pine, the lower for oak. The quantities include waste material.

oadway, ' clear.	No. of Joists.	Roadway, 16′ clear.	No. of Joists.	Roadway, 18′ clear.	No. of Joists.	Roadway, 20′ clear.	No. of Joists.	Roadway, 22′ clear.	No. of Joists.	Roadway, 24′ clear.	No. of Joists.	Size of Joists.	No. Hand-rail Posts per panel.	Panel Length.
156 483	8	156 550	9	156 617	10	156 683	11	156 750	12	156 817	13	$2\frac{1}{2}'' \times 8''$	2	10′
200 565	8	200 643	9	200 720	10	200 798	11	200 875	12	200 953	13	$2\frac{1}{2}'' \times 9''$	4	11′
200 620	8	200 705	9	200 790	10	200 875	11	200 960	12	200 1045	13	$2\frac{1}{2}'' \times 10''$	4	12′
228 712	8	228 809	9	228 906	10	228 1003	11	228 1100	12	228 1197	13	$2\frac{1}{2}'' \times 11''$	4	13′
228 770	8	228 875	9	228 980	10	228 1085	11	228 1190	12	228 1295	13	$2\frac{1}{2}'' \times 12''$	4	14′
256 845	8	256 960	9	256 1075	10	256 1190	11	256 1305	12	256 1420	13	$2\frac{1}{2}'' \times 12''$	4	15′
256 944	8	256 1072	9	256 1200	10	256 1328	11	256 1456	12	256 1584	13	$3'' \times 12''$	4	16′
284 1063	8	284 1207	9	284 1350	10	284 1494	11	284 1637	12	284 1781	13	$3'' \times 13''$	4	17′
284 1134	8	284 1287	9	284 1440	10	284 1593	11	284 1746	12	284 1899	13	$3'' \times 14''$	4	18′
312 1318	8	312 1495	9	312 1672	10	312 1848	11	312 2025	12	312 2202	13	$3\frac{1}{2}'' \times 14''$	4	19′
312 1447	8	312 1640	9	312 1833	10	312 2027	11	312 2220	12	312 2413	13	$4'' \times 14''$	4	20′
356 1615	8	356 1830	9	356 2045	10	356 2200	11	356 2475	12	356 2690	13	$4'' \times 15''$	6	21′
356 1709	8	356 1936	9	356 2163	10	356 2391	11	356 2618	12	356 2845	13	$4'' \times 16''$	6	22′
384 1829	8	384 2072	9	384 2315	10	384 2558	11	384 2801	12	384 3044	13	$4'' \times 16''$	6	23′
384 1992	9	384 2368	11	384 2616	12	384 2864	13	384 3112	14	384 3360	15	$4'' \times 16''$	6	24′

Joist

and guar

The

Panel Length.
10′
11′
12′
13′
14′
15′
16′
17′
18′
19′
20′
21′
22′
23′
24′

ANEL IN BRIDGES

Joists hand rail, two pieces, 2"×6"; hub rail, 2"×12";
and guarc
The e material.

Panel Length.	of ts.	Roadway, 24′ clear.	No. of Joists.	Size of Joists.	No. Hand-rail Posts per panel.	Panel Length.
10′		156 817	13	$2\frac{1}{2}'' \times 8''$	2	10′
11′		200 953	13	$2\frac{1}{2}'' \times 9''$	4	11′
12′		200 1013	13	$2\frac{1}{2}'' \times 9''$	4	12′
13′		228 1159	13	$2\frac{1}{2}'' \times 10''$	4	13′
14′		228 1257	13	$2\frac{1}{2}'' \times 11''$	4	14′
15′		256 1420	13	$2\frac{1}{2}'' \times 12''$	4	15′
16′		256 1480	13	$2\frac{1}{2}'' \times 12''$	4	16′
17′		284 1722	13	$3'' \times 12''$	4	17′
18′		284 1841	13	$3'' \times 13''$	4	18′
19′		312 2050	13	$3'' \times 14''$	4	19′
20′		312 2262	13	$3\frac{1}{2}'' \times 14''$	4	20′
21′		356 2518	14	$3\frac{1}{2}'' \times 14''$	6	21′
22′		356 2757	14	$4'' \times 14''$	6	22′
23′		384 3044	13	$4'' \times 16''$	6	23′
24′		384 3104	13	$4'' \times 16''$	6	24′

TABLE OF PINE AN

Joists and flooring of oak, and other lumber
and guard rail, 6″×6″.

The upper figures in each rectangle are for

Panel Length.	Roadway, 12′ clear.	No. of Joists.	Roadway, 14′ clear.	No. of Joists.	Roadway 16′ clear.
10′	156 417	7	156 483	8	156 550
11′	200 488	7	200 565	8	200 643
12′	200 518	7	200 600	8	200 683
13′	228 594	7	228 688	8	228 783
14′	228 645	7	228 747	8	228 849
15′	256 730	7	256 845	8	256 960
16′	256 760	7	256 880	8	256 1000
17′	284 888	7	284 1027	8	284 1166
18′	284 950	7	284 1098	8	284 1246
19′	312 1060	7	312 1225	8	312 1390
20′	312 1172	7	312 1353	8	312 1535
21′	356 1259	7	356 1544	8	356 1738
22′	356 1481	8	356 1694	9	356 1907
23′	384 1586	7	384 1829	8	384 2072
24′	384 1616	7	384 1864	8	384 2112

TABLE XVIII.

[cut off]BLE OF PINE AND OAK LUMBER REQUIRED PER PANEL IN BRIDGES OF CLASS C.

[cut off] of oak, and other lumber pine. Flooring $2\frac{1}{2}''$ thick; hand-rail posts, $4''\times6''\times4'$; hand rail, two pieces, $2''\times6''$; hub rail, $2''\times12''$;

[cut off] in each rectangle are for pine, the lower for oak. The quantities include waste material.

Roadway, 14′ clear.	No. of Joists.	Roadway, 16′ clear.	No. of Joists.	Roadway, 18′ clear.	No. of Joists.	Roadway, 20′ clear.	No. of Joists.	Roadway, 22′ clear.	No. of Joists.	Roadway, 24′ clear.	No. of Joists.	Size of Joists.	No. Hand-rail Posts per panel.	Panel Length.
156 483	8	156 550	9	156 617	10	156 683	11	156 750	12	156 817	13	$2\frac{1}{2}''\times8''$	2	10′
200 565	8	200 643	9	200 720	10	200 798	11	200 875	12	200 953	13	$2\frac{1}{2}''\times9''$	4	11′
200 600	8	200 683	9	200 765	10	200 848	11	200 930	12	200 1013	13	$2\frac{1}{2}''\times9''$	4	12′
228 688	8	228 783	9	228 877	10	228 971	11	228 1065	12	228 1159	13	$2\frac{1}{2}''\times10''$	4	13′
228 747	8	228 849	9	228 951	10	228 1053	11	228 1155	12	228 1257	13	$2\frac{1}{2}''\times11''$	4	14′
256 845	8	256 960	9	256 1075	10	256 1190	11	256 1305	12	256 1420	13	$2\frac{1}{2}''\times12''$	4	15′
256 880	8	256 1000	9	256 1120	10	256 1240	11	256 1360	12	256 1480	13	$2\frac{1}{2}''\times12''$	4	16′
284 1027	8	284 1166	9	284 1305	10	284 1444	11	284 1583	12	284 1722	13	$3''\times12''$	4	17′
284 1098	8	284 1246	9	284 1395	10	284 1544	11	284 1692	12	284 1841	13	$3''\times13''$	4	18′
312 1225	8	312 1390	9	312 1555	10	312 1720	11	312 1885	12	312 2050	13	$3''\times14''$	4	19′
312 1353	8	312 1535	9	312 1717	10	312 1808	11	312 2080	12	312 2262	13	$3\frac{1}{2}''\times14''$	4	20′
356 1544	8	356 1738	10	356 1933	11	356 2128	12	356 2323	13	356 2518	14	$3\frac{1}{2}''\times14''$	6	21′
356 1694	9	356 1907	10	356 2119	11	356 2332	12	356 2545	13	356 2757	14	$4''\times14''$	6	22′
384 1829	8	384 2072	9	384 2315	10	384 2558	11	384 2801	12	384 3044	13	$4''\times16''$	6	23′
384 1864	8	384 2112	9	384 2360	10	384 2608	11	384 2856	12	384 3104	13	$4''\times16''$	6	24′

Panel Length.	
10′	1

Panel Length.		Roadway, 22′ Clear.	Roadway, 24′ Clear.	Panel Length.
10′	[illegible]	Built-beam, 62# per foot Web, $\frac{1}{4}''\times 30''$ [illegible]	Built-beam, $64\frac{1}{2}$# per foot Web, $\frac{1}{4}''\times 30''$ [illegible]	10′

Panel Length.	Roadway, 12' Clear.	Roadway, 14' Clear.	Roadway…
[illegible]	10" 30# I	12" 42# I, or Built beam, 41# per foot Web, ¼" × 19" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	15" 50# I, o… Built beam, … Web, ¼" × 2… Up. fl., 2 2… L. fl., 2 …
[illegible]	10½" 31½# I	12" 42# I, or Built beam, 42½# per foot Web, ¼" × 20" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	15" 50# I, … Built beam, … Web, ¼" × 2… Up. fl., 2 2" … L. fl., 2 2" …
[illegible]	12" 42# I, or Built beam, 35½# per foot Web, ¼" × 18" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 2½" 3.5# angle	12" 42# I, or Built beam, 44# per foot Web, ¼" × 21" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	15" 50# I, … Built beam, … Web, ¼" × 2… Up. fl., 2 2" … L. fl., 2 2" × …
[illegible]	12" 42# I, or Built beam, 36½# per foot Web, ¼" × 18" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	15" 50# I, or Built beam, 45# per foot Web, ¼" × 22" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	15" 50# I, … Built-beam, … Web, ¼" × 2… Up. fl., 2 2" … L. fl., 2 2" × …
[illegible]	12" 42# I, or Built beam, 41# per foot Web, ¼" × 19" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	15" 50# I, or Built beam, 41# per foot Web, ¼" × 24" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	15" 50# I, … Built-beam, … Web, ¼" × 2… Up. fl., 2 2" … L. fl., 2 2½" …
[illegible]	12" 42# I, or Built beam, 42½# per foot Web, ¼" × 20" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	15" 50# I, or Built beam, 48½# per foot Web, ¼" × 25" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	Built-beam, … Web, ¼" × 2… Up. fl., 2 2" … L. fl., 2 2½" …
[illegible]	12" 42# I, or Built beam, 44# per foot Web, ¼" × 21" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	15" 50# I, or Built beam, 50# per foot Web, ¼" × 26" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	Built beam, … Web, ¼" × 2… Up. fl., 2 2" … L. fl., 2 …
[illegible]	15" 50# I, or Built beam, 45# per foot Web, ¼" × 22" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	15" 50# I, or Built beam, 51# per foot Web, ¼" × 25" Up. fl., 2 2" × 3" 4# angle L. fl., 2 2½" × 3" 4.4# angle	Built-beam, … Web, ¼" × 2… Up. fl., 2 2" … L. fl., 2 2" × …
18'	15" 50# I, or Built beam, 46½# per foot Web, ¼" × 23" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	15" 50# I, or Built beam, 52# per foot Web, ¼" × 26" Up. fl., 2 2" × 3" 4# angle L. fl., 2 2½" × 3" 4.4# angle	Built-beam, … Web, ¼" × 3… Up. fl., 2 2" … L. fl., 2 2" × …
19'	15" 50# I, or Built-beam, 48# per foot Web, ¼" × 24" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	Built beam, 53# per foot Web, ¼" × 27" Up. fl., 2 2" × 3" 4# angle L. fl., 2 2½" × 3" 4.4# angle	Built-beam, … Web, ¼" × 3… Up. fl., 2 2" … L. fl., 2 3" × …
20'	15" 50# I, or Built-beam, 49# per foot Web, ¼" × 25" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	Built-beam, 54½# per foot Web, ¼" × 27" Up. fl., 2 2½" × 3" 4.4# angle L. fl., 2 2" × 3" 5# angle	Built-beam, … Web, ¼" × 3… Up. fl., 2 2½… L. fl., 2 2½" …
21'	15" 50# I, or Built beam, 50# per foot Web, ¼" × 26" Up. fl., 2 2" × 2½" 3.5# angle L. fl., 2 2" × 3" 4# angle	Built-beam, 55½# per foot Web, ¼" × 28" Up. fl., 2 2" × 2½" 4.5# angle L. fl., 2 2" × 3" 5# angle	Built beam, … Web, ¼" × 3… Up. fl., 2 2" … L. fl., 2 2½" …
22'	15" 50# I, or Built beam, 51# per foot Web, ¼" × 25" Up. fl., 2 2" × 3" 4# angle L. fl., 2 2½" × 3" 4.4# angle	Built beam, 56½# per foot Web, ¼" × 29" Up. fl., 2 2" × 2½" 4.5# angle L. fl., 2 2" × 3½" 5.3# angle	Built-beam, … Web, ¼" × … Up. fl., 2 2" … L. fl., 2 3" …
23'	15" 50# I, or Built beam, 52# per foot Web, ¼" × 26" Up. fl., 2 2" × 3" 4# angle L. fl., 2 2½" × 3" 4.4# angle	Built-beam, 58½# per foot Web, ¼" × 30" Up. fl., 2 2½" × 2½" 4.9# angle L. fl., 2 2½" × 2½" 5.4# angle	Built beam, … Web, ¼" × … Up. fl., 2 2½… L. fl., 2 3 …
24'	Built beam, 53# per foot Web, ¼" × [illegible]" Up. fl., 2 2" × 3" 4# angle L. fl., 2 2½" × 3" 4.4# angle	Built-beam, 63½# per foot Web, ¼" × 30" Up. fl., 2 2" × 3" 6# angle L. fl., 2 2½" × 3" 6.7# angle	Built beam, … Web, ¼" × … Up. fl., 2 3… L. fl., 2 3" …
Panel Length.	Roadway, 12' Clear.	Roadway, 14' Clear.	Roadw…

TABLE XIX.

TABLE OF FLOOR BEAMS.

CLASS A.

Roadway, 14' Clear.	Roadway, 16' Clear.	Roadway, 18' Clear.	Roadway, 20' Clear.	Roadway, 22' Clear.	Roadway, 24' Clear.	Panel Length.
12" 42# I, or Built beam, 41# per foot Web, ⅜"×19" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built beam, 45# per foot Web, ⅜"×22" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 48½# per foot Web, ⅜"×25" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built beam, 55½# per foot Web, ⅜"×28" Up. fl., 2 2"×3" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built beam, 62# per foot Web, ⅜"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2½"×2½" 6.5# angle	Built-beam, 64½# per foot Web, ⅜"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 3"×3" 7.2# angle	10'
12" 42# I, or Built beam, 42½# per foot Web, ⅜"×20" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built beam, [illegible]# per foot Web, ⅜"×21" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built beam, 52# per foot Web, ⅜"×26" Up. fl., 2 2"×3" 4# angle L. fl., 2 2½"×3" 4.4# angle	Built beam, 57# per foot Web, ⅜"×29" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 64½# per foot Web, ⅜"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 2½"×3" 6.7# angle	Built-beam, 67½# per foot Web, ⅜"×30" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3" 7.7# angle	11'
12" 42# I, or Built beam, 44# per foot Web, ⅜"×21" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built beam, 48½# per foot Web, ⅜"×25" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 54½# per foot Web, ⅜"×27" Up. fl., 2 2"×3" 4.5# angle L. fl., 2 2"×3½" 5# angle	Built beam, 60# per foot Web, ⅜"×30" Up. fl., 2 2"×3" 5# angle L. fl., 2 3"×3" 5.9# angle	Built beam, 64½# per foot Web, ⅜"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 70# per foot Web, ⅜"×30" Up. fl., 2 3"×3½" 7.7# angle L. fl., 2 3"×3" 8.4# angle	12'
15" 50# I, or Built beam, 45# per foot Web, ⅜"×22" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built beam, 50# per foot Web, ⅜"×26" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built beam, 55½# per foot Web, ⅜"×28" Up. fl., 2 2"×3" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built beam, 62# per foot Web, ⅜"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2½"×2½" 6.5# angle	Built beam, 67½# per foot Web, ⅜"×30" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3" 7.7# angle	Built beam, 75# per foot Web, 7/16"×31" Up. fl., 2 2½"×2½" 6.5# angle L. fl., 2 3"×3" 7.2# angle	13'
15" 50# I, or Built beam, 41# per foot Web, ⅜"×24" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 52# per foot Web, ⅜"×26" Up. fl., 2 2"×3" 4# angle L. fl., 2 2½"×3" 4.4# angle	Built-beam, 57# per foot Web, ⅜"×29" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 63½# per foot Web, ⅜"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 2½"×3" 6.7# angle	Built-beam, 70# per foot Web, ⅜"×30" Up. fl., 2 3"×3½" 7.7# angle L. fl., 2 3"×3" 8.8# angle	Built-beam, 78# per foot Web, 7/16"×36" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	14'
15" 50# I, or Built beam, 48½# per foot Web, ⅜"×25" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 53# per foot Web, ⅜"×27" Up. fl., 2 2"×3" 4# angle L. fl., 2 2½"×3" 4.4# angle	Built beam, 58# per foot Web, ⅜"×30" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built beam, 65# per foot Web, ⅜"×30" Up. fl., 2 2"×3½" 6.4 angle L. fl., 2 3"×3" 7.2# angle	Built beam, 75# per foot Web, 7/16"×34" Up. fl., 2 2½"×2½" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 81# per foot Web, 7/16"×38" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	15'
15" 50# I, or Built beam, 50# per foot Web, ⅜"×26" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built beam, 55½# per foot Web, ⅜"×28" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 [illegible]×3" 5# angle	Built-beam, 60# per foot Web, ⅜"×30" Up. fl., 2 2"×3" 5# angle L. fl., 2 3"×3" 5.9# angle	Built beam, 67½# per foot Web, ⅜"×30" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3" 7.7# angle	Built-beam, 76½# per foot Web, 7/16"×35" Up. fl., 2 2½"×2½" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 82½# per foot Web, 7/16"×38" Up. fl., 2 2½"×3" 6.7# angle L. fl., 2 3"×3½" 7.7# angle	16'
15" 50# I, or Built beam, 51# per foot Web, ⅜"×25" Up. fl., 2 2"×3" 4# angle L. fl., 2 2½"×3" 4.4# angle	Built-beam, 56½# per foot Web, ⅜"×29" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built beam, 63½# per foot Web, ⅜"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2½"×2½" 6.5# angle	Built beam, 70# per foot Web, ⅜"×30" Up. fl., 2 3"×3½" 7.7# angle L. fl., 2 3"×3" 8.4# angle	Built-beam, 78# per foot Web, 7/16"×36" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 86# per foot Web, 7/16"×38" Up. fl., 2 3"×3½" 7.7# angle L. fl., 2 3"×3" 8.4# angle	17'
15" 50# I, or Built beam, 52# per foot Web, ⅜"×26" Up. fl., 2 2"×3" 4# angle L. fl., 2 2½"×3" 4.4# angle	Built beam, [illegible]8# per foot Web, ⅜"×3[illegible] Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 65# per foot Web, ⅜"×30" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 75# per foot Web, 7/16"×34" Up. fl., 2 2½"×2½" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 81# per foot Web, 7/16"×38" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 89# per foot Web, 7/16"×38" Up. fl., 2 3"×3" 8.4# angle L. fl., 2 3"×3½" 9# angle	18'
Built beam, 53# per foot Web, ⅜"×27" Up. fl., 2 2"×3" 4# angle L. fl., 2 2½"×3" 4.4# angle	Built-beam, 60# per foot Web, ⅜"×30" Up. fl., 2 2"×3" 5# angle L. fl., 2 3"×3" 5.9# angle	Built-beam, 66½# per foot Web, ⅜"×30" Up. fl., 2 2½"×3" 6.7# angle L. fl., 2 3"×3½" 7.7# angle	Built-beam, 76½# per foot Web, 7/16"×35" Up. fl., 2 2½"×2½" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 82½# per foot Web, 7/16"×38" Up. fl., 2 2½"×3" 6.7# angle L. fl., 2 3"×3½" 7.7# angle	Built-beam, 91# per foot Web, 7/16"×38" Up. fl., 2 3"×3½" 9# angle L. fl., 2 3"×4" 9.7# angle	19'
Built-beam, 54½# per foot Web, ⅜"×27" Up. fl., 2 2½"×3" 4.4# angle L. fl., 2 2"×4" 5# angle	Built-beam, 6[illegible]# per foot Web, ⅜"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2½"×2½" 6.5# angle	Built-beam, 69# per foot Web, ⅜"×30" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3" 8.4# angle	Built-beam, 78# per foot Web, 7/16"×36" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 86# per foot Web, 7/16"×38" Up. fl., 2 3"×3½" 7.7# angle L. fl., 2 3"×3" 8.4# angle	Built-beam, 94# per foot Web, 7/16"×38" Up. fl., 2 3"×4" 9.7# angle L. fl., 2 3"×3½" 10.4# angle	20'
Built-beam, 55½# per foot Web, ⅜"×28" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3" 5# angle	Built beam, 63½# per foot Web, ⅜"×3[illegible] Up. fl., 2 2"×3" 6# angle L. fl., 2 2½"×3" 6.7# angle	Built-beam, 70# per foot Web, ⅜"×30" Up. fl., 2 3"×3½" 7.7# angle L. fl., 2 3"×3" 8.4# angle	Built-beam, 79½# per foot Web, 7/16"×37" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 89# per foot Web, 7/16"×38" Up. fl., 2 3"×4" 8.4# angle L. fl., 2 3"×3½" 9# angle	Built-beam, 98# per foot Web, 7/16"×38" Up. fl., 2 3"×3½" 10.4# angle L. fl., 2 3"×3½" 11.7# angle	21'
Built beam, 56½# per foot Web, ⅜"×29" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 65# per foot Web, ⅜"×3[illegible] Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×[illegible] 7.2# angle	Built beam, 75# per foot Web, 7/16"×31" Up. fl., 2 2½"×2½" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built beam, 82½# per foot Web, 7/16"×38" Up. fl., 2 2½"×3" 6.7# angle L. fl., 2 3"×3½" 7.7# angle	Built-beam, 91# per foot Web, 7/16"×38" Up. fl., 2 3"×3½" 9# angle L. fl., 2 3"×4" 9.7# angle	Built-beam, 101# per foot Web, 7/16"×38" Up. fl., 2 3"×4" 11.2# angle L. fl., 2 3"×4" 12.7# angle	22'
Built-beam, 58½# per foot Web, ⅜"×30" Up. fl., 2 2½"×2½" 4.9# angle L. fl., 2 2½"×2½" 5.4# angle	Built beam, 6[illegible]# per foot Web, ⅜"×30" Up. fl., [illegible] 6.7# angle L. fl., 2 3[illegible]# angle	Built-beam, 76½# per foot Web, 7/16"×35" Up. fl., 2 2½"×2½" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built beam, 85# per foot Web, 7/16"×38" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3" 8.4# angle	Built-beam, 94# per foot Web, 7/16"×38" Up. fl., 2 3"×4" 9.7# angle L. fl., 2 3"×3½" 10.4# angle	Built-beam, 103# per foot Web, 7/16"×38" Up. fl., 2 3½"×4" 12# angle L. fl., 2 3½"×4" 13.6# angle	23'
Built beam, 63½# per foot Web, ⅜"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 2½"×3" 6.7# angle	Built beam, [illegible] per foot Web, ⅜"×3[illegible] Up. fl., 2 3[illegible]# angle L. fl., 2 3"×[illegible]# angle	Built beam, 78# per foot Web, 7/16"×36" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 86# per foot Web, 7/16"×38" Up. fl., 2 3"×3½" 7.[illegible]# angle L. fl., 2 3"×3" 8.4# angle	Built-beam, 98# per foot Web, 7/16"×38" Up. fl., 2 3"×3½" 10.4# angle L. fl., 2 3"×3½" 11.7# angle	Built-beam, 106# per foot Web, 7/16"×38" Up. fl., 2 4"×4" 12.6# angle L. fl., 2 3½"×4" 13.6# angle	24'
Roadway, 14' Clear.	Roadway, 16' Clear.	Roadway, 18' Clear.	Roadway, 20' Clear.	Roadway, 22' Clear.	Roadway, 24' Clear.	Panel Length.

Panel Length.	Roadway, 12 Clear.	Roadway, 14' Clear.	Roadwa
10	10" 30# I	10½" 31½# I	12" 42# I, Built-beam, Web, ¼"×1 Up. fl., 2 2" L. fl., 2 2"×
11'	10" 30# I	12" 42# I, or Built-beam, 38½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×2½" 3.5# angle	12" 42# I, Built-beam, Web, ¼"× Up. fl., 2 2' L. fl., 2 2"×
[illegible]	10" 30# I	12" 42# I, or Built-beam, 39½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, Built-beam, Web, ¼"× Up. fl., 2 2 L. fl., 2 2'×
[illegible]	10½" 31½# I	12" 42# I, or Built beam, 41# per foot Web, ¼"×19" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, Built-beam, Web, ¼"×2 Up. fl., 2 2" L. fl., 2 2"×
[illegible]	12" 42# I, or Built-beam, 38½# per foot Web, ¼"×1[illegible]" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×2½" 3.5# angle	12" 42# I, or Built beam, 42½# per foot Web, ¼"×20" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I. Built-beam, Web, ¼"× Up. fl., 2 2" L. fl., 2 2"×
[illegible]	12" 42# I, or Built-beam, 38½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×2½" 3.5# angle	15" 50# I, or Built-beam, 42½# per foot Web, ¼"×20" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, Built-beam Web, ¼"× Up. fl., 2 2 L. fl., 2 2"×
[illegible]	12" 42# I, or Built-beam, 39½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 44# per foot Web, ¼"×21" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I. Built beam Web, ¼"× Up. fl., 2 2 L. fl., 2 2½"
17'	12" 42# I, or Built beam, 39½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 45# per foot Web, ¼"×22" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I. Built-beam Web, ¼"× Up. fl., 2 2 L. fl., 2 2"
18'	12" 42# I, or Built-beam, 41# per foot Web, ¼"×19" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 40# per foot Web, ¼"×23" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam Web, ¼"× Up. fl., 2 2 L. fl., 2 2"
19'	12" 42# I, or Built-beam, 42½# per foot Web, ¼"×20" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 4[illegible]# per foot Web, ¼"×25" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam Web, ¼"× Up. fl., 2 2 L. fl., 2 2"
20'	15" 50# I, or Built-beam, 44# per foot Web, ¼"×21" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 50½# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam Web, ¼"× Up. fl., 2 L. fl., 2 2"
21'	15" 50# I, or Built-beam, 45# per foot Web, ¼"×22" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 52# per foot Web, ¼"×24" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	Built-beam Web, ¼"× Up. fl., 2 2 L. fl., 2 2½
22'	15" 50# I, or Built beam, 46# per foot Web, ¼"×23" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 52½# per foot Web, ¼"×25" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	Built-beam Web, ¼"× Up. fl., 2 L. fl., 2 2,
23'	15" 50# I, or Built beam, 47½# per foot Web, ¼"×24" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built beam, 53# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	Built-beam Web, ¼"× Up. fl., 2 L. fl., 2 3
24'	15" 50# I, or Built beam, [illegible] per foot Web, ¼"×25" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built beam, 51# per foot Web, ¼"×27" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	Built bea Web, ¼ Up. fl., 2 L. fl., 2 2
Panel Length.	Roadway, 12 Clear	Roadway, 14 Clear	Road

TABLE XX.

TABLE OF FLOOR BEAMS.

CLASS B.

Roadway, 14′ Clear.	Roadway, 16′ Clear.	Roadway, 18′ Clear.	Roadway, 20′ Clear.	Roadway, 22′ Clear.	Roadway, 24′ Clear.	Panel Length.
10½" 31½# I	12" 42# I, or Built-beam, 41½# per foot Web, ¼"×19" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 46# per foot Web, ¼"×22" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 50½# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 51# per foot Web, ¼"×27" Up. fl., 2 2"×3" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 58½# per foot Web, ¼"×30" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	10′
12" 42# I, or Built-beam, 38½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×2½" 3.5# angle	12" 42# I, or Built-beam, 43½# per foot Web, ¼"×20" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 48½# per foot Web, ¼"×24" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 53½# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 55½# per foot Web, ¼"×29" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 62# per foot Web, ¼"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2¾"×2¾" 6.5# angle	11′
12" 42# I, or Built-beam, 39½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 45# per foot Web, ¼"×21" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 50½# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 51½# per foot Web, ¼"×27" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 57# per foot Web, ¼"×30" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 63½# per foot Web, ¼"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 2½"×3" 6.7# angle	12′
12" 42# I, or Built-beam, 41# per foot Web, ¼"×19" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 46# per foot Web, ¼"×22" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 51½# per foot Web, ¼"×21" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 55½# per foot Web, ¼"×28" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 60# per foot Web, ¼"×30" Up. fl., 2 2"×3" 5# angle L. fl., 2 3"×3" 5.9# angle	Built-beam, 65# per foot Web, ¼"×30" Up. fl., 2 3"×3" 6.5# angle L. fl., 2 3"×3" 7.2# angle	13′
12" 42# I, or Built-beam, 42½# per foot Web, ¼"×20" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 47½# per foot Web, ¼"×23" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 52½# per foot Web, ¼"×25" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 57# per foot Web, ¼"×29" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 62# per foot Web, ¼"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2¾"×2¾" 6.5# angle	Built-beam, 67½# per foot Web, ¼"×30" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3½" 7.7# angle	14′
15" 50# I, or Built-beam, 42½# per foot Web, ¼"×20" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 49# per foot Web, ¼"×25" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 53½# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 59# per foot Web, ¼"×30" Up. fl., 2 2½"×2½" 4.9# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 63½# per foot Web, ¼"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 2½"×3" 6.7# angle	Built-beam, 70# per foot Web, ¼"×30" Up. fl., 2 3"×3½" 7.7# angle L. fl., 2 3"×3" 8.4# angle	15′
15" 50# I, or Built-beam, 44# per foot Web, ¼"×21" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 51½# per foot Web, ¼"×26" Up. fl., 2 2"×3" 4# angle L. fl., 2 2½"×3" 4.4# angle	Built-beam, 55½# per foot Web, ¼"×28" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 60# per foot Web, ¼"×30" Up. fl., 2 2"×3" 5# angle L. fl., 2 3"×3" 5.9# angle	Built-beam, 65½# per foot Web, ¼"×30" Up. fl., 2 3"×3" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 75# per foot Web, 5/16"×34" Up. fl., 2 2¾"×2¾" 6.5# angle L. fl., 2 3"×3" 7.2# angle	16′
15" 50# I, or Built-beam, 45# per foot Web, ¼"×22" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 51½# per foot Web, ¼"×25" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3" 5# angle	Built-beam, 57# per foot Web, ¼"×29" Up. fl., 2 2½"×2½" 4.9# angle L. fl., 2 2¾"×2¾" 5.4# angle	Built-beam, 62# per foot Web, ¼"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2¾"×2¾" 6.5# angle	Built-beam, 70# per foot Web, ¼"×30" Up. fl., 2 3"×3½" 7.7# angle L. fl., 2 3"×3" 8.4# angle	Built-beam, 76½# per foot Web, 5/16"×35" Up. fl., 2 2¾"×2¾" 6.5# angle L. fl., 2 3"×3" 7.2# angle	17′
15" 50# I, or Built-beam, 46# per foot Web, ¼"×23" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 52½# per foot Web, ¼"×29" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 58# per foot Web, ¼"×30" Up. fl., 2 2½"×2½" 4.9# angle L. fl., 2 2¾"×2¾" 5.4# angle	Built-beam, 63½# per foot Web, ¼"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 2½"×3" 6.7# [illegible]	Built-beam, 75# per foot Web, 5/16"×34" Up. fl., 2 2¾"×2¾" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 78# per foot Web, 5/16"×36" Up. fl., 2 2¾"×2¾" 6.5# angle L. fl., 2 3"×3" 7.2# angle	18′
15" 50# I, or Built-beam, 49# per foot Web, ¼"×25" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 53½# per foot Web, ¼"×27" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.5# angle	Built-beam, 60# per foot Web, ¼"×30" Up. fl., 2 2"×3½" 5.3# angle L. fl., 2 3"×3" 5.9# angle	Built-beam, 65# per foot Web, ¼"×30" Up. fl., 2 2"×3½" 6.4# [illegible] L. fl., 2 3"×3" 7.2# [illegible]	Built-beam, 76½# per foot Web, 5/16"×35" Up. fl., 2 2¾"×2¾" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 81# per foot Web, 5/16"×38" Up. fl., 2 2¾"×2¾" 6.5# angle L. fl., 2 3"×3" 7.2# angle	19′
15" 50# I, or Built-beam, 50½# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 55# per foot Web, ¼"×28" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 61½# per foot Web, ¼"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2"×3½" 6.4# angle	Built-beam, 67½# per foot Web, ¼"×30" Up. fl., 2 2½"×3" 7.2# [illegible] L. fl., 2 3"×3½" 7.7# [illegible]	Built-beam, 78# per foot Web, 5/16"×36" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 85# per foot Web, 5/16"×38" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3" 8.4# angle	20′
15" 50# I, or Built-beam, 52# per foot Web, ¼"×24" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 57# per foot Web, ¼"×29" Up. fl., 2 2½"×2½" 4.9# angle L. fl., 2 2¾"×2¾" 5.4# angle	Built-beam, 63½# per foot Web, ¼"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 2½"×3" 6.7# angle	Built-beam, 70# per foot Web, ¼"×30" Up. fl., 2 3"×3½" 7.7# [illegible] L. fl., 2 3"×3" 8.4# [illegible]	Built-beam, 79½# per foot Web, 5/16"×37" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 86# per foot Web, 5/16"×38" Up. fl., 2 3"×3½" 7.7# angle L. fl., 2 3"×3" 8.4# angle	21′
Built-beam, 52½# per foot Web, ¼"×25" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 58# per foot Web, ¼"×30" Up. fl., 2 2½"×2½" 4.9# angle L. fl., 2 2¾"×2¾" 5.4# angle	Built-beam, 65# per foot Web, ¼"×30" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 75# per foot Web, 5/16"×34" Up. fl., 2 2¾"×2¾" 6.5# [illegible] L. fl., 2 3"×3" 7.2# angle	Built-beam, 81# per foot Web, 5/16"×38" Up. fl., 2 2"×3½" 6.4# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 89# per foot Web, 5/16"×38" Up. fl., 2 3"×3" 8.4# angle L. fl., 2 3"×3½" 9# angle	22′
Built-beam, 53# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 60# per foot Web, ¼"×30" Up. fl., 2 2"×3½" 5.3# angle L. fl., 2 3"×3" 5.9# angle	Built-beam, 66½# per foot Web, ¼"×30" Up. fl., 2 2½"×3" 6.7# angle L. fl., 2 3"×3½" 7.7# angle	Built-beam, 76½# per foot Web, 5/16"×35" Up. fl., 2 2¾"×2¾" 6.5# [illegible] L. fl., 2 3"×3" [illegible]	Built-beam, 82½# per foot Web, 5/16"×38" Up. fl., 2 2½"×3" 6.7# angle L. fl., 2 3"×3½" 7.7# angle	Built-beam, 91# per foot Web, 5/16"×38" Up. fl., 2 3"×3½" 9# angle L. fl., 2 3"×4" 9.7# angle	23′
Built-beam, 54# per foot Web, ¼"×27" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3½" 5.3# angle	Built-beam, 61½# per foot Web, ¼"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2"×3½" 6.4# angle	Built-beam, 69# per foot Web, ¼"×30" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3" 8.4# angle	Built-beam, [illegible]# per foot Web, 5/16"×30" Up. fl., 2 2¾"×2¾" 6.5# [illegible] L. fl., 2 3"×3" [illegible]	Built-beam, 85# per foot Web, 5/16"×38" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3" 8.4# angle	Built-beam, 94# per foot Web, 5/16"×38" Up. fl., 2 3"×4" 9.7# angle L. fl., 2 3"×3½" 10.4# angle	24′
Roadway, 14′ Clear.	Roadway, 16′ Clear.	Roadway, 18′ Clear.	Roadway, 20′ Clear.	Roadway, 22′ Clear.	Roadway, 24′ Clear.	Panel Length.

Panel
Length.

Panel Length.	Roadway, 22′ Clear.	Roadway, 24′ Clear.	Panel Length.

Panel Length.	Roadway, 12′ Clear.	Roadway, 14′ Clear.	Roadway
[illegible]	9″ 23¼# I	10″ 30# I	12″ 42# I, o Built-beam, Web, ¼″ × 18 Up. fl., 2 2″ L. fl., 2 2″ ×
[illegible]	9″ 23¼# I	10″ 30# I	12″ 42# I, o Built-beam, Web, ¼″ × 18 Up. fl., 2 2″ × L. fl., 2 2″ ×
[illegible]	10″ 30# I	10½″ 31½# I	12″ 42# I, o Built-beam, Web, ¼″ × 1 Up. fl., 2 2″ L. fl., 2 2″ ×
[illegible]	10″ 30# I	12″ 42# I, or Built-beam, 38½# per foot Web, ¼″ × 18″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 2½″ 3.5# angle	12″ 42# I, o Built-beam, Web, ¼″ × 1 Up. fl., 2 2″ L. fl., 2 2″ ×
[illegible]	1[illegible] 30# I	12″ 42# I, or Built-beam, 38½# per foot Web, ¼″ × 18″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 2½″ 3.5# angle	12″ 42# I, o Built-beam, Web, ¼″ × 20 Up. fl., 2 2″ L. fl., 2 2″ ×
[illegible]	1[illegible] 30# I	12″ 42# I, or Built-beam, 39½# per foot Web, ¼″ × 18″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, o Built-beam, Web, ¼″ × 2 Up. fl., 2 2″ L. fl., 2 2″ ×
[illegible]	10½″ 31½# I	12″ 42# I, or Built-beam, 41# per foot Web, ¼″ × 19″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, o Built-beam, Web, ¼″ × 2 Up. fl., 2 2″ L. fl., 2 2″ ×
[illegible]	12″ 42# I, or Built beam, 38½# per foot Web, ¼″ × 18″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 2½″ 3.5# angle	12″ 42# I, or Built-beam, 42½# per foot Web, ¼″ × 20″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, o Built-beam, Web, ¼″ × 2 Up. fl., 2 2″ L. fl., 2 2″ ×
18′	12″ 42# I, or Built-beam, 38½# per foot Web, ¼″ × 18″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 2½″ 3.5# angle	15″ 50# I, or Built-beam, 42½# per foot Web, ¼″ × 20″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, o Built-beam, Web, ¼″ × 2 Up. fl., 2 2″ L. fl., 2 2″ ×
19′	12″ 42# I, or Built-beam, 39½# per foot Web, ¼″ × 18″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, or Built-beam, 44# per foot Web, ¼″ × 21″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, o Built-beam, Web, ¼″ × 2 Up. fl., 2 2″ L. fl., 2 2½″
20′	12″ 42# I, or Built-beam, 39½# per foot Web, ¼″ × 18″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, or Built beam, 45# per foot Web, ¼″ × 22″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, o Built-beam, Web, ¼″ × 2 Up. fl., 2 2″ L. fl., 2 2½″
21′	12″ 42# I, or Built-beam, 41# per foot Web, ¼″ × 19″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, or Built-beam, 46# per foot Web, ¼″ × 23″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	Built-beam, Web, ¼″ × 2 Up. fl., 2 2 L. fl., 2 2
22′	12″ 42# I, or Built beam, 4[illegible]½# per foot Web, ¼″ × 20″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, or Built-beam, 47½# per foot Web, ¼″ × 24″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	Built-beam, Web, ¼″ × 2 Up. fl., 2 2″ L. fl. [illegible]
23′	15″ 50# I, or Built beam, 44# per foot Web, ¼″ × 21″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, or Built-beam, 49# per foot Web, ¼″ × 25″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	Built bea[illegible] Web, ¼″ × Up. fl., L. fl., 2
24′	15″ 50# I, or Built beam, 43# per foot Web, ¼″ × 22″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	15″ 50# I, or Built beam, 50½# per foot Web, ¼″ × 26″ Up. fl., 2 2″ × 2½″ 3.5# angle L. fl., 2 2″ × 3″ 4# angle	Built beam, Web, ¼″ × 2 Up. fl., 2 2″ L. fl., 2 2″ ×
Panel Length.	Roadway, 12′ Clear.	Roadway, 14′ Clear.	Roadw

TABLE XXI.

TABLE OF FLOOR BEAMS.

CLASS C.

Roadway, 14' Clear.	Roadway, 16' Clear.	Roadway, 18' Clear.	Roadway, 20' Clear.	Roadway, 22' Clear.	Roadway, 24' Clear.	Panel Length.
10" 30# I	12" 42# I, or Built-beam, 39½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×2½ 3.5# angle	12" 42# I, or Built-beam, 43½# per foot Web, ¼"×20" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 48½# per foot Web, ¼"×24" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built beam, 51# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 56# per foot Web, ¼"×28" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	10'
10" 30# I	12" 42# I, or Built-beam, 40½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	12" 42# I, or Built-beam, 45# per foot Web, ¼"×21" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 49½# per foot Web, ¼"×25" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built beam, 53# per foot Web, ¼"×26" Up. fl., 2 2"×3" 4# angle L. fl., 2 2"×2½" 4.5# angle	Built-beam, 57# per foot Web, ¼"×29" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	11'
10½" 31½# I	12" 42# I, or Built-beam, 41½# per foot Web, ¼"×19" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 46# per foot Web, ¼"×22" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 50½# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built beam, 53# per foot Web, ¼"×26" Up. fl., 2 2"×3" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 58½# per foot Web, ¼"×30" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	12'
12" 42# I, or Built beam, 38½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×2½" 3.5# angle	12" 42# I, or Built-beam, 41½# per foot Web, ¼"×19" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 47# per foot Web, ¼"×23" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 52½# per foot Web, ¼"×26" Up. fl., 2 2"×3" 4# angle L. fl., 2 2"×2½" 4.5# angle	Built beam, 54½# per foot Web, ¼"×28" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 60# per foot Web, ¼"×30" Up. fl., 2 2"×3" 5# angle L. fl., 2 3"×3" 5.9# angle	13'
12" 42# I, or Built-beam, 38½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×2½" 3.5# angle	12" 42# I, or Built-beam, 43½# per foot Web, ¼"×20" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 48½# per foot Web, ¼"×24" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 53½# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 55½# per foot Web, ¼"×29" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 62# per foot Web, ¼"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2¾"×2¾" 6.5# angle	14'
12" 42# I, or Built-beam, 39½# per foot Web, ¼"×18" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 45# per foot Web, ¼"×21" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 50½# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built beam, 54½# per foot Web, ¼"×27" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 57# per foot Web, ¼"×30" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	Built-beam, 63½# per foot Web, ¼"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 2½"×3" 6.7# angle	15'
12" 42# I, or Built-beam, 41# per foot Web, ¼"×19" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 46# per foot Web, ¼"×22" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 51½# per foot Web, ¼"×24" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 55½# per foot Web, ¼"×28" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 60# per foot Web, ¼"×30" Up. fl., 2 2"×3" 5# angle L. fl., 2 3"×3" 5.9# angle	Built-beam, 65# per foot Web, ¼"×30" Up. fl., 2 3"×3" 6.5# angle L. fl., 2 3"×3" 7.2# angle	16'
12" 42# I, or Built-beam, 42½# per foot Web, ¼"×20" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 47½# per foot Web, ¼"×23" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 52½# per foot Web, ¼"×25" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 57# per foot Web, ¼"×29" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	Built-beam, 62# per foot Web, ¼"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2¾"×2¾" 6.5# angle	Built-beam, 67½# per foot Web, ¼"×30" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3½" 7.7# angle	17'
15" 50# I, or Built-beam, 42½# per foot Web, ¼"×20" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 49# per foot Web, ¼"×25" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×[illegible] 4# angle	Built-beam, 53½# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2½"×3" 5# angle	Built-beam, 59# per foot Web, ¼"×30" Up. fl., 2 2½"×2½" 4.9# angle L. fl., 2 2"×3¼" 5.3# angle	Built-beam, 63½# per foot Web, ¼"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 2½"×3" 6.7# angle	Built-beam, 70# per foot Web, ¼"×30" Up. fl., 2 3"×3½" 7.7# angle L. fl., 2 3"×3" 8.4# angle	18'
15" 50# I, or Built-beam, 44# per foot Web, ¼"×21" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 50½# per foot Web, ¼"×25" Up. fl., 2 2"×3" 4# angle L. fl., 2 2½"×3" 4.4# angle	Built-beam, 54½# per foot Web, ¼"×27" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	Built-beam, 60# per foot Web, ¼"×30" Up. fl., 2 2"×3" 5# angle L. fl., 2 3"×3" 5.9# angle	Built-beam, 65½# per foot Web, ¼"×30" Up. fl., 2 3"×3" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 75# per foot Web, 5/16"×34" Up. fl., 2 2¾"×2¾" 6.5# angle L. fl., 2 3"×3" 7.2# angle	19'
15" 50# I, or Built beam, 45# per foot Web, ¼"×22" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	15" 50# I, or Built-beam, 51½# per foot Web, ¼"×26" Up. fl., 2 2"×3" 4# angle L. fl., 2 2½"×[illegible] 4.4# angle	Built-beam, 55½# per foot Web, ¼"×28" Up. fl., 2 2"×2½" 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	Built beam, 62# per foot Web, ¼"×30" Up. fl., 2 2½"×3" 5.5# [illegible] L. fl., 2 2¾"×2¾" 6.5# [illegible]	Built-beam, 67½# per foot Web, ¼"×30" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3½" 7.7# angle	Built-beam, 76½# per foot Web, 5/16"×35" Up. fl., 2 2¾"×2¾" 6.5# angle L. fl., 2 3"×3" 7.2# angle	20'
15" 50# I, or Built-beam, 46# per foot Web, ¼"×23" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 51½# per foot Web, ¼"×2[illegible]" Up. fl., 2 2"×[illegible] 4.5# angle L. fl., 2 [illegible] # angle	Built-beam, 57# per foot Web, ¼"×29" Up. fl., 2 2¼"×2¼" 4.9# angle L. fl., 2 2¼"×2¼" 5.4# angle	Built-beam, 63½# per foot Web, ¼"×30" Up. fl., 2 2"×3" 6# angle L. fl., 2 2½"×3" 6.7# angle	Built beam, 70# per foot Web, ¼"×30" Up. fl., 2 3"×3½" 7.7# angle L. fl., 2 3"×3" 8.4# angle	Built-beam, 78# per foot Web, 5/16"×36" Up. fl., 2 2"×3¼" 6.4# angle L. fl., 2 3"×3" 7.2# angle	21'
15" 50# I, or Built-beam, 47½# per foot Web, ¼"×24" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built-beam, 52½# per foot Web, ¼"×26" Up. fl., 2 2"×2½ 4.5# angle L. fl., 2 2"×3¼" 5.3# angle	Built-beam, 58# per foot Web, ¼"×30" Up. fl., 2 2¼"×2¼" 4.9# angle L. fl., 2 2¼"×2¼" 5.4# angle	Built beam, 65# per foot Web, ¼"×30" Up. fl., 2 2"×3¼" 6.4# [illegible] L. fl., 2 3"×3" 7.2# [illegible]	Built-beam, 75# per foot Web, 5/16"×34" Up. fl., 2 2¾"×2¾" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 81# per foot Web, 5/16"×38" Up. fl., 2 2"×3¼" 6.4# angle L. fl., 2 3"×3" 7.2# angle	22'
15" 50# I, or Built beam, 49# per foot Web, ¼"×25" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built be[illegible] per foot Web, ¼"×[illegible] Up. fl., [illegible] 4.5# angle L. fl., [illegible] 5.3# angle	Built beam, 60# per foot Web, ¼"×30" Up. fl., 2 2"×3¼" 5.3# angle L. fl., 2 3"×3" 5.9# angle	Built beam, 66½# per [illegible] Web, ¼"×30" Up. fl., 2 2½"×3" 6.7# [illegible] L. fl., 2 3"×3½" 7.7# [illegible]	Built beam, 76½# per foot Web, 5/16"×35" Up. fl., 2 2¾"×2¾" 6.5# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 82½# per foot Web, 5/16"×38" Up. fl., 2 2½"×3" 6.7# angle L. fl., 2 3"×3½" 7.7# angle	23'
15" 50# I, or Built beam, 50½# per foot Web, ¼"×26" Up. fl., 2 2"×2½" 3.5# angle L. fl., 2 2"×3" 4# angle	Built beam, 55# per foot Web, ¼"×28" Up. fl., 2 2"×[illegible] 4.5# angle L. fl., 2 2"×3¼" [illegible]# angle	Built-beam, 61½# per foot Web, ¼"×30" Up. fl., 2 2½"×3" 5.5# angle L. fl., 2 2"×3¼" 6.4# angle	Built beam, 66# per [illegible] Web, ¼"×30" Up. fl., 2 3"×3" 7.2# [illegible] L. fl., 2 3"×3" 8.4# [illegible]	Built beam, 78# per foot Web, 5/16"×36" Up. fl., 2 2"×3¼" 6.4# angle L. fl., 2 3"×3" 7.2# angle	Built-beam, 85# per foot Web, 5/16"×38" Up. fl., 2 3"×3" 7.2# angle L. fl., 2 3"×3" 8.4# angle	24'
Roadway, 14' Clear.	Roadway, 16' Clear.	Roadway, 18' Clear.	Roadway, 20' Clear.	Roadway, 22' Clear.	Roadway, 24' Clear.	Panel Length.

TABLE XXII.

TABLE OF BEAM HANGERS.

TABLE XXII.

TABLE OF BEAM HANGERS.

CLASS A.

Panel Length.	Roadway, 12′ clear.	Roadway, 14′ clear.	Roadway, 16′ clear.	Roadway, 18′ clear.	Roadway, 20′ clear.	Roadway, 22′ clear.	Roadway, 24′ clear.	Panel Length.
	$1\frac{1}{4}''$ ⊙ N. up.	$1\frac{3}{8}''$ ⊙ N. up.	$1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{2}''$ ⊙ N. up.	$1\frac{1}{2}''$ ⊙ N. up.	$1\frac{5}{8}''$ ⊙ N. up.	$1\frac{5}{8}''$ ⊙ N. up.	20′
21′	$\frac{13}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{4}''$ ⊙ Up. $1''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{3}{16}''$ ⊙ Up. $1\frac{1}{16}''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{1}{4}''$ ⊙ Up. $1\frac{1}{8}''$ □ Up. $1\frac{5}{8}''$ ⊙ N. up.	$1\frac{5}{16}''$ ⊙ Up. $1\frac{3}{16}''$ □ Up. $1\frac{3}{4}''$ ⊙ N. up.	21′
22′	$\frac{15}{16}''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1\frac{1}{8}''$ ⊙ Up. $1''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{3}{16}''$ ⊙ Up. $1\frac{1}{16}''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{1}{4}''$ ⊙ Up. $1\frac{1}{8}''$ □ Up. $1\frac{5}{8}''$ ⊙ N. up.	$1\frac{5}{16}''$ ⊙ Up. $1\frac{1}{8}''$ □ Up. $1\frac{3}{4}''$ ⊙ N. up.	$1\frac{3}{8}''$ ⊙ Up. $1\frac{3}{16}''$ □ Up. $1\frac{3}{4}''$ ⊙ N. up.	22′
23′	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{8}''$ ⊙ Up. $1''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{3}{16}''$ ⊙ Up. $1\frac{1}{16}''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{1}{4}''$ ⊙ Up. $1\frac{1}{8}''$ □ Up. $1\frac{5}{8}''$ ⊙ N. up.	$1\frac{5}{16}''$ ⊙ Up. $1\frac{3}{16}''$ □ Up. $1\frac{3}{4}''$ ⊙ N. up.	$1\frac{3}{8}''$ ⊙ Up. $1\frac{1}{4}''$ □ Up. $1\frac{3}{4}''$ ⊙ N. up.	23′
24′	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $1''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{3}{16}''$ ⊙ Up. $1\frac{1}{16}''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{1}{4}''$ ⊙ Up. $1\frac{1}{8}''$ □ Up. $1\frac{5}{8}''$ ⊙ N. up.	$1\frac{5}{16}''$ ⊙ Up. $1\frac{1}{8}''$ □ Up. $1\frac{3}{4}''$ ⊙ N. up.	$1\frac{3}{8}''$ ⊙ Up. $1\frac{3}{16}''$ □ Up. $1\frac{3}{4}''$ ⊙ N. up.	$1\frac{7}{16}''$ ⊙ Up. $1\frac{1}{4}''$ □ Up. $1\frac{7}{8}''$ ⊙ N. up.	24′

TABLE XXII.

TABLE OF BEAM HANGERS.

CLASS A.

Four hangers per beam. Up. denotes ends upset; N. up., ends not upset.

Panel Length.	Roadway, 12′ clear.	Roadway, 14′ clear.	Roadway, 16′ clear.	Roadway, 18′ clear.	Roadway, 20′ clear.	Roadway, 22′ clear.	Roadway, 24′ clear.	Panel Length.
10′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	10′
11′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	11′
12′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	12′
13′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	13′

Panel Length.	Roadway, 12′ clear.	Roadway, 14′ clear.	Roadway, 16′ clear.	Roadway, 18′ clear.	Roadway, 20′ clear.	Roadway, 22′ clear.	Roadway, 24′ clear.	Panel Length.
10′	3/4″ □ Up. 1″ • N. up.	3/4″ □ Up. 1″ ⊙ N. up.	3/4″ □ Up. 1″ ⊙ N. up.	3/4″ □ Up. 1 1/8″ • N. up.	3/4″ □ Up. 1 1/4″ • N. up.	3/4″ □ Up. 1 1/4″ ⊙ N. up.	13/16″ □ Up. 1 1/4″ ⊙ N. up.	10′
11′	3/4″ • Up. 3/4″ □ Up. 1″ • N. up.	3/4″ • Up. 3/4″ □ Up. 1″ • N. up.	13/16″ • Up. 3/4″ □ Up. 1 1/8″ • N. up.	13/16″ • Up. 3/4″ □ Up. 1 1/8″ • N. up.	7/8″ • Up. 3/4″ □ Up. 1 1/4″ • N. up.	15/16″ ⊙ Up. 13/16″ □ Up. 1 1/4″ ⊙ N. up.	15/16″ • Up. 7/8″ □ Up. 1 1/4″ ⊙ N. up.	11′
12′	3/4″ ⊙ Up. 3/4″ □ Up. 1″ • N. up.	3/4″ • Up. 3/4″ □ Up. 1″ ⊙ N. up.	13/16″ • Up. 3/4″ □ Up. 1 1/8″ ⊙ N. up.	7/8″ ⊙ Up. 3/4″ □ Up. 1 1/4″ • N. up.	15/16″ • Up. 13/16″ □ Up. 1 1/4″ ⊙ N. up.	15/16″ ⊙ Up. 7/8″ □ Up. 1 1/4″ • N. up.	1″ ⊙ Up. 7/8″ □ Up. 1 3/8″ ⊙ N. up.	12′
13′	3/4″ • Up. 3/4″ □ Up. 1″ • N. up.	13/16″ • Up. 3/4″ □ Up. 1 1/8″ ⊙ N. up.	7/8″ • Up. 3/4″ □ Up. 1 1/4″ • N. up.	7/8″ • Up. 13/16″ □ Up. 1 1/4″ • N. up.	15/16″ • Up. 13/16″ □ Up. 1 1/4″ • N. up.	1″ • Up. 7/8″ □ Up. 1 3/8″ • N. up.	1 1/16″ ⊙ Up. 15/16″ □ Up. 1 3/8″ ⊙ N. up.	13′
14′	3/4″ • Up. 3/4″ □ Up. 1″ • N. up.	15/16″ • Up. 3/4″ □ Up. 1 1/8″ ⊙ N. up.	7/8″ • Up. 3/4″ □ Up. 1 1/4″ ⊙ N. up	15/16″ • Up. 13/16″ □ Up. 1 1/4″ ⊙ N. up.	1″ • Up. 7/8″ □ Up. 1 3/8″ ⊙ N. up.	1″ • Up. 7/8″ □ Up. 1 3/8″ • N. up.	1 1/16″ • Up. 15/16″ □ Up. 1 3/8″ ⊙ N. up.	14′
15′	15/16″ • Up. 3/4″ □ Up. 1 1/8″ • N. up.	7/8″ ⊙ Up. 3/4″ □ Up. 1 1/4″ • N. up.	15/16″ ⊙ Up. 13/16″ □ Up. 1 1/4″ • N. up.	15/16″ ⊙ Up. 13/16″ □ Up. 1 1/4″ • N. up.	1″ ⊙ Up. 7/8″ □ Up. 1 3/8″ • N. up.	1 1/16″ • Up. 15/16″ □ Up. 1 3/8″ ⊙ N. up.	1 1/8″ ⊙ Up. 1″ □ Up. 1 1/2″ ⊙ N. up.	15′
16′	15/16″ • Up. 3/4″ □ Up. 1 1/8″ • N. up.	7/8″ • Up. 13/16″ □ Up. 1 1/4″ ⊙ N. up.	15/16″ ⊙ Up. 13/16″ □ Up. 1 1/4″ ⊙ N. up.	1″ ⊙ Up. 7/8″ □ Up. 1 3/8″ ⊙ N. up.	1 1/16″ • Up. 15/16″ □ Up. 1 3/8″ ⊙ N. up.	1 1/16″ ⊙ Up. 1″ □ Up. 1 3/8″ ⊙ N. up.	1 1/8″ ⊙ Up. 1″ □ Up. 1 1/2″ ⊙ N. up.	16′
17′	13/16″ ⊙ Up. 3/4″ □ Up. 1 1/8″ ⊙ N. up.	7/8″ ⊙ Up. 13/16″ □ Up. 1 1/4″ ⊙ N. up.	15/16″ ⊙ Up. 7/8″ □ Up. 1 1/4″ ⊙ N. up.	1″ ⊙ Up. 15/16″ □ Up. 1 3/8″ ⊙ N. up.	1 1/16″ ⊙ Up. 15/16″ □ Up. 1 3/8″ ⊙ N. up.	1 1/8″ ⊙ Up. 1″ □ Up. 1 1/2″ ⊙ N. up.	1 3/16″ ⊙ Up. 1 1/16″ □ Up. 1 1/2″ ⊙ N. up.	17′
18′	7/8″ ⊙ Up. 3/4″ □ Up. 1 1/4″ ⊙ N. up.	15/16″ ⊙ Up. 13/16″ □ Up. 1 1/4″ ⊙ N. up.	1″ • Up. 7/8″ □ Up. 1 3/8″ ⊙ N. up.	1 1/16″ ⊙ Up. 15/16″ □ Up. 1 3/8″ ⊙ N. up.	1 1/8″ ⊙ Up. 1″ □ Up. 1 1/2″ ⊙ N. up.	1 3/16″ ⊙ Up. 1″ □ Up. 1 1/2″ ⊙ N. up.	1 3/16″ ⊙ Up. 1 1/16″ □ Up. 1 1/2″ ⊙ N. up.	18′
19′	7/8″ • Up. 13/16″ □ Up. 1 1/4″ • N. up.	15/16″ ⊙ Up. 7/8″ □ Up. 1 1/4″ ⊙ N. up.	1″ ⊙ up. 7/8″ □ Up. 1 3/8″ ⊙ N. up.	1 1/16″ ⊙ Up. 15/16″ □ Up. 1 3/8″ ⊙ N. up.	1 1/8″ ⊙ up. 1″ □ Up. 1 1/2″ ⊙ N. up.	1 3/16″ ⊙ Up. 1 1/16″ □ Up. 1 1/2″ ⊙ N. up.	1 1/4″ ⊙ Up. 1 1/8″ □ Up. 1 5/8″ ⊙ N. up.	19′
20′	15/16″ ⊙ Up. 13/16″ □ Up. 1 1/4″ • N. up.	1″ ⊙ Up. 7/8″ □ Up. 1 3/8″ • N. up.	1 1/16″ ⊙ Up. 15/16″ □ Up. 1 3/8″ • N. up.	1 1/8″ ⊙ Up. 1″ □ Up 1 1/2″ ⊙ N. up.	1 3/16″ ⊙ Up. 1 1/16″ □ Up. 1 1/2″ • N. up.	1 1/4″ ⊙ Up. 1 1/16″ □ Up. 1 5/8″ ⊙ N. up.	1 1/4″ ⊙ Up. 1 1/8″ □ Up. 1 5/8″ ⊙ N. up.	20′
21′	15/16″ • Up. 13/16″ □ Up. 1 1/4″ • N. up.	1″ • Up. 7/8″ □ Up. 1 3/8″ • N. up.	[illegible] • Up. [illegible] □ Up. 1 3/8″ ⊙ N. up.	1 1/8″ • Up. 1″ □ Up. 1 1/2″ • N. up.	1 3/16″ ⊙ Up. 1 1/16″ □ Up. 1 1/2″ • N. up.	1 1/4″ ⊙ Up. 1 1/8″ □ Up. 1 5/8″ • N. up.	1 5/16″ ⊙ Up. 1 3/16″ □ Up. 1 3/4″ • N. up.	21
22′	15/16″ • Up. 7/8″ □ Up. 1 1/4″ ⊙ N. up.	1 1/16″ • Up. 15/16″ □ Up. 1 3/8″ ⊙ N. up.	1 1/8″ • Up. 1″ □ Up. 1 1/2″ ⊙ N. up.	1 3/16″ ⊙ Up. 1 1/16″ □ Up. 1 1/2″ ⊙ N. up.	1 1/4″ ⊙ Up. 1 1/8″ □ Up. 1 5/8″ ⊙ N. up.	1 5/16″ ⊙ Up. 1 1/8″ □ Up. 1 3/4″ ⊙ N. up.	1 3/8″ • Up. 1 3/16″ □ Up. 1 3/4″ ⊙ N. up.	22′
23′	1″ ⊙ Up. 7/8″ □ Up. 1 3/8″ • N. up.	1 1/16″ ⊙ Up. 15/16″ □ Up. 1 3/8″ ⊙ N. up.	1 1/8″ ⊙ Up. 1″ □ Up. 1 1/2″ ⊙ N. up.	1 3/16″ ⊙ Up. 1 1/16″ □ Up. 1 1/2″ ⊙ N. up.	1 1/4″ ⊙ Up. 1 1/8″ □ Up. 1 5/8″ ⊙ N. up.	1 5/16″ ⊙ Up. 1 3/16″ □ Up. 1 3/4″ ⊙ N. up.	1 3/8″ ⊙ Up. 1 1/4″ □ Up. 1 3/4″ ⊙ N. up.	23′
24′	1″ • Up. 7/8″ □ Up. 1 3/8″ ⊙ N. up.	1 1/16″ ⊙ Up. 1″ □ Up. 1 3/8″ ⊙ N. up.	1 3/16″ • Up. 1 1/16″ □ Up. 1 1/2″ • N. up.	1 1/4″ ⊙ Up. 1 1/8″ □ Up. 1 5/8″ ⊙ N. up.	1 5/16″ ⊙ Up. 1 1/8″ □ Up. 1 3/4″ ⊙ N. up.	1 3/8″ ⊙ Up. 1 3/16″ □ Up. 1 3/4″ ⊙ N. up.	1 7/16″ ⊙ Up. 1 1/4″ □ Up. 1 7/8″ ⊙ N. up.	24′

TABLE XXIII.

TABLE XXIII.

TABLE OF BEAM HANGERS.

Panel Length.	Roadway, 12' clear.	Roadway, 14' clear.	Roadway, 16' clear.	Roadway, 18' clear.	Roadway, 20' clear.	Roadway, 22' clear.	Roadway, 24' clear.	Panel Length.
20'	3/4" □ Up. 1 1/4" ⊙ N. up.	3/4" □ Up. 1 1/4" ⊙ N. up.	13/16" □ Up. 1 1/4" ⊙ N. up.	7/8" □ Up. 1 1/4" ⊙ N. up.	7/8" □ Up. 1 3/8" • N. up.	15/16" □ Up. 1 3/8" ⊙ N. up.	1" □ Up. 1 1/2" ⊙ N. up.	20'
21'	13/16" ⊙ Up. 3/4" □ Up. 1 1/4" • N. up.	7/8" • Up. 13/16" □ Up. 1 1/4" • N. up.	15/16" • Up. 13/16" □ Up. 1 1/4" ⊙ N. up.	1" ⊙ Up. 7/8" □ Up. 1 3/8" ⊙ N. up.	1 1/16" ⊙ Up. 15/16" □ Up. 1 3/8" • N. up.	1 1/16" • Up. 1" □ Up. 1 3/8" • N. up.	1 1/8" ⊙ Up. 1" □ Up. 1 1/2" ⊙ N. up.	21'
22'	13/16" • Up. 3/4" □ Up. 1 1/8" • N. up.	7/8" • Up. 13/16" □ Up. 1 1/4" ⊙ N. up.	15/16" • Up. 7/8" □ Up. 1 1/4" ⊙ N. up.	1" ⊙ Up. 15/16" □ Up. 1 3/8" • N. up.	1 1/16" ⊙ Up. 15/16" □ Up. 1 3/8" • N. up.	1 1/8" • Up. 1" □ Up. 1 1/2" ⊙ N. up.	1 3/16" • Up. 1 1/16" □ Up. 1 1/2" ⊙ N. up.	22'
23'	7/8" • Up. 3/4" □ Up. 1 1/4" ⊙ N. up.	15/16" ⊙ Up. 13/16" □ Up. 1 1/4" ⊙ N. up.	1" • Up. 7/8" □ Up. 1 1/4" • N. up.	1 1/16" • Up. 15/16" □ Up. 1 3/8" • N. up.	1 1/8" • Up. 1" □ Up. 1 1/2" • N. up.	1 1/8" • Up. 1" □ Up. 1 1/2" ⊙ N. up.	1 3/16" • Up. 1 1/16" □ Up. 1 1/2" ⊙ N. up.	23'
24'	7/8" ⊙ Up. 13/16" □ Up. 1 1/4" ⊙ N. up.	15/16" • Up. 13/16" □ Up. 1 1/4" ⊙ N. up.	1" ⊙ Up. 7/8" □ Up. 1 3/8" • N. up.	1 1/16" ⊙ Up. 15/16" □ Up. 1 3/8" • N. up.	1 1/8" ⊙ Up. 1" □ Up. 1 1/2" • N. up.	1 3/16" • Up. 1 1/16" □ Up. 1 1/2" • N. up.	1 1/4" • Up. 1 1/4" □ Up. 1 5/8" • N. up.	24'

TABLE XXIII.

TABLE OF BEAM HANGERS.

CLASS B.

Four hangers per beam. Up. denotes ends upset; N. up., ends not upset.

Panel Length.	Roadway, 12′ clear.	Roadway, 14′ clear.	Roadway, 16′ clear.	Roadway, 18′ clear.	Roadway, 20′ clear.	Roadway, 22′ clear.	Roadway, 24′ clear.	Panel Length.
10′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	10′
11′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N up.	11′
12′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	12′
13′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	13′
14′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up.	14′

Panel Length.	Roadway, 12' clear.	Roadway, 14' clear	Roadway, 16' clear.	Roadway, 18' clear.	Roadway, 20' clear.	Roadway, 22' clear.	Roadway, 24' clear.	Panel Length.
11'	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	11'
12'	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	12'
13'	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	13'
14'	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	14'
15'	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	15'
16'	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	16'
17'	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	17'
18'	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	18'
19'	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	19'
20'	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{8}''$ ⊙ Up. $1''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	20'
21'	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $1''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{8}''$ ⊙ Up. $1''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	21'
22'	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{8}''$ ⊙ Up. $1''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{3}{16}''$ ⊙ Up. $1\frac{1}{16}''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	22'
23'	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{8}''$ ⊙ Up. $1''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{1}{8}''$ ⊙ Up. $1''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{3}{16}''$ ⊙ Up. $1\frac{1}{16}''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	23'
24'	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{8}''$ ⊙ Up. $1''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{3}{16}''$ ⊙ Up. $1\frac{1}{16}''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	$1\frac{1}{4}''$ ⊙ Up. $1\frac{1}{8}''$ □ Up. $1\frac{5}{8}''$ ⊙ N. up.	24'

TABLE XXIV.

TABLE OF BEAM HANGERS.

TABLE XXIV.

TABLE OF BEAM HANGERS.

Panel Length.	Roadway, 12′ clear.	Roadway, 14′ clear.	Roadway, 16′ clear.	Roadway, 18′ clear.	Roadway, 20′ clear.	Roadway, 22′ clear.	Roadway, 24′ clear.	Panel Length.
20′	¾″ □ Up. 1″ · N. up.	¾″ □ Up. 1″ · N. up.	¾″ □ Up. 1¼″ · N. up.	¾″ □ Up. 1¼″ ⊙ N. up.	13/16″ □ Up. 1¼″ ⊙ N. up.	⅞″ □ Up. 1⅜″ ⊙ N. up.	⅞″ □ Up. 1⅜″ ⊙ N. up.	20′
21′	¾″ · Up. ¾″ □ Up. 1″ · N. up.	13/16″ · Up. ¾″ □ Up. 1¼″ · N. up.	⅞″ · Up. ¾″ □ Up. 1¼″ · N. up.	⅞″ · Up. 13/16″ □ Up. 1¼″ · N. up.	13/16″ ⊙ Up. ⅞″ □ Up. 1¼″ ⊙ N. up.	1″ ⊙ Up. ⅞″ □ Up. 1⅜″ ⊙ N. up.	1 1/16″ ⊙ Up. 15/16″ □ Up. 1⅜″ ⊙ N. up.	21′
22′	¾″ · Up. ¾″ □ Up. 1″ · N. up.	13/16″ · Up. ¾″ □ Up. 1¼″ · N. up.	⅞″ · Up. 13/16″ □ Up. 1¼″ · N. up.	13/16″ · Up. 13/16″ □ Up. 1¼″ ⊙ N. up.	1″ · Up. ⅞″ □ Up. 1⅜″ · N. up.	1″ ⊙ Up. 15/16″ □ Up. 1⅜″ · N. up.	1 1/16″ ⊙ Up. 15/16″ □ Up. 1⅜″ ⊙ N. up.	22′
23′	13/16″ · Up. ¾″ □ Up. 1¼″ ⊙ N. up.	⅞″ · Up. ¾″ □ Up. 1¼″ ⊙ N. up.	⅞″ ⊙ Up. 13/16″ □ Up. 1¼″ · N. up.	13/16″ · Up. ⅞″ □ Up. 1¼″ · N. up.	1″ ⊙ Up. ⅞″ □ Up. 1⅜″ ⊙ N. up.	1 1/16″ ⊙ Up. 15/16″ □ Up. 1⅜″ ⊙ N. up.	1⅛″ ⊙ Up. 1″ □ Up. 1½″ ⊙ N. up.	
24′	13/16″ · Up. ¾″ □ Up. 1¼″ · N. up.	⅞″ · Up. ¾″ □ Up. 1¼″ ⊙ N. up.	13/16″ ⊙ Up. 13/16″ □ Up. 1¼″ · N. up.	1″ ⊙ Up. ⅞″ □ Up. 1⅜″ · N. up.	1 1/16″ · Up. 15/16″ □ Up. 1⅜″ · N. up.	1 1/16″ · Up. 1 15/16″ □ Up. 1⅜″ ⊙ N. up.	1⅛″ · Up. 1″ □ Up. 1½″ ⊙ N. up.	24′

TABLE XXIV.

TABLE OF BEAM HANGERS.

CLASS C.

Four hangers per beam. Up. denotes ends upset; N. up., ends not upset.

Panel Length.	Roadway, 12′ clear.	Roadway, 14′ clear.	Roadway, 16′ clear.	Roadway, 18′ clear	Roadway, 20′ clear.	Roadway, 22′ clear.	Roadway, 24′ clear.	Panel Length.
10′	[illegible] ⊙ Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ ⊙ N. up.	3/4″ ⊙ Up. 3/4″ □ Up. 1″ ⊙ N. up.	10′
11′	3/4″ ⊙ Up. 3/4″ □ Up. 1″ ⊙ N. up	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ ⊙ N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ ⊙ Up. 3/4″ □ Up. 1″ ⊙ N. up.	3/4″ · Up. 3/4″ □ Up. 1″ ⊙ N. up.	11′
12′	3/4″ ⊙ Up. 3/4″ □ Up. 1″ ⊙ N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ ⊙ N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	13/16″ ⊙ Up. 3/4″ □ Up. 1 1/8″ ⊙ N. up.	12′
13′	3/4″ ⊙ Up. 3/4″ □ Up. 1″ ⊙ N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	3/4″ · Up. 3/4″ □ Up. 1″ · N. up.	13/16″ · Up. 3/4″ □ Up. 1 1/8″ · N. up.	13/16″ ⊙ Up. 3/4″ □ Up. 1 1/8″ · N. up.	13′
14′	3/4″ ⊙ Up. 3/4″ □ Up.	3/4″ · Up. 3/4″ □ Up.	3/4″ · Up. 3/4″ □ Up.	3/4″ ⊙ Up. 3/4″ □ Up.	3/4″ ⊙ Up. 3/4″ □ Up.	13/16″ ⊙ Up. 3/4″ □ Up.	3/4″ ⊙ Up. 3/4″ □ Up.	14′

Panel Length.	Roadway, 12′ clear.	Roadway, 14′ clear.	Roadway, 16′ clear.	Roadway, 18′ clear.	Roadway, 20′ clear.	Roadway, 22′ clear.	Roadway, 24′ clear.	Panel Length.
11′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	11′
12′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	12′
13′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. [illegible] Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	13′
14′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	14′
15′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	15′
16′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	16′
17′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	17′
18′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	18′
19′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	19′
20′	$\frac{3}{4}''$ ⊙ Up. [illegible] Up. $1''$ ⊙ N. up.	$\frac{3}{4}''$ ⊙ Up. [illegible] Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. [illegible] Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	20′
21′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	21′
22′	$\frac{3}{4}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1''$ ⊙ N. up.	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	22′
23′	[illegible] ⊙ Up. [illegible] □ Up. [illegible] ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{8}''$ ⊙ Up. $1''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	23′
24′	$\frac{13}{16}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{8}''$ ⊙ N. up.	$\frac{7}{8}''$ ⊙ Up. $\frac{3}{4}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$\frac{15}{16}''$ ⊙ Up. $\frac{13}{16}''$ □ Up. $1\frac{1}{4}''$ ⊙ N. up.	$1''$ ⊙ Up. $\frac{7}{8}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{16}''$ ⊙ Up. $1\frac{15}{16}''$ □ Up. $1\frac{3}{8}''$ ⊙ N. up.	$1\frac{1}{8}''$ ⊙ Up. $1''$ □ Up. $1\frac{1}{2}''$ ⊙ N. up.	24′
Panel Length.	Roadway, 12′ clear.	Roadway, 14′ clear.	Roadway, 16′ clear.	Roadway, 18′ clear.	Roadway, 20′ clear.	Roadway, 22′ clear.	Roadway, 24′ clear.	Panel Length.

6.

6. PANEL 7

Span in Feet.	No. of Panels.	Intensity of Wind Pressure.	12' Clear Roadway. Panel 1.	Panel 2.	Panel 3.	Panel 4.	14' Clear Roadway. Panel 1.	Panel 2.	Panel 3.	Panel 4.	Panel 5.	16' Clear Roadway. Panel 1.	Panel 2.	Panel 3.	Panel 4.	Panel 5.	Panel 6.	Panel 7.	Panel 1.
40	3	40	¾″ ⊙	¾″ ⊙			¾″ ⊙	¾″ ⊙				¾″ ⊙	[illegible]						[illegible]
50	4	40	¾″ ⊙	¾″ ⊙			¾″ ⊙	¾″ ⊙				1⅛″ ⊙	[illegible]						[illegible]
60	5	40	1″ ⊙	1″ ⊙	¾″ ⊙		1⅛″ ⊙	¾″ ⊙	¾″ ⊙			1⅛″ ⊙	[illegible]	¾″ ⊙					[illegible]
70	5	40	2 4″ 6# [1¼″ ⊙	2 4″ 6# [¾″ ⊙ ¾″ ⊙	¾″ ⊙ ¾″ ⊙		2 4″ 6# [1⅛″ ⊙	2 4″ 6# [1⅛″ ⊙ 1⅛″ ⊙	¾″ ⊙ ¾″ ⊙			2 4″ 6# [1⅛″ ⊙	[illegible] 6# [	¾″ ⊙ ¾″ ⊙					2 4″ 6# [1″ ⊙
80	5	40	2 4″ 6# [1¼″ ⊙	2 4″ 6# [¾″ ⊙ 1⅛″ ⊙	¾″ ⊙ ¾″ ⊙		2 4″ 6# [1⅛″ ⊙	2 4″ 6# [⅞″ ⊙ 1⅛″ ⊙	¾″ ⊙ ¾″ ⊙			2 4″ 6# [1¼″ ⊙	[illegible] 6# [	¾″ ⊙ ¾″ ⊙					2 4″ 6.5# 1¼″ ⊙
90	5	40	2 4″ 6# [1¼″ ⊙	2 4″ 6# [1″ ⊙ 1⅛″ ⊙	¾″ ⊙ ¾″ ⊙		2 5″ 7# [1⅛″ ⊙	2 4″ 6# [1″ ⊙ 1″ ⊙	¾″ ⊙ ¾″ ⊙			2 5″ 7# [1¼″ ⊙	[illegible] # [	¾″ ⊙ ¾″ ⊙					2 5″ 7.2# 1¼″ ⊙
100	5	40	2 5″ 7# [1¼″ ⊙	2 4″ 6# [1″ ⊙ 1⅛″ ⊙	¾″ ⊙ 1⅛″ ⊙		2 5″ 7.5# [1⅛″ ⊙	2 4″ 6# [1″ ⊙ 1⅛″ ⊙	¾″ ⊙ 1⅛″ ⊙			2 5″ 8# [1¼″ ⊙	[illegible] # [	¾″ ⊙ ¾″ ⊙					2 6″ 8.5# 1⅛″ ⊙
110	6	35	2 5″ 7# [1¼″ ⊙	2 4″ 6# [1⅛″ ⊙ 1⅛″ ⊙	2 4″ 6# [⅞″ ⊙ ⅞″ ⊙		2 5″ 7.8# [1⅛″ ⊙	2 4″ 6# [1⅛″ ⊙ 1⅛″ ⊙	2 4″ 6# [⅞″ ⊙ ¾″ ⊙			2 6″ 8.5# [1¾″ ⊙	[illegible] 6# [	2 4″ 6# [1⅛″ ⊙ ¾″ ⊙					2 6″ 9.5# 1⅛″ ⊙
120	6	35	2 5″ 7.8# [1⅛″ ⊙	2 4″ 6# [1¼″ ⊙ 1⅛″ ⊙	2 4″ 6# [¾″ ⊙ 1″ ⊙		2 6″ 8.5# [1¼″ ⊙	2 4″ 6# [1¼″ ⊙ 1¼″ ⊙	2 4″ 6# [⅞″ ⊙ 1⅛″ ⊙			2 6″ 9# [1⅞″ ⊙	2 [illegible] 6# [	2 4″ 6# [¾″ ⊙ 1⅛″ ⊙					2 7″ 10.5# 1¼″ ⊙
130	7	35	2 6″ 8.5# [1¾″ ⊙	2 4″ 6# [1¼″ ⊙ 1¼″ ⊙	2 4″ 6# [1″ ⊙ 1¼″ ⊙	¾″ ⊙	2 6″ 8.8# [1½″ ⊙	2 4″ 6# [1¼″ ⊙ 1⅛″ ⊙	2 4″ 6# [1″ ⊙ 1⅛″ ⊙	¾″ ⊙ 1⅛″ ⊙		2 6″ 9.5# [1½″ ⊙	[illegible] 6# [	2 4″ 6# [1″ ⊙ 1⅛″ ⊙	¾″ ⊙ 1⅛″ ⊙				2 7″ 10.5# 1⅛″ ⊙
140	7	35	2 6″ 8.5# [1⅞″ ⊙	2 4″ 6# [1⅛″ ⊙ 1⅛″ ⊙	2 4″ 6# [1⅛″ ⊙ 1⅛″ ⊙	1⅛″ ⊙	2 6″ 9.7# [1⅜″ ⊙	2 4″ 6# [1⅛″ ⊙ 1⅛″ ⊙	2 4″ 6# [1″ ⊙ 1⅛″ ⊙	1⅛″ ⊙ ⅞″ ⊙		2 7″ 10.5# [1⅛″ ⊙	[illegible] 6# [	2 4″ 6# [1″ ⊙ 1¼″ ⊙	[illegible]				2 7″ 11.5# 1½″ ⊙
150	8	35					2 7″ 10.5# [1⅞″ ⊙	2 4″ 6.5# [1⅛″ ⊙ 1⅛″ ⊙	2 4″ 6# [1⅛″ ⊙ 1¼″ ⊙	2 4″ 6# [1⅛″ ⊙ 1″ ⊙		2 7″ 11.5# [1⅝″ ⊙	[illegible] # [	2 4″ 6# [1⅛″ ⊙ 1⅛″ ⊙	2 4″ 6# [¾″ ⊙ 1″ ⊙				2 8″ 12.5# 1⅝″ ⊙
160	8	30					2 4″ 6# [2 1¼″ ⊙ 2 4″ 6# [1⅛″ ⊙	2 4″ 6# [1¼″ ⊙ ⅞″ ⊙ 4″ 8# I	2 4″ 6# [1¼″ ⊙ ⅞″ ⊙ 4″ 8# I	2 4″ 6# [1⅛″ ⊙ ⅞″ ⊙ 4″ 8# I		2 4″ 6.8# [2 1¼″ ⊙ 2 4″ 6# [	[illegible] # [# I	2 4″ 6# [1⅛″ ⊙ 4″ 8# I	2 4″ 6# [4″ 8# I				2 5″ 7# [2 1¼″ ⊙ 2 4″ 6# [
170	9	30					2 4″ 6# [2 1¼″ ⊙ 2 4″ 6# [1⅛″ ⊙	2 4″ 6# [1⅛″ ⊙ 4″ 8# I	2 4″ 6# [1⅛″ ⊙ 4″ 8# I	2 4″ 6# [1⅛″ ⊙ 4″ 8# I 1″ ⊙	¾″ ⊙ ¾″ ⊙	2 4″ 6.8# [2 1″ ⊙ 2 4″ 6.1# [	[illegible] # [# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	¾″ ⊙			2 5″ 7# [2 1¼″ ⊙ 2 4″ 6.8# [
180	9	30					2 4″ 6# [2 1¼″ ⊙ 2 4″ 6# [1¼″ ⊙	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	¾″ ⊙ 1⅛″ ⊙	2 4″ 6.0# [2 1¼″ ⊙ 2 4″ 6# [	[illegible] # [# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	¾″ ⊙ 1⅛″ ⊙			2 5″ 7# [2 1¼″ ⊙ 2 4″ 6.5# [
190	10	30					2 4″ 6# [2 1¼″ ⊙ 2 4″ 6# [1¼″ ⊙	2 4″ 6# [4″ 8# I 1″ ⊙	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	2 4″ 7# [2 1⅛″ ⊙ 2 4″ 6# [	[illegible] # [# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I			2 5″ 7# [2 1¼″ ⊙ 2 4″ 6.7# [
200	10	30										2 5″ 7# [2 1¼″ ⊙ 2 4″ 6# [	[illegible] # [# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I			2 5″ 7# [2 1¼″ ⊙ [illegible]
210	11	30										2 5″ 7# [2 1¼″ ⊙ 2 4″ 6# [	[illegible] # [# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	[illegible]		2 5″ 7# [2 5″ 7# [
220	11	30										2 5″ 7# [2 1⅛″ ⊙ 2 4″ 6.2# [	[illegible] # [4″ 8# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	[illegible]		2 5″ 7.6# [illegible] # [
230	12	30										2 5″ 7.3# [2 1⅛″ ⊙ 2 4″ 6.4# [	[illegible] # [# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I		[illegible] # [
240	12	30										2 5″ 7.6# [2 1¼″ ⊙ 2 4″ 6.6# [	[illegible] # [# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	2 4″ 6# [4″ 8# I	[illegible] # [4″ 8# I		[illegible] # [
250	13	30										2 5″ 8# [2 1¼″ ⊙ 2 4″ 6.7# [	[illegible] # [# I	[illegible] # [4″ 8# I	[illegible] # [4″ 8# I	[illegible] # [4″ 8# I	[illegible] # [4″ 8# I	[illegible]	[illegible] # [
260	13	30										2 5″ 8.3# [2 1¼″ ⊙ 2 5″ [illegible] # [	[illegible] # [# I	[illegible] # [4″ 8# I	[illegible] # [4″ 8# I	[illegible] # [4″ 8# I	[illegible] # [4″ 8# I	[illegible]	[illegible] # [
270	14	30																	[illegible] # [
280	14	30																	[illegible] # [
290	15	30																	[illegible] # [
300	15	30																	[illegible] # [

TABLE XXV.

TABLE OF LATERAL SYSTEMS AND SWAY BRACING

[illegible] Panel 5.	Panel 6.	Panel 7.	18' Clear Roadway. Panel 1.	Panel 2.	Panel 3.	Panel 4.	Panel 5.	Panel 6.	Panel 7.	Panel 8.	20' Clear Roadway. Panel 1.	Panel 2.	Panel 3.	Panel 4.	Panel 5.	Panel 6.	Panel 7.	Panel 8.	Panel 1.	Panel 2.
			2 4" 1# [	[illegible] 1# [							2 4" 1# [	2 4" 6# [							2 4" 6.5# [	2 4" 6# [
			2 4" 6.5# [	[illegible] 1# [							2 5" 7# [	2 4" 6# [							[illegible] # [	2 4" 6# [
			2 5" 7.5# [	[illegible] 6# [							2 5" 8# [	2 4" 6# [							2 6" 8.5# [	2 4" 6# [
			2 6" 8.5# [	2 4" 1# [							2 6" 9# [	2 4" 6# [							2 6" 9.5# [	2 4" 6# [
			2 6" 9.5# [	[illegible] 1# [	2 4" 6# [						2 7" 10.5# [	2 4" 6# [	2 4" 6# [						2 7" 10.5# [	2 4" 6# [
			2 7" 10.5# [	[illegible] 1# [	2 4" 1# [						2 7" 10.5# [	2 4" 6# [	2 4" 6# [						2 7" 11# [	2 4" 6# [
			2 7" 10.5# [	[illegible] 5 # [	2 4" 6# [						2 7" 11.5# [	2 5" 7# [	2 4" 6# [						2 7" 12.5# [	2 5" 7# [
			2 7" 11.5# [	[illegible] 5 # [	2 4" 6# [						2 8" 12.5# [	2 5" 7# [	2 4" 6# [						2 8" 12.5# [	2 5" 7# [
			2 8" 12.5# [	[illegible] 5 # [	2 4" 6# [	2 4" 1# [					2 8" 12.5# [	2 5" 7# [	2 4" 6# [	2 4" 6# [					2 8" 13# [	2 5" 7# [
			2 5" 7# [	[illegible] 4" 6# [	2 4" 1# [	2 4" 6# [					2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [					2 5" 7.5# [	2 4" 6# [
			2 4" 6# [	4" 8# I	4" 8# I	4" 8# I					2 4" 6.5# [	5" 10# I	5" 10# I	5" 10# I					2 5" 7# [	5" 10# I
			2 5" 7# [	[illegible] 4" 6# [	2 4" 6# [	2 4" 6# [					2 5" 7.5# [	2 4" 6# [	2 4" 6# [	2 4" 6# [					2 5" 8# [	2 4" 6.5# [
			2 4" 6.5# [	4" 8# I	4" 8# I	4" 8# I					2 5" 7# [	4" 8# I	4" 8# I	4" 8# I					2 5" 7# [	5" 10# I
			2 5" 7# [	[illegible] 4" 6# [	2 4" 6# [	2 4" 6# [					2 5" 7# [	2 4" 6.5# [	2 4" 6# [	2 4" 6# [					2 5" 7.5# [	2 4" 6.5# [
			2 4" 6.5# [	4" 8# I	4" 8# I	4" 8# I					2 5" 7# [	4" 8# I	4" 8# I	4" 8# I					2 5" 8# [	4" 8# I
[illegible] 4" 6# [			2 5" 7# [	[illegible] 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				2 5" 7.5# [	2 4" 6.5# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				2 5" 7.5# [	2 5" 7# [
[illegible] 8# I			2 4" 6.5# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 7.4# [	5" 10# I
[illegible] 6# [			2 5" 7# [	[illegible] 4" 6# [	4" 6# [	2 4" 6# [	4" 6# [				2 5" 7.5# [	2 4" 6.5# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				2 5" 8.2# [	2 5" 7# [
[illegible] 8# I			2 4" 6.5# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 7.5# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 7.8# [	5" 10# I
[illegible] 4" 6# [			2 5" 7.5# [	[illegible] 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				2 5" 8# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				2 6" 8.5# [	2 5" 7# [
[illegible] 8# I			2 5" 7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 7.5# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 8# [	5" 10# I
[illegible] 6# [			2 5" 7.5# [	[illegible] # [	2 4" 6# [	2 4" 6# [	2 4" 1# [				2 5" 8.4# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				2 6" 8.5# [	2 5" 7# [
[illegible] 8# I			[illegible] # [	[illegible] 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 7.4# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 8# [	5" 10# I
[illegible] 6# [	2 4" 6# [		2 5" 8# [	[illegible] # [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			2 6" 8.5# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			2 6" 8.5# [	2 5" 7# [
[illegible] 8# I	4" 8# I		[illegible] # [	[illegible] # I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			2 5" 7.4# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			2 5" 8.5# [	5" 10# I
[illegible] 6# [	[illegible] # [		[illegible] 8# [	[illegible] # [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			2 6" 8.5# [	2 5" 7# [	2 4" 6.5# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			2 6" 8.7# [	2 5" 7# [
[illegible] 8# I	[illegible] # I		[illegible] # [	[illegible] # I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			2 5" 7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			2 5" 8.5# [	5" 10# I
[illegible] 6# [	[illegible] # [		[illegible] # [	[illegible] # [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			2 6" 8.6# [	2 5" 7# [	2 4" 6.5# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			2 6" 9.1# [	2 5" 7.1# [
[illegible] 8# I	[illegible] # I		[illegible] # [	[illegible] # I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			2 5" 7.9# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			2 6" 8.6# [	5" 10# I
[illegible] 6# [	[illegible] # [		[illegible] # [	[illegible] # [	2 4" 6# [	2 4" 6# [	2 4" 6# [	[illegible] 6# [			2 6" 9# [	2 5" 7# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			2 6" 9.5# [	2 5" 7.1# [
[illegible] 8# I	[illegible] # I		[illegible] # [	[illegible] # I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			2 5" 7.9# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			2 6" 8.5# [	5" 10# I
			[illegible] # [	[illegible] # [	2 4" 6# [	2 4" 6# [	[illegible] 6# [	2 4" 6# [	2 4" 6# [		2 6" 9.5# [	2 5" 7# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [		2 6" 10# [	2 5" 7.1# [
			[illegible]	[illegible] # I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		2 5" 8# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		2 6" 8.8# [	5" 10# I
			[illegible] # [	[illegible] # [	2 4" 6# [	2 4" 6# [	[illegible] 6# [	[illegible] 6# [	[illegible] 6# [		2 6" 10# [	2 5" 7# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [		2 6" 10.5# [	2 5" 7.8# [
			[illegible] 6# [	[illegible] # I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		2 5" 8# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		2 6" 8.8# [	5" 10# I
			2 6" 9# [	2 4" 6# [	2 4" 6# [	[illegible] 6# [	[illegible] 6# [	2 4" 6# [	2 4" 6# [		2 6" 10.5# [	2 5" 7# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [		2 7" 11.5# [	2 5" 7.1# [
			2 5" 8# [	[illegible] # I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		2 5" 8# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		2 6" 8.8# [	[illegible] # I
			[illegible] 6# [	[illegible] # [	2 4" 6# [	[illegible] 6# [	[illegible] 6# [	[illegible] 6# [	2 4" 6# [		2 7" 10.5# [	2 5" 7# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [		2 7" 10.5# [	[illegible]
			2 5" 8# [	[illegible] # [	4" 8# I	4" 8# I	4" 8# I	4" 8# [	4" 8# I		2 6" 8.5# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		2 6" 8.8# [	5" 1 # I

TABLE XXV.

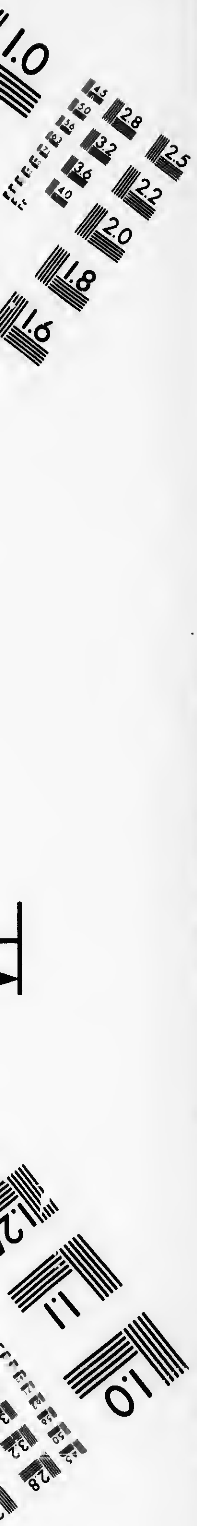
TABLE OF LATERAL SYSTEMS AND SWAY BRACING

18' Clear Roadway								20' Clear Roadway							
Panel 1.	Panel 2.	Panel 3.	Panel 4.	Panel 5.	Panel 6.	Panel 7.	Panel 8.	Panel 1.	Panel 2.	Panel 3.	Panel 4.	Panel 5.	Panel 6.	Panel 7.	Panel 8.
2 4" 1# [	2 4" 6# [							2 4" 6# [	2 4" 6# [						
2 4" 6.5# [	2 4" 6# [							2 5" 7# [	2 4" 6# [						
2 5" 7.2# [	2 4" 6# [							2 5" 8# [	2 4" 6# [						
2 6" 8.5# [	2 4" 6# [							2 6" 6# [	2 4" 6# [						
2 6" 9.5# [	2 4" 6# [	2 4" 6# [						2 7" 10.5# [	2 4" 6# [	2 4" 6# [					
2 7" 10.5# [	2 4" 6# [	2 4" 6# [						2 7" 10.5# [	2 4" 6# [	2 4" 6# [					
2 7" 10.5# [	2 5" 7# [	2 4" 6# [						2 7" 11.2# [	2 5" 7# [	2 4" 6# [					
2 7" 11.5# [	2 5" 7# [	2 4" 6# [						2 8" 12.5# [	2 5" 7# [	2 4" 6# [					
2 8" 12.5# [	2 5" 7# [	2 4" 6# [	2 4" 6# [					2 8" 12.5# [	2 5" 7# [	2 4" 6# [	2 4" 6# [				
2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [					2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				
2 4" 6# [	4" 8# I	4" 8# I	4" 8# I					2 4" 6.5# [	5" 10# I	5" 10# I	5" 10# I				
2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [					2 5" 7.5# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				
2 4" 6.8# [	4" 8# I	4" 8# I	4" 8# I					2 5" 7# [	4" 8# I	4" 8# I	4" 8# I				
2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [					2 5" 7# [	2 4" 6.5# [	2 4" 6# [	2 4" 6# [				
2 4" 6.5# [	4" 8# I	4" 8# I	4" 8# I					2 5" 7# [	4" 8# I	4" 8# I	4" 8# I				
2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				2 5" 7.2# [	2 4" 6.5# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			
2 4" 6.7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I			
2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				2 5" 7.6# [	2 4" 6.5# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			
2 4" 6.9# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 7.2# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I			
2 5" 7.2# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				2 5" 8# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			
2 5" 7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 7.3# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I			
2 5" 7.6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				2 5" 8.4# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			
2 5" 7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I				2 5" 7.4# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I			
2 5" 8# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			2 6" 8.5# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [		
2 5" 7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			2 5" 7.4# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		
2 5" 8.5# [	2 4" 6.4# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			2 6" 8.5# [	2 5" 7# [	2 4" 6.5# [	2 4" 6# [	2 4" 6# [	2 4" 6# [		
2 5" 7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			[illegible]	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		
2 6" 8.5# [	2 4" 6.4# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			[illegible] 8# [	2 5" 7# [	[illegible]	2 4" 6# [	2 4" 6# [	2 4" 6# [		
2 5" 7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			2 5" 7.9# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		
2 6" 8.5# [	2 4" 6.8# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [			2 6" 9# [	2 5" 7# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [		
2 5" 7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I			[illegible]	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		
[illegible]	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [		[illegible] 9.5# [	2 5" 7# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	
2 5" 7.3# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		[illegible]	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	
[illegible]	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [		[illegible]	2 5" 7# [	[illegible]	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	
2 5" 7.6# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		[illegible]	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	
2 6" 9.5# [	2 4" 6.8# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [		[illegible] 9.5# [	2 5" 7# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	
2 5" 7.7# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		[illegible] 8.5# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	
2 6" 10# [	2 4" 6.8# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [		[illegible] 10.5# [	[illegible]	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	2 4" 6# [	
2 5" 8# [	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I		[illegible]	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	4" 8# I	

Panel 7	Panel 8	22' Clear Roadway. Panel 1.	Panel 2.	Panel 3.	Panel 4.	Panel 5.	Panel 6.	Panel 7.	Panel 8.	24' Clear Roadway. Panel 1.	Panel 2.	Panel 3.	Panel 4.	Panel 5.	Panel 6.	Panel 7.	Panel 8.	Span in Feet.
		[illegible]	[illegible]							[illegible]	[illegible]							40
		[illegible]	[illegible]							1¼" ⊙	[illegible]							50
		¾"	[illegible]	1"						¾" ⊙	1"	1" ⊙						60
		2 4" 6.8# [	2 4" 6# [							2 5" 7# [	2 4" 6# [							
		1"	[illegible]	[illegible]						1" ⊙	[illegible]	1" ⊙						70
		[illegible] # [	2 4" 6# [							5" [illegible] # [	2 4" 6# [							
		1⅛"	[illegible]	[illegible]						1⅛"	[illegible]	[illegible]						80
		2 6" 8.5# [	2 4" 6# [							2 6" 6# [	[illegible] 6# [							
		1⅛"	[illegible]	[illegible]							[illegible]	[illegible]						90
		2 6" 6.5# [	2 4" 6# [							[illegible] # [	2 4" 6# [							
		1¼"	[illegible]	[illegible]						1¼	[illegible]	[illegible]						100
		2 7" 10.5# [	2 4" 6# [	2 4" 6# [						2 7" 10.5# [	2 4" 6.3# [	2 4" 6# [						
		1¼"	1⅛"	1⅛"						1¼"	1"	1⅛"						110
		[illegible] 11# [	2 4" 6# [	2 4" 6# [						2 7" 11.4# [	[illegible] # [	2 4" 6# [						
		[illegible]	1⅛"	1⅛"						1⅜"	[illegible]	[illegible]						120
		[illegible] 1.5# [	2 5" 7# [	2 4" 1# [						2 8" 12.5# [	2 5 [illegible] # [	2 4" 6.5# [						
		1"	[illegible]	[illegible]						1⅜" ⊙	[illegible]	[illegible]	¾" ⊙					130
		[illegible] 12.[illegible]# [	2 5" 7# [	2 4" 6# [						2 8" 13# [	2 5" 7.5# [	2 4" 6.5# [						
		[illegible]	[illegible]	[illegible]	[illegible]					[illegible]	[illegible]	[illegible]	[illegible]					140
		2 8" 1.1# [	2 5" 7# [	2 5" 7# [	2 4" 6# [					2 8" 13.5# [	2 5" 7.8# [	2 5" 7# [	2 4" 6# [					
		[illegible]	[illegible]	[illegible]	[illegible]					1¼"	[illegible]	[illegible]	[illegible]					150
		2 5" 7.5# [	2 4" 6# [	2 4" 6# [	2 4" 6# [					2 5" 8# [	2 4" 6.4# [	2 4" 6# [	2 4" 6# [					
		2 5" 7# [	5" 10# I	5" 10# I	5" 10# I					2 5" 7# [	5" 10# I	5" 10# I	5" 10# I					160
		2 5" 8# [	2 4" 6.2# [	2 4" 6# [	2 4" 6# [					2 6" 8.5# [	2 4" 6.5# [	2 4" 6# [	2 4" 6# [	1⅛" ⊙				
		2 5" 7# [	5" 10# I	5" 10# I	5" 10# I					2 5" 7.8# [	5" 10# I	5" 10# I	5" 10# I	1⅛" ⊙				170
		2 5" 7.2# [	2 4" 6.8# [	2 4" 6# [	2 4" 6# [					2 5" 8.4# [	2 5" [illegible] # [	[illegible] 6# [	2 4" 1# [	1⅛" ⊙				
		2 5" 7# [	[illegible] 8# I	[illegible] 8# I	[illegible] 8# I					2 5" 7.1# [	5" 10# I	5" 10# I	5" 10# I	1⅛" ⊙				180
		2 5" 7.7# [	2 5" [illegible] # [	2 4" 6# [	2 4" 6# [	2 4" [illegible] # [				2 6" 8.5# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	2 4" 6# [				
		2 5" 7.1# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I				2 5" 7.9# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I				190
		2 5" 8.2# [	2 5" [illegible] # [	2 4" 6# [	2 4" 6# [	2 4" 6# [				2 6" 8.5# [	2 5" 7# [	2 4" 6.4# [	2 4" 6# [	2 4" 6# [				
		2 5" 7.5# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I				2 5" 8.3# [	5" 10# I	5" 10# I	5" 10# I	[illegible] # I				200
		2 6" 8.5# [	2 5" 7# [	2 4" 6.5# [	2 4" 6# [	2 4" 1# [	¾"			2 6" 8.5# [	2 5" 7# [	2 5" 7# [	2 4" 6# [	2 4" [illegible] # [	¾" ⊙			
		2 5" 8# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	1⅛"			2 6" 8.5# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	¾" ⊙			210
		2 6" 8.5# [	2 5" 7# [	2 4" 6.8# [	2 4" 6# [	2 4" 6# [	¾"			2 6" 8.5# [	2 5" 7.3# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	¾" ⊙			
		2 5" 8# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	1⅛"			2 6" 8.5# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	1⅛" ⊙			220
		2 6" 8.5# [	2 5" [illegible] # [	[illegible] 6.5# [	2 4" 6# [	2 4" 6# [	2 4" [illegible] # [			2 6" 8.0# [	2 5" 7.3# [	2 5" 7# [	2 4" 6.3# [	2 4" 6# [	2 4" 1# [			
		2 5" 8# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	5" 10# I			2 6" 8.5# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	5" 10# I			230
		2 6" 8.7# [	2 5" [illegible] # [	2 5" [illegible] # [	2 4" 6# [	2 4" 6# [	2 4" 6# [			2 6" 9# [	2 5" 7.3# [	2 5" 7# [	2 4" 6.3# [	2 4" 1# [	2 4" 6# [			
		2 5" 8.3# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	5" 10# I			2 6" 8.5# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	5" 10# I			240
		2 6" 9.1# [	2 5" [illegible] # [	2 5" 7# [	2 4" [illegible] # [	2 4" 6# [	2 4" 6# [			2 6" 9.5# [	2 5" 7.6# [	2 5" 7# [	2 5" 7# [	2 4" 6# [	2 4" 6# [	¾"		
		2 6" 8.5# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	5" 10# I	1"		2 6" 8.5# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	5" 10# I			250
		2 6" 9.5# [	[illegible] # [	2 5" [illegible] # [	2 4" 6.[illegible] # [	2 4" 6# [	2 4" 6# [	¾"		2 6" 10# [	2 6" 8.5# [	2 5" 7# [	2 5" 7# [	2 4" 6.3# [	2 4" [illegible] # [			
		2 6" 8.5# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	5" 10# I	1"		2 6" 8.5# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	5" 10# I			260
[illegible] 6# [		2 6" 10# [	2 5" 7.[illegible] # [	2 5" 7# [	2 4" 6.0# [	2 4" 6# [	2 4" 6# [	2 4" 1# [		2 6" 10.5# [	2 6" 8.5# [	2 5" [illegible] # [	2 5" [illegible] # [	[illegible] # [	2 4" 6# [	2 4" 1# [		
[illegible] 8# I		2 6" 8.5# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	5" 10# I	5" 10# I		2 6" 8.5# [	5" 10# I	5" 10# I	5" 10# I	5" 10# I	[illegible] # I	5" 10# I		270
[illegible] 6# [		[illegible] # [	2 5" 7.6# [	2 5" [illegible] # [	2 4" 6.8# [	2 4" 6# [	2 4" 6# [	2 4" 1# [		2 7" 10.5# [	2 6" 8.5# [	2 5" [illegible] # [	2 5" [illegible] # [	2 4" [illegible] # [	[illegible] # [	[illegible] # [		
[illegible] 8# I		2 6" 8.[illegible] # [	5" [illegible] # I	5" 10# I	5" 10# I	5" 10# I	5" [illegible] I	5" 10# I		2 6" 8.0# [	5" 10# I	5" [illegible] # I	5" 10# I	5" 10# I	[illegible] # I	5" 10# I		280
[illegible] 6# [	¾"	[illegible] # [	2 5" [illegible] # [	[illegible] # [	2 4" [illegible] # [	2 4" [illegible] # [	[illegible]	[illegible] # [	¾"	2 7" 10.5# [	2 6" 8.5# [	2 5" 7.5# [	2 5" 7# [	2 5 [illegible] # [	[illegible]	[illegible] # [	¾"	
[illegible] 8# I	1⅛"	2 6" [illegible] # [	[illegible] # I	5" [illegible] # I	5" [illegible] # I	5" [illegible] # I	[illegible] I	5" [illegible] # I	1⅛"	2 6" 9.1# [	5" [illegible] # I	[illegible] # I	5" 10# I	5" 10# I	5" [illegible] # I	5" [illegible] # I	1⅛"	290
[illegible] 6# [	¾"	2 7" 10.5# [	[illegible] # [	[illegible] # [	[illegible] # [	[illegible] # [		[illegible] # [		2 7" 10.5# [	[illegible] # [	2 5" 7# [	2 5" [illegible] # [	2 5 [illegible]	[illegible] # [	[illegible] # [		
[illegible] 8# I	1⅛"	2 6" 8.5# [	5" [illegible] # I	5" [illegible] # I	5" [illegible] I	5" [illegible]	[illegible] I	5" [illegible] I	1⅛"	2 6" 9.3# [	5" 10# I	5" [illegible] # I	5" 10# I	5" [illegible] I	[illegible] # I	[illegible] # I		300

IMAGE EVALUATION
TEST TARGET (MT-3)

6"



SA

Havor
ertic

3⅜"
10.1
11.4
12.6
13.9
15.1
16.4
17.7
18.9
20.2
21.5
22.7
24.0
25.3
26.9
27.8
29.1
30.3
32.9
35.4
37.9
40.
43.0
45.
48.
50.

TA

SAW

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Upper horizontal line shows diameter of pin.

3⅜″		5⅜″	5½″	5⅝″	5¾″	5⅞″	6″	
10.13	0							½″
11.40	1							9⁄16″
12.66	3							⅝″
13.92	4							11⁄16″
15.19	5							¾″
16.45	6	26.20						13⁄16″
17.72	8	28.22	28.88	29.53				⅞″
18.98	9	30.23	30.94	31.64	32.34			15⁄16″
20.25	0	32.25	33.00	33.75	34.50	35.25	36.00	1″
21.52	1	34.27	35.01	35.86	36.66	37.45	38.25	1 1⁄16″
22.78	3	36.28	37.13	37.97	38.81	39.66	40.50	1⅛″
24.05	4	38.30	39.19	40.08	40.97	41.86	42.75	1 3⁄16″
25.31	5	40.31	41.25	42.19	43.13	44.06	45.00	1¼″
26.58	6	42.33	43.31	44.30	45.28	46.27	47.25	1 5⁄16″
27.84	8	44.34	45.38	46.41	47.44	48.47	49.50	1⅜″
29.11	9	46.36	47.44	48.52	49.59	50.67	51.75	1 7⁄16″
30.38	0	48.38	49.50	50.63	51.75	52.88	54.00	1½″
32.91	3	52.41	53.63	54.84	56.06	57.28	58.50	1⅝″
35.44	5	56.44	57.75	59.06	60.38	61.69	63.00	1¾″
37.97	8	60.47	61.88	63.28	64.69	66.09	67.50	1⅞″
40.50	0	64.50	66.00	67.50	69.00	70.50	72.00	2″
43.03	3	68.53	70.13	71.72	73.31	74.91	76.50	2⅛″
45.56	5	72.56	74.25	75.94	77.62	79.31	81.00	2¼″
48.09	8	76.59	78.38	80.16	81.94	83.72	85.50	2⅜″
50.63	0	80.63	82.50	84.38	86.25	88.13	90.00	2½″
	3	84.66	86.63	88.59	90.56	92.53	94.50	2⅝″
		88.69	90.75	92.81	94.87	96.94	99.00	2¾″
		92.72	94.88	97.03	99.19	101.34	103.50	2⅞″
		96.75	99.00	101.25	103.50	105.75	108.00	3″
		100.78	103.13	105.47	107.81	1[illegible]0.16	112.50	3⅛″
		104.81	107.25	109.69	112.12	114.56	11[illegible].00	3¼″
		108.84	111.38	113.91	116.44	118.97	121.50	3⅜″
		112.88	115.50	118.13	[illegible]0.75	123.38	120.00	[illegible]
		116.91	119.63	122.34	125.06	127.78	130.50	[illegible]
			123.75	126.56	129.37	132.19	135.00	3¾″
					133.69	136.59	139.50	3⅞″
							144.00	4″

TABLE XXV

TABLE FOR FINDING THE NECESSARY WIDTH OF BE

Having given the total pressure on said surface, and the diameter of the pin. This table is calculated for a working compressive stress of 6 t
Vertical lines of inches show widths

CLASS A.

	$1\frac{1}{2}''$	$1\frac{5}{8}''$	$1\frac{3}{4}''$	$1\frac{7}{8}''$	$2''$	$2\frac{1}{8}''$	$2\frac{1}{4}''$	$2\frac{3}{8}''$	$2\frac{1}{2}''$	$2\frac{5}{8}''$
$\frac{1}{2}''$	4.50	4.88	5.25	5.63	6.00	6.38	6.75	7.13	7.50	7.88
$\frac{9}{16}''$	5.06	5.48	5.91	6.33	6.75	7.17	7.59	8.02	8.44	8.86
$\frac{5}{8}''$	5.63	6.09	6.57	7.03	7.50	7.97	8.44	8.91	9.37	9.84
$\frac{11}{16}''$	6.19	6.70	7.22	7.73	8.25	8.77	9.28	9.80	10.31	10.83
$\frac{3}{4}''$	6.75	7.31	7.88	8.44	9.00	9.56	10.13	10.69	11.25	11.81
$\frac{13}{16}''$	7.31	7.92	8.53	9.14	9.75	10.36	10.97	11.58	12.18	12.80
$\frac{7}{8}''$	7.88	8.53	9.19	9.84	10.50	11.16	11.81	12.47	13.1[illegible]	13.78
$\frac{15}{16}''$	8.44	9.14	9.84	10.55	11.25	11.95	12.66	13.36	14.06	14.77
$1''$	9.00	9.75	10.50	11.25	12.00	12.75	13.50	14.25	14.99	15.75
$1\frac{1}{16}''$	9.56	10.36	11.16	11.95	12.75	13.55	14.34	15.14	15.93	16.73
$1\frac{1}{8}''$	10.13	10.97	11.81	12.66	13.50	14.34	15.19	16.03	16.87	17.72
$1\frac{3}{16}''$	10.69	11.58	12.47	13.36	14.25	15.14	16.03	16.92	17.80	18.70
$1\frac{1}{4}''$	11.25	12.19	13.13	14.06	15.00	15.94	16.88	17.81	18.74	19.69
$1\frac{5}{16}''$	11.81	12.80	13.78	14.77	15.75	16.73	17.72	18.70	19.67	20.67
$1\frac{3}{8}''$	12.38	13.41	14.44	15.47	16.50	17.53	18.56	19.59	20.61	21.66
$1\frac{7}{16}''$		14.02	15.09	16.17	17.25	18.33	19.41	20.48	21.55	22.64
$1\frac{1}{2}''$				16.88	18.00	19.13	20.25	21.38	22.48	23.63
$1\frac{5}{8}''$						20.52	21.94	23.16	24.36	25.59
$1\frac{3}{4}''$							23.63	24.94	26.23	27.56
$1\frac{7}{8}''$								26.72	28.10	29.53
$2''$									29.97	31.50
$2\frac{1}{8}''$										
$2\frac{1}{4}''$										
$2\frac{3}{8}''$										
$2\frac{1}{2}''$										
$2\frac{5}{8}''$										
$2\frac{3}{4}''$										
$2\frac{7}{8}''$										
$3''$										
$3\frac{1}{8}''$										
$3\frac{1}{4}''$										
$3\frac{3}{8}''$										
$3\frac{1}{2}''$										
$3\frac{5}{8}''$										
$3\frac{3}{4}''$										
$3\frac{7}{8}''$										
$4''$										

	$2\frac{3}{4}''$	$2\frac{7}{8}''$	$3''$	$3\frac{1}{8}''$	$3\frac{1}{4}''$	$3\frac{3}{8}''$	$3\frac{1}{2}''$	$3\frac{5}{8}''$	$3\frac{3}{4}''$	$3\frac{7}{8}''$
$\frac{1}{2}''$	8.25	8.63	9.00	9.38	9.75	10.13	10.50	10.88	11.25	11.63
$\frac{9}{16}''$	9.28	9.71	10.12	10.55	10.97	11.40	11.81	12.23	12.66	13.08
$\frac{5}{8}''$	10.31	10.78	11.25	11.72	12.19	12.66	13.13	13.59	14.06	14.53
$\frac{11}{16}''$	11.34	11.86	12.38	12.89	13.41	13.92	14.44	14.95	15.47	15.98
$\frac{3}{4}''$	12.38	12.94	13.50	14.06	14.63	15.19	15.75	16.31	16.88	17.44
$\frac{13}{16}''$	13.41	14.02	14.63	15.23	15.84	16.45	17.06	17.67	18.28	18.89
$\frac{7}{8}''$	14.44	15.09	15.75	16.41	17.06	17.72	18.38	19.03	19.69	20.34
$\frac{15}{16}''$	15.47	16.17	16.88	17.58	18.28	18.98	19.69	20.39	21.09	21.80
$1''$	16.50	17.25	18.00	18.75	19.50	20.25	21.00	21.75	22.50	23.25
$1\frac{1}{16}''$	17.53	18.33	19.13	19.92	20.72	21.52	22.31	23.11	23.91	24.70
$1\frac{1}{8}''$	18.56	19.41	20.25	21.09	21.94	22.78	23.63	24.47	25.31	26.16
$1\frac{3}{16}''$	19.57	20.48	21.38	22.27	23.16	24.05	24.94	25.83	26.72	27.61
$1\frac{1}{4}''$	20.63	21.56	22.50	23.44	24.38	25.31	26.25	27.19	28.13	29.06
$1\frac{5}{16}''$	21.66	22.64	23.63	24.61	25.59	26.58	27.56	28.55	29.53	30.52
$1\frac{3}{8}''$	22.69	23.72	24.75	25.78	26.81	27.84	28.88	29.91	30.94	31.97
$1\frac{7}{16}''$	23.72	24.80	25.88	26.95	28.03	29.11	30.19	31.27	32.34	33.42
$1\frac{1}{2}''$	24.75	25.88	27.00	28.13	29.25	30.38	31.50	32.63	33.75	34.88
$1\frac{5}{8}''$	26.81	28.03	29.25	30.47	31.69	32.91	34.13	35.34	36.56	37.78
$1\frac{3}{4}''$	28.88	30.19	31.50	32.81	34.13	35.44	36.75	38.06	39.38	40.69
$1\frac{7}{8}''$	30.94	32.34	33.75	35.16	36.56	37.97	39.38	40.78	42.19	43.59
$2''$	33.00	34.50	36.00	37.50	39.00	40.50	42.00	43.50	45.00	46.51
$2\frac{1}{8}''$	35.06	36.65	38.25	39.84	41.44	43.03	44.63	46.22	47.81	49.42
$2\frac{1}{4}''$		38.81	40.50	42.19	43.88	45.56	47.25	48.94	50.63	52.32
$2\frac{3}{8}''$				44.53	46.32	48.09	49.88	51.66	53.44	55.23
$2\frac{1}{2}''$					48.76	50.63	52.50	54.38	56.25	58.14
$2\frac{5}{8}''$							55.13	57.09	59.06	61.04
$2\frac{3}{4}''$									61.88	63.95
$2\frac{7}{8}''$										
$3''$										
$3\frac{1}{8}''$										
$3\frac{1}{4}''$										
$3\frac{3}{8}''$										
$3\frac{1}{2}''$										
$3\frac{5}{8}''$										
$3\frac{3}{4}''$										
$3\frac{7}{8}''$										
$4''$										

TABLE XXVI.

Y WIDTH OF BEARING-SURFACE AT EACH END OF PINS,

g compressive stress of 6 tons per □″ on the projection of the semi-intrados upon a diametral plane. Upper horizontal line shows diameter of pin.
lines of inches show widths of bearings.

CLASS A.

3½″	3⅝″	3¾″	3⅞″	4″	4⅛″	4¼″	4⅜″	4½″	4⅝″	4¾″	4⅞″	5″	5⅛″	5¼″	5⅜″	5½″	5⅝″	5¾″	5⅞″	6″	
10.50	10.88	11.25	11.63	12.00	12.38	12.75	13.13	13.50	13.88												½″
11.81	12.23	12.66	13.08	13.50	13.92	14.34	14.77	15.19	15.61	16.03											9/16″
13.13	13.59	14.06	14.53	15.00	15.47	15.94	16.41	16.88	17.34	17.81	18.28										⅝″
14.44	14.95	15.47	15.98	16.50	17.02	17.53	18.05	18.56	19.08	19.59	20.11	20.63									11/16″
15.75	16.31	16.88	17.44	18.00	18.56	19.13	19.69	20.25	20.80	21.38	21.94	22.50	23.06	23.63							¾″
17.06	17.67	18.28	18.89	19.50	20.11	20.72	21.33	21.94	22.54	23.16	23.76	24.38	24.98	25.59	26.20						13/16″
18.38	19.03	19.69	20.34	21.00	21.66	22.31	22.97	23.63	24.27	24.94	25.59	26.25	26.91	27.56	28.22	28.88	29.53				⅞″
19.69	20.39	21.09	21.80	22.50	23.20	23.91	24.61	25.31	26.01	26.72	27.42	28.13	28.83	29.53	30.23	30.94	31.64	32.34			15/16″
21.00	21.75	22.50	23.25	24.00	24.75	25.50	26.25	27.00	27.74	28.50	29.25	30.00	30.75	31.50	32.25	33.00	33.75	34.50	35.25	36.00	1″
22.31	23.11	23.91	24.70	25.50	26.30	27.09	27.89	28.69	29.47	30.28	31.08	31.88	32.67	33.47	34.27	35.01	35.86	36.66	37.45	38.25	1 1/16″
23.63	24.47	25.31	26.16	27.00	27.84	28.69	29.53	30.38	31.21	32.06	32.90	33.75	34.59	35.44	36.28	37.13	37.97	38.81	39.66	40.50	1⅛″
24.94	25.83	26.72	27.61	28.50	29.39	30.28	31.17	32.06	32.94	33.84	34.73	35.63	36.52	37.41	38.30	39.19	40.08	40.97	41.86	42.75	1 3/16″
26.25	27.19	28.13	29.06	30.00	30.94	31.88	32.81	33.75	34.68	35.63	36.56	37.50	38.44	39.38	40.31	41.25	42.19	43.13	44.06	45.00	1¼″
27.56	28.55	29.53	30.52	31.50	32.48	33.47	34.45	35.44	36.41	37.41	38.39	39.38	40.36	41.34	42.33	43.31	44.30	45.28	46.27	47.25	1 5/16″
28.88	29.91	30.94	31.97	33.00	34.03	35.06	36.09	37.13	38.15	39.19	40.22	41.25	42.28	43.31	44.34	45.38	46.41	47.44	48.47	49.50	1⅜″
30.19	31.27	32.34	33.42	34.50	35.58	36.66	37.73	38.81	39.88	40.97	42.04	43.13	44.20	45.28	46.36	47.44	48.52	49.59	50.67	51.75	1 7/16″
31.50	32.63	33.75	34.88	36.00	37.13	38.25	39.38	40.50	41.62	42.75	43.87	45.00	46.13	47.25	48.38	49.50	50.63	51.75	52.88	54.00	1½″
34.13	35.34	36.56	37.78	39.00	40.22	41.44	42.66	43.88	45.08	46.31	47.53	48.75	49.97	51.19	52.41	53.63	54.84	56.06	57.28	58.50	1⅝″
36.75	38.06	39.38	40.69	42.00	43.31	44.63	45.94	47.25	48.55	49.88	51.18	52.50	53.81	55.13	56.44	57.75	59.06	60.38	61.69	63.00	1¾″
39.38	40.78	42.19	43.59	45.00	46.41	47.81	49.22	50.63	52.02	53.44	54.84	56.25	57.66	59.06	60.47	61.88	63.28	64.69	66.09	67.50	1⅞″
42.00	43.50	45.00	46.51	48.00	49.50	51.00	52.50	54.00	55.49	57.00	58.49	60.00	61.50	63.00	64.50	66.00	67.50	69.00	70.50	72.00	2″
44.63	46.22	47.81	49.42	51.00	52.59	54.19	55.78	57.38	58.96	60.56	62.15	63.75	65.34	66.94	68.53	70.13	71.72	73.31	74.91	76.50	2⅛″
47.25	48.94	50.63	52.32	54.00	55.69	57.38	59.06	60.75	62.43	64.13	65.81	67.50	69.19	70.88	72.56	74.25	75.94	77.62	79.31	81.00	2¼″
49.88	51.66	53.44	55.23	57.00	58.78	60.56	62.34	64.13	65.90	67.69	69.46	71.25	73.03	74.81	76.59	78.38	80.16	81.94	83.72	85.50	2⅜″
52.50	54.38	56.25	58.14	60.00	61.86	63.75	65.63	67.50	69.37	71.25	73.12	75.00	76.88	78.75	80.63	82.50	84.38	86.25	88.13	90.00	2½″
55.13	57.09	59.06	61.04	63.00	64.97	66.94	68.91	70.88	72.83	74.81	76.77	78.75	80.72	82.69	84.66	86.63	88.59	90.56	92.53	94.50	2⅝″
		61.88	63.95	66.00	68.06	70.13	72.19	74.25	76.30	78.38	80.43	82.50	84.56	86.63	88.69	90.75	92.81	94.87	96.94	99.00	2¾″
				69.00	71.16	73.31	75.47	77.63	79.77	81.94	84.08	86.25	88.41	90.56	92.72	94.88	97.03	99.19	101.34	103.50	2⅞″
					74.25	76.50	78.75	81.00	83.24	85.50	87.74	90.00	92.25	94.50	96.75	99.00	101.25	103.50	105.75	108.00	3″
							82.03	84.38	86.71	89.06	91.40	93.75	96.09	98.44	100.78	103.13	105.47	107.81	110.16	112.50	3⅛″
									90.18	92.63	95.05	97.50	99.94	102.38	104.81	107.25	109.69	112.12	114.56	117.00	3¼″
											98.71	101.25	103.78	106.31	108.84	111.38	113.91	116.44	118.97	121.50	3⅜″
													107.64	110.25	112.88	115.50	118.13	120.75	123.38	126.00	3½″
															116.91	119.63	122.34	125.06	127.78	130.50	3⅝″
																123.75	126.56	129.37	132.19	135.00	3¾″
																		133.69	136.59	139.50	3⅞″
																				144.00	4″

XV

BE

compres
lines c

AND

$3\frac{1}{4}''$
12.19
13.71
15.23
16.76
18.28
19.80
21.33
22.85
24.38
25.90
27.42
28.95
30.47
31.99
33.52
35.04
36.56
39.61
42.66
45.70
48.75
51.80
54.84

XVII.

BEARING-SURF

compressive stress of $7\frac{1}{2}$ ton a diametral plane. Upper lines of inches show widt

AND C.

3¼″	3⅜″	3½″	3⅝″	4¾″	4⅞″	5″	
12.19	12.66	13.13	13.59	17.81	18.28	18.75	½″
13.71	14.24	14.77	15.29	20.04	20.57	21.09	9/16″
15.23	15.82	16.41	16.99	22.27	22.85	23.44	⅝″
16.76	17.40	18.05	18.69	24.49	25.14	25.78	11/16″
18.28	18.98	19.69	20.39	26.72	27.42	28.13	¾″
19.80	20.57	21.33	22.09	28.95	29.71	30.47	13/16″
21.33	22.15	22.97	23.79	31.17	31.99	32.81	⅞″
22.85	23.73	24.61	25.49	33.40	34.28	35.16	15/16″
24.38	25.31	26.25	27.19	35.63	36.56	37.50	1″
25.90	26.89	27.89	28.89	37.85	38.85	39.84	1 1/16″
27.42	28.48	29.53	30.59	40.08	41.13	42.19	1⅛″
28.95	30.06	31.17	32.29	42.30	43.42	44.53	1 3/16″
30.47	31.64	32.81	33.98	44.53	45.70	46.88	1¼″
31.99	33.22	34.45	35.68	46.76	47.99	49.22	1 5/16″
33.52	34.80	36.09	37.38	48.98	50.27	51.56	1⅜″
35.04	36.39	37.73	39.08	51.21	52.56	53.91	1 7/16″
36.56	37.97	39.38	40.78	53.44	54.84	56.25	1½″
39.61	41.13	42.66	44.18	57.89	59.41	60.94	1⅝″
42.66	44.30	45.94	47.58	62.34	63.98	65.63	1¾″
45.70	47.46	49.22	50.98	66.80	68.56	70.31	1⅞″
48.75	50.63	52.50	54.38	71.25	73.13	75.00	2″
51.80	53.79	55.78	57.77	75.70	77.70	79.69	2⅛″
54.84	56.95	59.06	61.17	80.16	82.27	84.38	2¼″
	60.12	62.34	64.57	84.61	86.84	89.06	2⅜″
			67.97	89.06	91.41	93.75	2½″
			5	93.52	95.98	98.44	2⅝″
			9	97.97	100.56	103.13	2¾″
			3	102.42	105.12	107.81	2⅞″
			6	106.88	109.60	112.50	3″
			0	111.33	114.26	117.19	3⅛″
				115.78	118.83	121.86	3¼″
					123.40	126.56	3⅜″

TABLE XXV

TABLE FOR FINDING THE NECESSARY WIDTH OF BE

Having given the total pressure on said area, and the diameter of the pin. This table is calculated for a working compre
horizontal line shows diameter of pin. Vertical lines c

CLASSES B AND

	1 1/4″	1 3/8″	1 1/2″	1 5/8″	1 3/4″	1 7/8″	2″	2 1/8″	2 1/4″	2 3/8″	2 1/2″	2 5/8″	2 3/4″	2 7/8″	3″	3 1/8″	3 1/4″
1/2″	4.69	5.16	5.63	6.08	6.56	7.03	7.5	7.97	8.44	8.91	9.38	9.84	10.31	10.78	11.25	11.72	12.19
9/16″	5.27	5.80	6.33	6.86	7.38	7.91	8.44	8.96	9.49	10.02	10.55	11.07	11.60	12.13	12.66	13.18	13.71
5/8″	5.86	6.45	7.03	7.62	8.20	8.79	9.38	9.96	10.55	11.13	11.72	12.30	12.89	13.48	14.06	14.65	15.23
11/16″	6.45	7.09	7.73	8.38	9.02	9.67	10.31	10.96	11.60	12.25	12.89	13.54	14.18	14.82	15.47	16.11	16.76
3/4″	7.03	7.73	8.44	9.14	9.84	10.55	11.25	11.95	12.66	13.36	14.06	14.77	15.47	16.17	16.88	17.58	18.28
13/16″	7.62	8.38	9.14	9.90	10.66	11.43	12.19	12.95	13.71	14.47	15.23	16.00	16.76	17.52	18.28	19.04	19.80
7/8″	8.20	9.02	9.84	10.66	11.48	12.30	13.13	13.95	14.77	15.59	16.41	17.23	18.05	18.87	19.69	20.51	21.33
15/16″	8.79	9.67	10.55	11.43	12.30	13.18	14.06	14.94	15.82	16.70	17.58	18.46	19.34	20.21	21.09	21.97	22.85
1″	9.37	10.31	11.25	12.19	13.13	14.06	15.00	15.94	16.88	17.81	18.75	19.69	20.63	21.56	22.50	23.44	24.38
1 1/16″	9.96	10.96	11.95	12.95	13.95	14.94	15.94	16.93	17.93	18.93	19.92	20.92	21.91	22.91	23.91	24.90	25.90
1 1/8″	10.55	11.60	12.66	13.71	14.77	15.82	16.88	17.93	18.99	20.04	21.09	22.15	23.20	24.26	25.31	26.37	27.42
1 3/16″	11.13	12.25	13.36	14.47	15.59	16.70	17.81	18.93	20.04	21.15	22.27	23.38	24.49	25.61	26.72	27.83	28.95
1 1/4″	11.72	12.89	14.06	15.23	16.41	17.58	18.75	19.92	21.09	22.27	23.44	24.61	25.78	26.95	28.13	29.30	30.47
1 5/16″		13.53	14.77	16.00	17.23	18.46	19.69	20.92	22.15	23.38	24.61	25.84	27.07	28.30	29.53	30.76	31.99
1 3/8″				16.76	18.05	19.36	20.63	21.91	23.20	24.49	25.78	27.07	28.36	29.65	30.94	32.23	33.52
1 7/16″					18.87	20.21	21.56	22.91	24.26	25.61	26.95	28.30	29.65	31.00	32.34	33.69	35.04
1 1/2″							22.50	23.91	25.31	26.72	28.13	29.53	30.94	32.34	33.75	35.16	36.56
1 5/8″									27.42	28.95	30.47	32.00	33.52	35.04	36.56	38.09	39.61
1 3/4″										31.17	32.81	34.45	36.09	37.73	39.38	41.02	42.66
1 7/8″												36.91	38.67	40.43	42.19	43.95	45.70
2″													41.25	43.13	45.00	46.88	48.75
2 1/8″															47.81	49.80	51.80
2 1/4″																	54.84
2 3/8″																	
2 1/2″																	
2 5/8″																	
2 3/4″																	
2 7/8″																	
3″																	
3 1/8″																	
3 1/4″																	
3 3/8″																	

TABLE XXVII.

WIDTH OF BEARING-SURFACE AT EACH END OF PINS,

ted for a working compressive stress of 7½ tons per □″ on the projection of the semi-intrados upon a diametral plane. Upper
r of pin. Vertical lines of inches show widths of bearings.

CLASSES B AND C.

2⅞″	3″	3⅛″	3¼″	3⅜″	3½″	3⅝″	3¾″	3⅞″	4″	4⅛″	4¼″	4⅜″	4½″	4⅝″	4¾″	4⅞″	5″	
10.78	11.25	11.72	12.19	12.66	13.13	13.59	14.06	14.53	15.00	15.47	15.94	16.41	16.88	17.34	17.81	18.28	18.75	½″
12.13	12.66	13.18	13.71	14.24	14.77	15.29	15.82	16.35	16.88	17.40	17.93	18.46	18.94	19.51	20.04	20.57	21.09	9/16″
13.48	14.06	14.65	15.23	15.82	16.41	16.99	17.58	18.16	18.75	19.34	19.92	20.51	21.09	21.68	22.27	22.85	23.44	⅝″
14.82	15.47	16.11	16.76	17.40	18.05	18.69	19.34	19.98	20.63	21.27	21.91	22.56	23.20	23.85	24.49	25.14	25.78	11/16″
16.17	16.88	17.58	18.28	18.98	19.69	20.39	21.09	21.80	22.50	23.20	23.91	24.61	25.31	26.02	26.72	27.42	28.13	¾″
17.52	18.28	19.04	19.80	20.57	21.33	22.09	22.85	23.61	24.38	25.14	25.90	26.66	27.42	28.18	28.95	29.71	30.47	13/16″
18.87	19.69	20.5[illegible]	21.33	22.15	22.97	23.79	24.61	25.43	26.25	27.07	27.89	28.71	29.53	30.35	31.17	31.99	32.81	⅞″
20.21	21.09	21.97	22.85	23.73	24.61	25.49	26.37	27.25	28.13	29.00	29.88	30.76	31.64	32.52	33.40	34.28	35.16	15/16″
21.56	22.50	23.44	24.38	25.31	26.25	27.19	28.13	29.06	30.00	30.94	31.86	32.81	33.75	34.69	35.63	36.56	37.50	1″
22.91	23.91	24.90	25.90	26.89	27.89	28.89	29.88	30.88	31.88	32.87	33.87	34.86	35.86	36.86	37.85	38.85	39.84	1 1/16″
24.26	25.31	26.37	27.42	28.48	29.53	30.59	31.64	32.70	33.75	34.80	35.86	36.91	37.97	39.02	40.08	41.13	42.19	1⅛″
25.61	26.72	27.83	28.95	30.06	31.17	32.29	33.40	34.51	35.63	36.74	37.85	38.96	40.08	41.19	42.30	43.42	44.53	1 3/16″
26.95	28.13	29.30	30.47	31.64	32.81	33.98	35.16	36.33	37.50	38.67	39.84	41.02	42.19	43.36	44.53	45.70	46.88	1¼″
28.30	29.53	30.76	31.99	33.22	34.45	35.68	36.91	38.14	39.38	40.61	41.84	43.07	44.30	45.53	46.76	47.99	49.22	1 5/16″
29.65	30.94	32.23	33.52	34.80	36.09	37.38	38.67	39.96	41.25	42.54	43.83	45.12	46.41	47.70	48.98	50.27	51.56	1⅜″
31.00	32.34	33.69	35.04	36.39	37.73	39.08	40.43	41.78	43.13	44.47	45.82	47.17	48.52	49.86	51.21	52.56	53.91	1 7/16″
32.34	33.75	35.16	36.56	37.97	39.38	40.78	42.19	43.59	45.00	46.41	47.81	49.22	50.63	52.03	53.44	54.84	56.25	1½″
35.04	36.56	38.09	39.61	41.13	42.66	44.18	45.70	47.23	48.75	50.27	51.80	53.32	54.84	56.37	57.89	59.41	60.94	1⅝″
37.73	39.38	41.02	42.66	44.30	45.94	47.58	49.22	50.86	52.50	54.14	55.78	57.42	59.06	60.70	62.34	63.98	65.63	1¾″
40.43	42.19	43.95	45.70	47.46	49.22	50.98	52.73	54.49	51.25	58.01	59.77	61.52	63.28	65.04	66.80	68.56	70.31	1⅞″
43.13	45.00	46.88	48.75	50.63	52.50	54.38	56.25	58.12	60.00	61.88	63.75	65.63	67.50	69.38	71.25	73.13	75.00	2″
	47.81	49.80	51.80	53.79	55.78	57.77	59.77	61.76	63.75	65.74	67.73	69.73	71.72	73.71	75.70	77.70	79.69	2⅛″
			54.84	56.95	59.06	61.17	63.28	65.39	67.50	69.61	71.72	73.83	75.94	78.05	80.16	82.27	84.38	2¼″
				60.12	62.34	64.57	66.80	69.02	71.25	73.48	75.70	77.93	80.16	82.38	84.61	86.84	89.06	2⅜″
						67.97	70.31	72.66	75.00	77.34	79.69	82.03	84.38	86.72	89.06	91.41	93.75	2½″
							73.83	76.29	78.75	81.21	83.67	86.13	88.59	91.05	93.52	95.98	98.44	2⅝″
									82.50	85.08	87.66	90.23	92.81	95.39	97.97	100.56	103.13	2¾″
										88.95	91.64	94.34	97.03	99.73	102.42	105.12	107.81	2⅞″
												98.44	101.25	104.06	106.88	109.69	112.50	3″
													105.47	108.40	111.33	114.26	117.19	3⅛″
															115.78	118.83	121.86	3¼″
																123.40	126.56	3⅜″

00
601
354
106
859
601
354
106
859
60
35
10
85
60
35
10
85

	12" [C.			15" [A			Thickness of Web in Inches.
	W	*A*	*F*	*W*	*A*	*F*	
							0.250
							0.275
.00							0.300
601							0.325
.354							0.350
106							0.375
.859							0.400
601							0.425
.354	30.00	9.00	2.71				0.450
106	30.72	9.22	2.73				0.475
.859	31.72	9.52	2.75				0.500
.00	32.72	9.82	2.78	40.00	12.00	3.53	0.525
35	33.72	10.12	2.80	41.25	12.38	3.56	0.550
10	34.72	10.42	2.83	42.50	12.75	3.58	0.575
85	35.72	10.72	2.85	43.75	13.13	3.61	0.600
60	36.72	11.02	2.88	45.00	13.50	3.63	0.625
35	37.72	11.32	2.90	46.25	13.88	3.66	0.650
10	38.72	11.62	2.93	47.50	14.25	3.68	0.675
85	39.72	11.92	2.95	48.75	14.63	3.71	0.700
	40.72	12.22	2.98	50.00	15.00	3.73	0.725
	41.72	12.52	3.00	51.25	15.38	3.76	0.750
	42.72	12.82	3.03	52.50	15.75	3.78	0.775
	43.72	13.12	3.05	53.75	16.13	3.81	0.800
	44.72	13.42	3.08	55.00	16.50	3.83	0.825
	45.72	13.72	3.10	56.25	16.88	3.86	0.850
	46.72	14.02	3.13	57.50	17.25	3.88	0.875
	47.72	14.32	3.15	58.75	17.63	3.91	0.900
	48.72	14.62	3.18	60.00	18.00	3.93	0.925
	49.72	14.92	3.20				0.950

Thickness of Web in Inches.	4″ [. *A*.			4″ [. *B*.			5″ [. *A*.		
	W	*A*	*F*	*W*	*A*	*F*	*W*	*A*	*F*
0.250	6.05	1.82	1.62	7.08	2.13	1.75	7.02	2.11	1.69
0.275	6.31	1.92	1.65	7.41	2.23	1.77	7.44	2.24	1.72
0.300	6.71	2.02	1.67	7.74	2.33	1.80	7.85	2.36	1.74
0.325	7.00	2.10	1.70	8.08	2.43	1.83	8.27	2.49	1.77
0.350				8.41	2.53	1.85			
0.375				8.74	2.63	1.87			
0.400				9.00	2.70	1.89			
0.425									
0.450									
0.475									
0.500									
0.525									
0.550									
0.575									
0.600									
0.625									
0.650									
0.675									
0.700									
0.725									
0.750									
0.775									
0.800									
0.825									
0.850									
0.875									
0.900									
0.925									
0.950									

	5" [. A.			5" [. B.			6" [. A.			6" [. B.	
F	W	A	F	W	A	F	W	A	F	W	A
1.75	7.02	2.11	1.69	9.08	2.73	1.94	8.58	2.57	1.81	10.46	3.14
1.77	7.44	2.24	1.72	9.50	2.85	1.96	9.08	2.72	1.84	10.96	3.29
1.80	7.85	2.36	1.74	9.91	2.98	1.99	9.50	2.85	1.86	11.46	3.44
1.83	8.27	2.49	1.77	10.33	3.10	2.01				11.96	3.59
1.85				10.75	3.23	2.04				12.46	3.74
1.87				11.17	3.35	2.06				12.96	3.89
1.89				11.58	3.48	2.09				13.46	4.04
				12.00	3.60	2.11				13.96	4.19
				12.42	3.73	2.14				14.46	4.34
				12.83	3.85	2.16				14.96	4.49
				13.25	3.98	2.19				15.46	4.64
				13.68	4.10	2.21				15.96	4.79
				14.00	4.20	2.23					

TABLE XXVIII.

FINDING THE DIMENSIONS OF UNION IRON MILLS' CHANNEL BARS.

the weight per foot in pounds, *A* the area of section in square inches, and *F* the width of flange in inches.

7″ [. *B.*			8″ [. *A.*			8″ [. *B.*			9″ [. *A.*			9″ [. *B.*			10″ [. *A.*			10″ [. *B.*		
W	*A*	*F*	*W*	*A*	*F*	*W*	*A*	*F*	*W*	*A*	*F*	*W*	*A*	*F*	*W*	*A*	*F*	*W*	*A*	*F*
			12.79	3.84	2.02															
09	4.23	2.30	13.46	4.04	2.04	16.00	4.80	2.30					5.40	2.43				17.50	5.25	2.
68	4.40	2.33	14.13	4.24	2.07	16.59	4.98	2.32	14.50	4.35	2.50	..	5.58	2.45	16.00	4.80	2.52	18.33	5.50	2.
26	4.58	2.35	14.79	4.44	2.10	17.25	5.18	2.35				19.33	5.81	2.48				19.17	5.75	2.
84	4.75	2.38	15.46	4.64	2.12	17.92	5.38	2.37				20.10	6.03	2.50				20.00	6.00	2.
43	4.93	2.40				18.59	5.58	2.40				20.85	6.26	2.53				20.83	6.25	2
01	5.10	2.43				19.25	5.78	2.42				21.60	6.48	2.55				21.67	6.50	2.
59	5.28	2.45				19.92	5.98	2.45				22.35	6.71	2.58				22.50	6.75	2.
17	5.45	2.48				20.89	6.18	2.47				23.10	6.93	2.60				23.33	7.00	2.6
76	5.63	2.50				21.25	6.38	2.50				23.85	7.16	2.63				24.17	7.25	2.6
34	5.80	2.53				21.92	6.58	2.52				24.60	7.38	2.65				25.00	7.50	2.6
92	5.98	2.55				22.59	6.78	2.55				25.35	7.61	2.68				25.83	7.75	2.6
						23.26	6.98	2.57				26.10	7.83	2.70				26.67	8.00	2.7
						23.92	7.18	2.60				26.85	8.06	2.73				27.50	8.25	2.7
						24.59	7.38	2.62				27.60	8.28	2.75				28.33	8.50	2.7
						25.25	7.58	2.65				28.35	8.51	2.78				29.17	8.75	2.7
						25.92	7.78	2.67				29.10	8.73	2.80				30.00	9.00	2.8
						26.59	7.98	2.70				29.85	8.96	2.83						
						27.25	8.18	2.72												
						27.92	8.38	2.75												

S.

A.	10″ [. B.			10″ [. C.			12″ [. A.			12″ [. B.			12″ [. C.			15″ [A			Thickness of Web in Inches.
F	W	A	F	W	A	F	W	A	F	W	A	F	W	A	F	W	A	F	
																			0.250
																			0.275
	17.50	5.25	2.43	20.00	6.00	2.56													0.300
2.52	18.33	5.50	2.46	20.67	6.20	2.58	20.00	6.00	3.01	22.54	6.76	3.01							0.325
	19.17	5.75	2.48	21.50	6.45	2.61				23.54	7.06	3.04							0.350
	20.00	6.00	2.51	22.33	6.70	2.63				24.54	7.36	3.06							0.375
	20.83	6.25	2.53	23.17	6.95	2.66				25.54	7.66	3.09							0.400
	21.67	6.50	2.56	24.00	7.20	2.68				26.54	7.96	3.11							0.425
	22.50	6.75	2.58	24.83	7.45	2.71				27.54	8.26	3.14	30.00	9.00	2.71				0.450
	23.33	7.00	2.61	25.67	7.70	2.73				28.54	8.56	3.16	30.72	9.22	2.73				0.475
	24.17	7.25	2.63	26.50	7.95	2.76				29.54	8.86	3.19	31.72	9.52	2.75				0.500
	25.00	7.50	2.66	27.33	8.20	2.78							32.72	9.82	2.78	40.00	12.00	3.53	0.525
	25.83	7.75	2.68	28.17	8.45	2.81							33.72	10.12	2.80	41.25	12.38	3.56	0.550
	26.67	8.00	2.71	29.00	8.70	2.83							34.72	10.42	2.83	42.50	12.75	3.58	0.575
	27.50	8.25	2.73	29.83	8.95	2.86							35.72	10.72	2.85	43.75	13.13	3.61	0.600
	28.33	8.50	2.76	30.67	9.20	2.88							36.72	11.02	2.88	45.00	13.50	3.63	0.625
	29.17	8.75	2.78	31.50	9.45	2.91							37.72	11.32	2.90	46.25	13.88	3.66	0.650
	30.00	9.00	2.81	32.33	9.70	2.93							38.72	11.62	2.93	47.50	14.25	3.68	0.675
				33.17	9.95	2.96							39.72	11.92	2.95	48.75	14.63	3.71	0.700
				34.00	10.20	2.98							40.72	12.22	2.98	50.00	15.00	3.73	0.725
				34.83	10.45	3.01							41.72	12.52	3.00	51.25	15.38	3.76	0.750
													42.72	12.82	3.03	52.50	15.75	3.78	0.775
													43.72	13.12	3.05	53.75	16.13	3.81	0.800
													44.72	13.42	3.08	55.00	16.50	3.83	0.825
													45.72	13.72	3.10	56.25	16.88	3.86	0.850
													46.72	14.02	3.13	57.50	17.25	3.88	0.875
													47.72	14.32	3.15	58.75	17.63	3.91	0.900
													48.72	14.62	3.18	60.00	18.00	3.93	0.925
													49.72	14.92	3.20				0.950

OF

Fe

OF

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4″
4½″
5″
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21″
21½″
22″

Width, in inches.	Weight per foot, in pounds.			End allowance for one bar.	Rivet-Heads.	
	¼″	5/16″	⅜″		Diameter, in inches.	Weight of two heads, in pounds.
1½	1.25			0.145	½	0.08
1⅝	1.36			0.153	9/16	0.12
1¾	1.46			0.161	⅝	0.16
1⅞	1.57	1.95		0.180	11/16	0.20
2	1.67	2.08		0.188	¾	0.25
2⅛	1.78	2.21		0.197	13/16	0.32
2¼	1.88	2.34		0.215	⅞	0.40
2⅜		2.47	2.97	0.223	15/16	0.47
2½		2.60	3.13	0.231	1	0.55
2⅝		2.73	3.29	0.250		
2¾		2.86	3.44	0.258		
2⅞		3.00	3.60	0.266		
3		3.13	3.75	0.274		
3⅛		3.26	3.91	0.282		
3¼		3.39	4.06	0.291		
3⅜			4.22	0.299		
3½			4.38	0.307		
3⅝			4.54	0.315		
3¾			4.69	0.323		
3⅞			4.85	0.332		
4			5.00	0.340		
4⅛			5.16	0.348		

TABLE OF LE

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Approximate Lengths, in Feet, betw

	4″	4½″	5″	5½″	6″	6½″	7″	7½″	8″	8½″	9″	9½″	10″	10½″	11″	11½″	12″	12½″	13″
4″	0.468	0.500	0.534	0.563	0.599	0.636	0.669	0.706	0.741										
4½″	0.500	0.530	0.560	0.591	0.627	0.660	0.695	0.730	0.766	0.802	0.843								
5″	0.534	0.560	0.591	0.621	0.652	0.684	0.719	0.751	0.787	0.824	0.861	0.897	0.934						
5½″	0.563	0.591	0.621	0.650	0.679	0.711	0.745	0.776	0.809	0.844	0.880	0.916	0.952	0.990	1.027				
6″	0.599	0.627	0.652	0.679	0.709	0.737	0.770	0.801	0.835	0.868	0.904	0.936	0.974	1.009	1.045	1.081	1.119		
6½″	0.636	0.660	0.684	0.711	0.737	0.768	0.797	0.829	0.860	0.890	0.929	0.960	0.996	1.031	1.065	1.104	1.138	1.175	1.214
7″	0.669	0.695	0.719	0.745	0.770	0.797	0.827	0.857	0.886	0.920	0.951	0.985	1.019	1.052	1.086	1.122	1.157	1.195	1.230
7½″	0.706	0.730	0.751	0.776	0.801	0.829	0.857	0.884	0.916	0.945	0.979	1.012	1.043	1.074	1.110	1.145	1.180	1.218	1.250
8″	0.741	0.766	0.787	0.809	0.835	0.860	0.886	0.916	0.943	0.972	1.004	1.038	1.067	1.102	1.135	1.169	1.204	1.238	1.274
8½″		0.802	0.824	0.844	0.868	0.890	0.920	0.945	0.972	1.002	1.034	1.063	1.094	1.128	1.160	1.194	1.226	1.261	1.295
9″		0.843	0.861	0.880	0.904	0.929	0.951	0.979	1.004	1.034	1.061	1.090	1.123	1.153	1.187	1.219	1.252	1.284	1.320
9½″			0.897	0.916	0.936	0.960	0.985	1.012	1.038	1.063	1.090	1.121	1.150	1.181	1.213	1.246	1.275	1.312	1.344
10″			0.934	0.952	0.974	0.996	1.019	1.043	1.067	1.094	1.123	1.150	1.180	1.210	1.242	1.270	1.306	1.338	1.369
10½″				0.990	1.009	1.031	1.052	1.074	1.102	1.128	1.153	1.181	1.210	1.240	1.268	1.300	1.331	1.362	1.395
11″				1.027	1.045	1.065	1.086	1.110	1.135	1.160	1.187	1.213	1.242	1.268	1.298	1.329	1.357	1.390	1.422
11½″					1.081	1.104	1.122	1.145	1.169	1.194	1.219	1.246	1.270	1.300	1.329	1.356	1.388	1.422	1.450
12″					1.119	1.138	1.157	1.180	1.204	1.226	1.252	1.275	1.306	1.331	1.357	1.388	1.419	1.448	1.477
12½″						1.175	1.195	1.218	1.238	1.261	1.284	1.312	1.338	1.362	1.390	1.422	1.448	1.476	1.506
13″						1.214	1.230	1.250	1.274	1.295	1.320	1.344	1.369	1.395	1.422	1.450	1.477	1.506	1.535
13½″							1.269	1.289	1.309	1.332	1.353	1.377	1.404	1.428	1.453	1.480	1.509	1.536	1.565
14″							1.307	1.327	1.347	1.367	1.388	1.414	1.437	1.459	1.485	1.514	1.540	1.567	1.595
14½″								1.364	1.383	1.402	1.426	1.447	1.471	1.494	1.518	1.546	1.572	1.599	1.627
15″								1.399	1.420	1.440	1.458	1.478	1.502	1.527	1.550	1.577	1.604	1.630	1.657
15½″									1.456	1.475	1.495	1.517	1.538	1.559	1.584	1.610	1.637	1.664	1.689
16″									1.494	1.514	1.531	1.553	1.572	1.596	1.619	1.644	1.670	1.697	1.723
16½″										1.548	1.567	1.588	1.610	1.630	1.650	1.679	1.704	1.730	1.753
17″										1.587	1.602	1.625	1.644	1.667	1.689	1.714	1.738	1.762	1.786
17½″											1.640	1.661	1.680	1.704	1.725	1.749	1.771	1.795	1.821
18″											1.678	1.700	1.720	1.739	1.759	1.779	1.807	1.829	1.852
18½″												1.739	1.755	1.776	1.795	1.817	1.840	1.864	1.887
19″												1.774	1.793	1.814	1.833	1.851	1.877	1.900	1.923
19½″													1.830	1.850	1.868	1.888	1.913	1.936	1.956
20″													1.867	1.886	1.905	1.926	1.947	1.969	1.990
20½″														1.925	1.940	1.959	1.983	2.005	2.026
21″														1.959	1.977	1.996	2.020	2.040	2.060
21½″															2.016	2.034	2.057	2.076	2.099
22″															2.051	2.070	2.094	2.114	2.134
	4″	4½″	5″	5½″	6″	6½″	7″	7½″	8″	8½″	9″	9½″	10″	10½″	11″	11½″	12″	12½″	13″

TABLE XXIX.

TABLE OF LENGTHS OF LATTICE OR LACING BARS.

Weight per foot of same, and weight of rivet-heads.

oximate Lengths, in Feet, between Centres of Rivets.

½″	12″	12½″	13″	13½″	14″	14½″	15″	15½″	16″	16½″	17″	17½″	18″	18½″	19″	19½″	20″	20½″	21″	21½″	22″
081	1.119																				
.104	1.138	1.175	1.214																		
.122	1.157	1.195	1.230	1.269	1.307																
.145	1.180	1.218	1.250	1.289	1.327	1.364	1.399														
.169	1.204	1.238	1.274	1.309	1.347	1.383	1.420	1.456	1.494												
.194	1.226	1.261	1.295	1.332	1.367	1.402	1.440	1.475	1.514	1.548	1.587										
.219	1.252	1.284	1.320	1.353	1.388	1.426	1.458	1.495	1.531	1.567	1.602	1.640	1.678								
.246	1.275	1.312	1.344	1.377	1.414	1.447	1.478	1.517	1.553	1.588	1.625	1.661	1.700	1.739	1.774						
.270	1.306	1.338	1.369	1.404	1.437	1.471	1.502	1.538	1.572	1.610	1.644	1.680	1.720	1.755	1.793	1.830	1.867				
.300	1.331	1.362	1.395	1.428	1.459	1.494	1.527	1.559	1.596	1.630	1.667	1.704	1.739	1.776	1.814	1.850	1.886	1.925	1.959		
.329	1.357	1.390	1.422	1.453	1.485	1.518	1.550	1.584	1.619	1.650	1.689	1.725	1.759	1.795	1.833	1.868	1.905	1.940	1.977	2.016	2.051
.356	1.388	1.422	1.450	1.480	1.514	1.546	1.577	1.610	1.644	1.679	1.714	1.749	1.779	1.817	1.851	1.888	1.926	1.959	1.996	2.034	2.070
.388	1.419	1.448	1.477	1.509	1.540	1.572	1.604	1.637	1.670	1.704	1.738	1.771	1.807	1.840	1.877	1.913	1.947	1.983	2.020	2.057	2.094
.422	1.448	1.476	1.506	1.536	1.567	1.599	1.630	1.664	1.697	1.730	1.762	1.795	1.829	1.864	1.900	1.936	1.969	2.005	2.040	2.076	2.114
.450	1.477	1.506	1.535	1.565	1.595	1.627	1.657	1.689	1.723	1.753	1.786	1.821	1.852	1.887	1.923	1.956	1.990	2.026	2.060	2.099	2.134
.480	1.509	1.536	1.565	1.593	1.623	1.652	1.681	1.714	1.745	1.776	1.811	1.842	1.876	1.911	1.944	1.978	2.013	2.048	2.083	2.120	2.153
.514	1.540	1.567	1.595	1.623	1.650	1.680	1.710	1.742	1.771	1.805	1.835	1.869	1.902	1.935	1.969	2.003	2.037	2.071	2.106	2.140	2.178
.546	1.572	1.599	1.627	1.652	1.680	1.709	1.740	1.769	1.801	1.831	1.863	1.895	1.928	1.960	1.994	2.028	2.061	2.095	2.129	2.164	2.200
.577	1.604	1.630	1.657	1.681	1.710	1.740	1.770	1.800	1.830	1.861	1.892	1.925	1.956	1.988	2.022	2.053	2.089	2.122	2.155	2.189	2.225
.610	1.637	1.664	1.689	1.714	1.742	1.769	1.800	1.829	1.859	1.890	1.921	1.951	1.982	2.016	2.046	2.081	2.115	2.146	2.179	2.214	2.248
.644	1.670	1.697	1.723	1.745	1.771	1.801	1.830	1.859	1.889	1.920	1.950	1.980	2.013	2.042	2.077	2.109	2.140	2.172	2.206	2.240	2.273
.679	1.704	1.730	1.753	1.776	1.805	1.831	1.861	1.890	1.920	1.948	1.977	2.009	2.038	2.070	2.102	2.133	2.165	2.199	2.229	2.264	2.297
1.714	1.738	1.762	1.786	1.811	1.835	1.863	1.892	1.921	1.950	1.977	2.008	2.037	2.068	2.100	2.131	2.161	2.192	2.224	2.257	2.288	2.324
1.749	1.771	1.795	1.821	1.842	1.869	1.895	1.925	1.951	1.980	2.009	2.037	2.067	2.098	2.129	2.159	2.189	2.221	2.253	2.284	2.319	2.349
1.779	1.807	1.829	1.852	1.876	1.902	1.928	1.956	1.982	2.013	2.038	2.068	2.098	2.127	2.156	2.187	2.219	2.249	2.280	2.313	2.343	2.374
1.817	1.840	1.864	1.887	1.911	1.935	1.960	1.988	2.016	2.042	2.070	2.100	2.129	2.156	2.186	2.216	2.244	2.276	2.308	2.337	2.368	2.400
1.851	1.877	1.900	1.923	1.944	1.969	1.994	2.022	2.046	2.077	2.102	2.131	2.159	2.187	2.216	2.244	2.273	2.304	2.334	2.364	2.396	2.428
1.888	1.913	1.936	1.956	1.978	2.003	2.028	2.053	2.081	2.109	2.133	2.161	2.189	2.219	2.244	2.273	2.303	2.333	2.363	2.393	2.425	2.445
1.926	1.947	1.969	1.990	2.013	2.037	2.061	2.089	2.115	2.140	2.165	2.192	2.221	2.249	2.276	2.304	2.333	2.362	2.392	2.423	2.452	2.482
1.959	1.983	2.005	2.026	2.048	2.071	2.095	2.122	2.146	2.172	2.199	2.224	2.253	2.280	2.308	2.334	2.363	2.392	2.422	2.450	2.479	2.510
1.996	2.020	2.040	2.060	2.083	2.106	2.129	2.155	2.179	2.206	2.229	2.257	2.284	2.313	2.337	2.364	2.393	2.423	2.450	2.478	2.509	2.543
2.034	2.057	2.076	2.099	2.120	2.140	2.164	2.189	2.214	2.240	2.264	[illegible]	2.319	2.343	2.368	2.396	2.425	2.452	2.479	2.509	2.542	2.570
2.070	2.094	2.114	2.134	2.153	2.178	2.200	2.225	2.248	2.273	2.297	[illegible]	2.349	2.374	2.400	2.428	2.455	2.482	2.510	2.543	2.570	2.596
11½″	12″	12½″	13″	13½″	14″	14½″	15″	15½″	16″	16½″	17″	17½″	18″	18½″	19″	19½″	20″	20½″	21″	21½″	22″

TABLE XXIX.

TABLE OF LENGTHS OF LATTICE OR LACING BARS.

Weight per foot of same, and weight of rivet-heads.

Approximate Lengths, in Feet, between Centres of Rivets.

½″	11″	11½″	12″	12½″	13″	13½″	14″	14½″	15″	15½″	16″	16½″	17″	17½″	18″	18½″	19″	19½″
990	1.027																	
009	1.045	1.081	1.119															
031	1.065	1.104	1.138	1.175	1.214													
052	1.086	1.122	1.157	1.195	1.230	1.269	1.307											
074	1.110	1.145	1.180	1.218	1.250	1.289	1.327	1.364	1.399									
102	1.135	1.169	1.204	1.238	1.274	1.309	1.347	1.383	1.420	1.456	1.494							
128	1.160	1.194	1.226	1.261	1.295	1.332	1.367	1.402	1.440	1.475	1.514	1.548	1.587					
153	1.187	1.219	1.252	1.284	1.320	1.353	1.388	1.426	1.458	1.495	1.531	1.567	1.602	1.640	1.678			
181	1.213	1.246	1.275	1.312	1.344	1.377	1.414	1.447	1.478	1.517	1.553	1.588	1.625	1.661	1.700	1.739	1.774	
210	1.242	1.270	1.306	1.338	1.369	1.404	1.437	1.471	1.502	1.538	1.572	1.610	1.644	1.680	1.720	1.755	1.793	1.830
240	1.268	1.300	1.331	1.362	1.395	1.428	1.459	1.494	1.527	1.559	1.596	1.630	1.667	1.704	1.739	1.776	1.814	1.850
268	1.298	1.329	1.357	1.390	1.422	1.453	1.485	1.518	1.550	1.584	1.619	1.650	1.689	1.725	1.759	1.795	1.833	1.868
.300	1.329	1.356	1.388	1.422	1.450	1.480	1.514	1.546	1.577	1.610	1.644	1.679	1.714	1.749	1.779	1.817	1.851	1.888
.331	1.357	1.388	1.419	1.448	1.477	1.509	1.540	1.572	1.604	1.637	1.670	1.704	1.738	1.771	1.807	1.840	1.877	1.913
.362	1.390	1.422	1.448	1.476	1.506	1.536	1.567	1.599	1.630	1.664	1.697	1.730	1.762	1.795	1.829	1.864	1.900	1.936
.395	1.422	1.450	1.477	1.506	1.535	1.565	1.595	1.627	1.657	1.689	1.723	1.753	1.786	1.821	1.852	1.887	1.923	1.956
.428	1.453	1.480	1.509	1.536	1.565	1.593	1.623	1.652	1.681	1.714	1.745	1.776	1.811	1.842	1.876	1.911	1.944	1.978
.459	1.485	1.514	1.540	1.567	1.595	1.623	1.650	1.680	1.710	1.742	1.771	1.805	1.835	1.869	1.902	1.935	1.969	2.003
.494	1.518	1.546	1.572	1.599	1.627	1.652	1.680	1.709	1.740	1.769	1.801	1.831	1.863	1.895	1.928	1.960	1.994	2.028
.527	1.550	1.577	1.604	1.630	1.657	1.681	1.710	1.740	1.770	1.800	1.830	1.861	1.892	1.925	1.956	1.988	2.022	2.053
.559	1.584	1.610	1.637	1.664	1.689	1.714	1.742	1.769	1.800	1.829	1.859	1.890	1.921	1.951	1.982	2.016	2.046	2.081
.596	1.619	1.644	1.670	1.697	1.723	1.745	1.771	1.801	1.830	1.859	1.889	1.920	1.950	1.980	2.013	2.042	2.077	2.109
.630	1.650	1.679	1.704	1.730	1.753	1.776	1.805	1.831	1.861	1.890	1.920	1.948	1.977	2.009	2.038	2.070	2.102	2.133
.667	1.689	1.714	1.738	1.762	1.786	1.811	1.835	1.863	1.892	1.921	1.950	1.977	2.008	2.037	2.068	2.100	2.131	2.161
.704	1.725	1.749	1.771	1.795	1.821	1.842	1.869	1.895	1.925	1.951	1.980	2.009	2.037	2.067	2.098	2.129	2.159	2.189
.739	1.759	1.779	1.807	1.829	1.852	1.876	1.902	1.928	1.956	1.982	2.013	2.038	2.068	2.098	2.127	2.156	2.187	2.219
.776	1.795	1.817	1.840	1.864	1.887	1.911	1.935	1.960	1.988	2.016	2.042	2.070	2.100	2.129	2.156	2.186	2.216	2.244
.814	1.833	1.851	1.877	1.900	1.923	1.944	1.969	1.994	2.022	2.046	2.077	2.102	2.131	2.159	2.187	2.216	2.244	2.273
.850	1.868	1.888	1.913	1.936	1.956	1.978	2.003	2.028	2.053	2.081	2.109	2.133	2.161	2.189	2.219	2.244	2.273	2.303
.886	1.905	1.926	1.947	1.969	1.990	2.013	2.037	2.061	2.089	2.115	2.140	2.165	2.192	2.221	2.249	2.276	2.304	2.333
.925	1.940	1.959	1.983	2.005	2.026	2.048	2.071	2.095	2.122	2.146	2.172	2.199	2.224	2.253	2.280	2.308	2.334	2.363
.959	1.977	1.996	2.020	2.040	2.060	2.083	2.106	2.129	2.155	2.179	2.206	2.229	2.257	2.284	2.313	2.337	2.364	2.393
	2.016	2.034	2.057	2.076	2.099	2.120	2.140	2.164	2.189	2.214	2.240	2.264	2.288	2.319	2.343	2.368	2.396	2.425
	2.051	2.070	2.094	2.114	2.134	2.153	2.178	2.200	2.225	2.248	2.273	2.297	2.324	2.349	2.374	2.400	2.428	2.455
0½″	11″	11½″	12″	12½″	13″	13½″	14″	14½″	15″	15½″	16″	16½″	17″	17½″	18″	18½″	19″	19½″

18″	18½″	19″	19½″	20″	20½″	21″	21½″	22″	
									4″
									4½″
									5″
									5½″
									6″
									6½″
									7″
									7½″
									8″
									8½″
1.678									9″
1.700	1.739	1.774							9½″
1.720	1.755	1.793	1.830	1.867					10″
1.739	1.776	1.814	1.850	1.886	1.925	1.959			10½″
1.759	1.795	1.833	1.868	1.905	1.940	1.977	2.016	2.051	11″
1.779	1.817	1.851	1.888	1.926	1.959	1.996	2.034	2.070	11½″
1.807	1.840	1.877	1.913	1.947	1.983	2.020	2.057	2.094	12″
1.829	1.864	1.900	1.936	1.969	2.005	2.040	2.076	2.114	12½″
1.852	1.887	1.923	1.956	1.990	2.026	2.060	2.099	2.134	13″
1.876	1.911	1.944	1.978	2.013	2.048	2.083	2.120	2.153	13½″
1.902	1.935	1.969	2.003	2.037	2.071	2.106	2.140	2.178	14″
1.928	1.960	1.994	2.028	2.061	2.095	2.129	2.164	2.200	14½″
1.956	1.988	2.022	2.053	2.089	2.122	2.155	2.189	2.225	15″
1.982	2.016	2.046	2.081	2.115	2.146	2.179	2.214	2.248	15½″
2.013	2.042	2.077	2.109	2.140	2.172	2.206	2.240	2.273	16″
2.038	2.070	2.102	2.133	2.165	2.199	2.229	2.264	2.297	16½″
2.068	2.100	2.131	2.161	2.192	2.224	2.257	2.288	2.324	17″
2.098	2.129	2.159	2.189	2.221	2.253	2.284	2.319	2.349	17½″
2.127	2.156	2.187	2.219	2.249	2.280	2.313	2.343	2.374	18″
2.156	2.186	2.216	2.244	2.276	2.308	2.337	2.368	2.400	18½″
2.187	2.216	2.244	2.273	2.304	2.334	2.364	2.396	2.428	19″
2.219	2.244	2.273	2.303	2.333	2.363	2.393	2.425	2.445	19½″
2.249	2.276	2.304	2.333	2.362	2.392	2.423	2.452	2.482	20″
2.280	2.308	2.334	2.363	2.392	2.422	2.450	2.479	2.510	20½″
2.313	2.337	2.364	2.393	2.423	2.450	2.478	2.509	2.543	21″
2.343	2.368	2.396	2.425	2.452	2.479	2.509	2.542	2.570	21½″
2.374	2.400	2.428	2.455	2.482	2.510	2.543	2.570	2.596	22″
18″	18½″	19″	19½″	20″	20½″	21″	21½″	22″	

Width, in inches.	Weight per foot, in pounds.			End allowance for one bar.	Rivet-Heads.	
	¼″	5/16″	⅜″		Diameter, in inches.	Weight of two heads, in pounds.
1½	1.25			0.145	½	0.08
1⅝	1.36			0.153	9/16	0.12
1¾	1.46			0.161	⅝	0.16
1⅞	1.57	1.95		0.180	11/16	0.20
2	1.67	2.08		0.188	¾	0.25
2⅛	1.78	2.21		0.197	13/16	0.32
2¼	1.88	2.34		0.215	⅞	0.40
2⅜		2.47	2.97	0.223	15/16	0.47
2½		2.60	3.13	0.231	1	0.55
2⅝		2.73	3.29	0.250		
2¾		2.86	3.44	0.258		
2⅞		3.00	3.60	0.266		
3		3.13	3.75	0.274		
3⅛		3.26	3.91	0.282		
3¼		3.39	4.06	0.291		
3⅜			4.22	0.299		
3½			4.38	0.307		
3⅝			4.54	0.315		
3¾			4.69	0.323		
3⅞			4.85	0.332		
4			5.00	0.340		
4⅛			5.16	0.348		

TABLE
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SPAC

$D =$ d
faces of c
given, the
containing

D
4″
5″
6″
7″
8″
9″
10″
12″
15″

TABLE XXX.

TABLE OF SIZES OF LATTICE BARS FOR CHANNELS OF VARIOUS DEPTHS, AND SPACED AT VARIOUS DISTANCES.

D = depth of channel, and d = distance between inner faces of channels. If the value of d lie between the values given, the size of lattice bars is to be taken from the column containing the *next largest* value of d.

D	Sizes of Lattice-Bars.				
	$d = D$	$d = 1.25D$	$d = 1.5D$	$d = 1.75D$	$d = 2D$
4″	$\frac{1}{4}'' \times 1\frac{1}{2}''$	$\frac{1}{4}'' \times 1\frac{1}{2}''$	$\frac{1}{4}'' \times 1\frac{1}{2}''$	$\frac{1}{4}'' \times 1\frac{5}{8}''$	$\frac{1}{4}'' \times 1\frac{3}{4}''$
5″	$\frac{1}{4}'' \times 1\frac{1}{2}''$	$\frac{1}{4}'' \times 1\frac{1}{2}''$	$\frac{1}{4}'' \times 1\frac{5}{8}''$	$\frac{1}{4}'' \times 1\frac{3}{4}''$	$\frac{1}{4}'' \times 1\frac{7}{8}''$
6″	$\frac{1}{4}'' \times 1\frac{5}{8}''$	$\frac{1}{4}'' \times 1\frac{3}{4}''$	$\frac{1}{4}'' \times 1\frac{7}{8}''$	$\frac{1}{4}'' \times 1\frac{7}{8}''$	$\frac{1}{4}'' \times 2''$
7″	$\frac{1}{4}'' \times 1\frac{3}{4}''$	$\frac{1}{4}'' \times 1\frac{7}{8}''$	$\frac{1}{4}'' \times 1\frac{7}{8}''$	$\frac{1}{4}'' \times 2''$	$\frac{5}{16}'' \times 1\frac{7}{8}''$
8″	$\frac{1}{4}'' \times 1\frac{7}{8}''$	$\frac{5}{16}'' \times 1\frac{7}{8}''$	$\frac{5}{16}'' \times 1\frac{7}{8}''$	$\frac{5}{16}'' \times 2''$	$\frac{5}{16}'' \times 2\frac{1}{8}''$
9″	$\frac{5}{16}'' \times 2\frac{1}{8}''$	$\frac{5}{16}'' \times 2\frac{1}{8}''$	$\frac{5}{16}'' \times 2\frac{1}{4}''$	$\frac{5}{16}'' \times 2\frac{3}{8}''$	$\frac{5}{16}'' \times 2\frac{1}{2}''$
10″	$\frac{5}{16}'' \times 2\frac{1}{4}''$	$\frac{5}{16}'' \times 2\frac{1}{4}''$	$\frac{5}{16}'' \times 2\frac{3}{8}''$	$\frac{5}{16}'' \times 2\frac{1}{2}''$	
12″	$\frac{3}{8}'' \times 2\frac{3}{8}''$	$\frac{3}{8}'' \times 2\frac{1}{2}''$	$\frac{3}{8}'' \times 2\frac{5}{8}''$		
15″	$\frac{3}{8}'' \times 2\frac{5}{8}''$	$\frac{3}{8}'' \times 2\frac{3}{4}''$			

TABLE
NELS
AT V

$D =$ c
faces of c
given, the
containin

D
4″
5″
6″
7″
8″
9″
10″
12″
15″

TABLE XXXI.

TABLE OF SIZES OF LACING-BARS FOR CHANNELS OF VARIOUS DEPTHS, AND SPACED AT VARIOUS DISTANCES.

D = depth of channel, and *d* = distance between inner faces of channels. If the value of *d* lie between the values given, the size of lacing-bars is to be taken from the column containing the *next largest* value of *d*.

D	Sizes of Lacing-Bars.				
	$d = D$	$d = 1.25D$	$d = 1.5D$	$d = 1.75D$	$d \mathrel{\overline{\gtrless}} 2D$
4″	$\frac{1}{4}'' \times 1\frac{3}{4}''$	$\frac{1}{4}'' \times 1\frac{3}{4}''$	$\frac{1}{4}'' \times 1\frac{7}{8}''$	$\frac{1}{4}'' \times 2''$	$\frac{1}{4}'' \times 2\frac{1}{4}''$
5″	$\frac{1}{4}'' \times 1\frac{7}{8}''$	$\frac{1}{4}'' \times 2''$	$\frac{1}{4}'' \times 2\frac{1}{4}''$	$\frac{5}{16}'' \times 2''$	$\frac{5}{16}'' \times 2\frac{1}{4}''$
6″	$\frac{1}{4}'' \times 2''$	$\frac{1}{4}'' \times 2\frac{1}{4}''$	$\frac{5}{16}'' \times 2''$	$\frac{5}{16}'' \times 2\frac{1}{4}''$	$\frac{5}{16}'' \times 2\frac{1}{4}''$
7″	$\frac{1}{4}'' \times 2\frac{1}{4}''$	$\frac{5}{16}'' \times 2''$	$\frac{5}{16}'' \times 2\frac{1}{4}''$	$\frac{5}{16}'' \times 2\frac{1}{4}''$	$\frac{5}{16}'' \times 2\frac{3}{8}''$
8″	$\frac{5}{16}'' \times 2\frac{1}{4}''$	$\frac{5}{16}'' \times 2\frac{1}{4}''$	$\frac{5}{16}'' \times 2\frac{3}{8}''$	$\frac{5}{16}'' \times 2\frac{5}{8}''$	$\frac{5}{16}'' \times 2\frac{1}{2}''$
9″	$\frac{5}{16}'' \times 2\frac{3}{8}''$	$\frac{5}{16}'' \times 2\frac{1}{2}''$	$\frac{5}{16}'' \times 2\frac{1}{2}''$	$\frac{5}{16}'' \times 3''$	$\frac{3}{8}'' \times 2\frac{1}{2}''$
10″	$\frac{5}{16}'' \times 2\frac{1}{2}''$	$\frac{5}{16}'' \times 3''$	$\frac{3}{8}'' \times 2\frac{1}{2}''$	$\frac{3}{8}'' \times 3''$	
12″	$\frac{3}{8}'' \times 3''$	$\frac{3}{8}'' \times 3\frac{1}{4}''$	$\frac{3}{8}'' \times 3\frac{1}{2}''$		
15″	$\frac{3}{8}'' \times 3\frac{1}{4}''$	$\frac{3}{8}'' \times 4''$			

TABLE OF STAY PLATES.

To be used in connection with latticing or double-riveted lacing

TABLE OF STAY PLATES.

To be used in connection with latticing or double-riveted lacing.

D = depth of channel, d = distance between inner faces of channels, t = thickness of stay plate, l = length of same, and n = number of rivets on each side of stay plate.

If the value of d lie between the values given, the size of stay plates is to be taken from the column containing the *next largest* value of d.

D	Sizes of Stay Plates.														
	$d = D$.			$d = 1.25D$.			$d = 1.5D$.			$d = 1.75D$.			$d \geqq 2D$.		
	t	l	n	t	l	n	t	l	n	t	l	n	t	l	n
4"	¼"	4½"	3	¼"	4¾"	3	¼"	5"	3	¼"	5¼"	3	¼"	5½"	3
5"	¼"	5"	3	¼"	5¼"	3	¼"	5½"	3	¼"	5¾"	4	¼"	6"	4
6"	¼"	5¾"	3	¼"	6"	3	¼"	6¼"	3	¼"	6½"	3	¼"	6¾"	3
7"	¼"	6¼"	3	¼"	6½"	3	¼"	6¾"	3	¼"	7"	3	¼"	7¼"	4
8"	5/16"	6½"	3	5/16"	6¾"	3	5/16"	7"	3	5/16"	7¼"	4	5/16"	7½"	4
9"	5/16"	7½"	3	5/16"	7¾"	3	5/16"	8"	3	5/16"	8¼"	3	5/16"	8½"	4
10"	5/16"	8"	3	5/16"	8¼"	3	5/16"	8½"	4	5/16"	8¾"	4	5/16"	9"	4
12"	⅜"	8½"	4	⅜"	8¾"	4	[illegible]	9"	4	⅜"	9¼"	4			
15"	⅜"	10"	4	⅜"	11"	4	⅜"	12"	5						

TABLE OF STAY PLATES.

TABLE XXXIII.

TABLE OF STAY PLATES.

To be used in connection with single-rivet lacing.

D = depth of channel, d = distance between inner faces of channels, t = thickness of stay plate, l = length of same, and n = number of rivets on each side of stay plate.

If the value of d lie between the values given, the size of stay plates is to be taken from the column containing the *next largest* value of d.

Sizes of Stay Plates.

D	$d = D$.			$d = 1.25D$.			$d = 1.5D$.			$d = 1.75D$.			$d \geqq 2D$.		
	t	l	n	t	l	n	t	l	n	t	l	n	t	l	n
4"	$\frac{1}{4}$"	6"	4	$\frac{1}{4}$"	$6\frac{1}{2}$"	4	$\frac{1}{4}$"	7"	5	$\frac{1}{4}$"	$7\frac{1}{2}$"	5	$\frac{1}{4}$"	8"	5
5"	$\frac{1}{4}$"	8"	5	$\frac{1}{4}$"	$8\frac{1}{2}$"	6	$\frac{1}{4}$"	9"	6	$\frac{1}{4}$"	$9\frac{1}{2}$"	7	$\frac{1}{4}$"	10"	7
6"	$\frac{1}{4}$"	9"	5	$\frac{1}{4}$"	$9\frac{1}{2}$"	5	$\frac{1}{4}$"	10"	5	$\frac{1}{4}$"	$10\frac{1}{2}$"	6	$\frac{1}{4}$"	11"	6
7"	$\frac{1}{4}$"	$10\frac{1}{2}$"	6	$\frac{1}{4}$"	11"	6	$\frac{1}{4}$"	$11\frac{1}{2}$"	6	$\frac{1}{4}$"	12"	7	$\frac{1}{4}$"	$12\frac{1}{2}$"	7
8"	$\frac{5}{16}$"	11"	6	$\frac{5}{16}$"	$11\frac{1}{2}$"	6	$\frac{5}{16}$"	12"	7	$\frac{5}{16}$"	$12\frac{1}{2}$"	7	$\frac{5}{16}$"	13"	7
9"	$\frac{5}{16}$"	$12\frac{1}{2}$"	6	$\frac{5}{16}$"	13"	6	$\frac{5}{16}$"	$13\frac{1}{2}$"	6	$\frac{5}{16}$"	14"	6	$\frac{5}{16}$"	$14\frac{1}{2}$"	7
10"	$\frac{5}{16}$"	$13\frac{1}{2}$"	6	$\frac{5}{16}$"	14"	6	$\frac{5}{16}$"	$14\frac{1}{2}$"	7	$\frac{5}{16}$"	15"	7	$\frac{5}{16}$"	$15\frac{1}{2}$"	7
12"	[illegible]	16"	8	$\frac{3}{8}$"	17"	8	$\frac{3}{8}$"	18"	9	$\frac{3}{8}$"	19"	9			
15"	[illegible]	20"	8	$\frac{3}{8}$"	22"	9	$\frac{3}{8}$"	24"	10						

Formu
e the dia

Dia.	1
[illegible]	3
[illegible]	3
[illegible]	3
[illegible]	3
[illegible]	3
[illegible]	3
[illegible]	3
[illegible]	4
[illegible]	4
[illegible]	4
[illegible]	4
[illegible]	4
[illegible]	4
[illegible]	4
[illegible]	4
[illegible]	4
[illegible]	4
[illegible]	4
[illegible]	5
Dia.	1

BRIDGES OF CLASS A.

Formula, f roller in inches. The first and last vertical lines
e the diam the permissible pressures on the rollers.

Dia.	10" "	25"	26"	27"	28"	29"	30"	Dia.
[illegible]	3.3104	8.27	8.60	8.93	9.26	9.59	9.92	1¾"
[illegible]	3.4222	8.56	8.90	9.24	9.58	9.93	10.27	1⅞"
[illegible]	3.5449	8.84	9.19	9.55	9.90	10.25	10.61	2"
[illegible]	3.6475	9.11	9.47	9.84	10.20	10.57	10.93	2⅛"
[illegible]	3.7500	9.38	9.75	10.13	10.50	10.88	11.25	2¼"
[illegible]	3.8525	9.63	10.02	10.40	10.79	11.17	11.56	2⅜"
[illegible]	3.9549	9.88	10.28	10.67	11.07	11.46	11.86	2½"
[illegible]	4.0572	10.13	10.53	10.94	11.34	11.75	12.15	2⅝"
[illegible]	4.1595	10.37	10.78	11.19	11.61	12.02	12.44	2¾"
[illegible]	4.2417	10.60	11.02	11.45	11.87	12.29	12.72	2⅞"
[illegible]	4.3339	10.83	11.26	11.69	12.12	12.56	12.99	3"
[illegible]	4.4261	11.05	11.50	11.92	12.37	12.82	13.26	3⅛"
[illegible]	4.5182	11.27	11.72	12.17	12.62	13.07	13.52	3¼"
[illegible]	4.5902	11.48	11.94	12.40	12.86	13.32	13.78	3⅜"
[illegible]	4.6822	11.69	12.16	12.63	13.10	13.56	14.03	3½"
[illegible]	4.7642	11.90	12.38	12.85	13.33	13.80	14.28	3⅝"
[illegible]	4.8462	12.10	12.59	13.07	13.55	14.04	14.52	3¾"
[illegible]	4.9281	12.30	12.79	13.29	13.78	14.27	14.76	3⅞"
[illegible]	5.0000	12.50	13.00	13.50	14.00	14.50	15.00	4"
Dia.	10" "	25"	26"	27"	28"	29"	30"	Dia.

TA

TABLE OF PERMISSIBLE PRESSU

Formula, $p = 0.25\sqrt{d}$, where p is the pressure in tons per linea
...e the diameters, and the upper and lower lines the length of roll

Dia.	10″	11″	12″	13″	14″	15″	16″	17″	18″
	3.31	3.64	3.97	4.30	4.63	4.96	5.29	5.62	5.95
	3.42	3.77	4.11	4.45	4.79	5.13	5.48	5.82	6.1
	3.54	3.89	4.24	4.60	4.95	5.30	5.66	6.01	6.36
	3.64	4.01	4.37	4.74	5.10	5.47	5.83	6.19	6.60
	3.75	4.13	4.50	4.88	5.25	5.63	6.00	6.38	6.75
	3.85	4.24	4.62	5.01	5.39	5.78	6.16	6.55	6.94
	3.95	4.35	4.74	5.14	5.53	5.93	6.32	6.72	7.1
	4.05	4.46	4.86	5.27	5.67	6.08	6.48	6.89	7.2
	4.15	4.56	4.98	5.39	5.80	6.22	6.63	7.05	7.4
	4.24	4.66	5.09	5.51	5.93	6.36	6.78	7.21	7.6
	4.33	4.76	5.20	5.63	6.06	6.50	6.93	7.36	7.7
	4.42	4.86	5.30	5.74	6.19	6.63	7.07	7.51	7.9
	4.51	4.96	5.41	5.86	6.31	6.76	7.21	7.66	8.1
	4.59	5.05	5.51	5.97	6.43	6.89	7.35	7.81	8.2
	4.68	5.14	5.61	6.08	6.55	7.02	7.48	7.95	8.4
	4.76	5.24	5.71	6.19	6.66	7.14	7.62	8.09	8.5
	4.84	5.33	5.81	6.29	6.78	7.26	7.75	8.23	8.7
	4.92	5.41	5.91	6.40	6.89	7.38	7.87	8.37	8.8
	5.00	5.50	6.00	6.50	7.00	7.50	8.00	8.50	9.0
Dia.	10″	11″	12″	13″	14″	15″	16″	17″	18″

TABLE XXXIV.

MISSIBLE PRESSURES ON ROLLERS FOR BRIDGES OF CLASS A.

e pressure in tons per lineal inch of roller, and *d* the diameter of roller in inches. The first and last vertical lines ver lines the length of rollers. The intermediate spaces contain the permissible pressures on the rollers.

15″	16″	17″	18″	19″	20″	21″	22″	23″	24″	25″	26″	27″	28″	29″	30″	Dia.
4.96	5.29	5.62	5.95	6.28	6.61	6.94	7.28	7.61	7.94	8.27	8.60	8.93	9.26	9.59	9.92	1¾″
5.13	5.48	5.82	6.16	6.50	6.85	7.19	7.53	7.87	8.22	8.56	8.90	9.24	9.58	9.93	10.27	1⅞″
5.30	5.66	6.01	6.36	6.72	7.07	7.43	7.80	8.13	8.49	8.84	9.19	9.55	9.90	10.25	10.61	2″
5.47	5.83	6.19	6.60	6.92	7.29	7.65	8.02	8.38	8.75	9.11	9.47	9.84	10.20	10.57	10.93	2⅛″
5.63	6.00	6.38	6.75	7.13	7.50	7.88	8.25	8.63	9.00	9.38	9.75	10.13	10.50	10.88	11.25	2¼″
5.78	6.16	6.55	6.94	7.32	7.71	8.09	8.48	8.86	9.25	9.63	10.02	10.40	10.79	11.17	11.56	2⅜″
5.93	6.32	6.72	7.12	7.51	7.91	8.30	8.70	9.09	9.49	9.88	10.28	10.67	11.07	11.46	11.86	2½″
6.08	6.48	6.89	7.29	7.70	8.10	8.51	8.91	9.32	9.72	10.13	10.53	10.94	11.34	11.75	12.15	2⅝″
6.22	6.63	7.05	7.46	7.88	8.29	8.71	9.12	9.54	9.95	10.37	10.78	11.19	11.61	12.02	12.44	2¾″
6.36	6.78	7.21	7.63	8.05	8.48	8.90	9.33	9.75	10.17	10.60	11.02	11.45	11.87	12.29	12.72	2⅞″
6.50	6.93	7.36	7.79	8.23	8.66	9.09	9.53	9.96	10.39	10.83	11.26	11.69	12.12	12.56	12.99	3″
6.63	7.07	7.51	7.95	8.40	8.84	9.28	9.72	10.16	10.61	11.05	11.50	11.92	12.37	12.82	13.26	3⅛″
6.76	7.21	7.66	8.11	8.56	9.01	9.46	9.92	10.37	10.82	11.27	11.72	12.17	12.62	13.07	13.52	3¼″
6.89	7.35	7.81	8.27	8.73	9.19	9.65	10.10	10.56	11.02	11.48	11.94	12.40	12.86	13.32	13.78	3⅜″
7.02	7.48	7.95	8.42	8.89	9.35	9.82	10.29	10.76	11.22	11.69	12.16	12.63	13.10	13.56	14.03	3½″
7.14	7.62	8.09	8.57	9.04	9.52	10.00	10.47	10.95	11.42	11.90	12.38	12.85	13.33	13.80	14.28	3⅝″
7.26	7.75	8.23	8.71	9.20	9.68	10.17	10.65	11.13	11.62	12.10	12.59	13.07	13.55	14.04	14.52	3¾″
7.38	7.87	8.37	8.86	9.35	9.84	10.33	10.83	11.32	11.81	12.30	12.79	13.29	13.78	14.27	14.76	3⅞″
7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00	12.50	13.00	13.50	14.00	14.50	15.00	4″
15″	16″	17″	18″	19″	20″	21″	22″	23″	24″	25″	26″	27″	28″	29″	30″	Dia.

TA[illegible]

Formula [illegible]
the diam[illegible]

Dia.	10"
	3.8[illegible]
	3.9[illegible]
	4.1[illegible]
	4.2[illegible]
	4.4[illegible]
	4.5[illegible]
	4.[illegible]
	4.8[illegible]
	4.9[illegible]
	5.0[illegible]
	5.1[illegible]
	5.3[illegible]
	5.4[illegible]
	5.5[illegible]
	5.63
	5.74
	5.85
Dia.	10"

TA ES OF CLASSES B AND C.

Formula roller in inches. The first and last vertical lines the dian he permissible pressures on the rollers.

Dia.	10"	25"	26"	27"	28"	29"	30"	Dia.
	3.8	9.57	9.95	10.34	10.72	11.10	11.48	$1\frac{1}{2}$"
	3.9	9.96	10.36	10.76	11.16	11.55	11.95	$1\frac{5}{8}$"
	4.1	10.34	10.75	11.16	11.58	11.99	12.40	$1\frac{3}{4}$"
	4.2	10.70	11.13	11.55	11.98	12.41	12.84	$1\frac{7}{8}$"
	4.4	11.05	11.49	11.93	12.38	12.82	13.26	2"
	4.5	11.39	11.84	12.30	12.75	13.21	13.67	$2\frac{1}{8}$"
	4.6	11.72	12.19	12.66	13.13	13.60	14.06	$2\frac{1}{4}$"
	4.8	12.04	12.52	13.00	13.48	13.97	14.45	$2\frac{3}{8}$"
	4.9	12.35	12.85	13.34	13.83	14.33	14.82	$2\frac{1}{2}$"
	5.0	12.66	13.17	13.67	14.18	14.69	15.19	$2\frac{5}{8}$"
	5.1	12.96	13.48	13.99	14.51	15.03	15.55	$2\frac{3}{4}$"
	5.3	13.25	13.78	14.31	14.84	15.37	15.90	$2\frac{7}{8}$"
	5.41	13.53	14.07	14.62	15.16	15.70	16.24	3"
	5.52	13.81	14.36	14.91	15.47	16.02	16.57	$3\frac{1}{8}$"
	5.63	14.09	14.65	15.21	15.78	16.34	16.90	$3\frac{1}{4}$"
	5.74	14.35	14.93	15.50	16.07	16.65	17.22	$3\frac{3}{8}$"
	5.85	14.62	15.20	15.78	16.37	16.95	17.54	$3\frac{1}{2}$"
Dia.	10"	25"	26"	27"	28"	29"	30"	Dia.

T

TABLE OF PERMISSIBLE PRESSURES

Formula, $p = 0.3125\sqrt{d}$, where p is the pressure in tons per l
the diameters, and the upper and lower lines the length of r

D a.	10″	11″	12″	13″	14″	15″	16″	17″	18
	3.83	4.21	4.59	4.98	5.36	5.74	6.12	6.51	6.
	3.98	4.38	4.78	5.18	5.58	5.98	6.37	6.77	7.
	4.13	4.55	4.96	5.37	5.79	6.20	6.61	7.03	7.
	4.28	4.71	5.13	5.56	5.99	6.42	6.85	7.27	7.
	4.42	4.86	5.30	5.75	6.19	6.63	7.07	7.51	7.
	4.56	5.01	5.47	5.92	6.38	6.83	7.29	7.74	8.
	4.69	5.16	5.63	6.09	6.56	7.03	7.50	7.97	8.
	4.82	5.30	5.78	6.26	6.74	7.22	7.71	8.19	8.
	4.94	5.44	5.93	6.42	6.92	7.41	7.91	8.40	8.
	5.06	5.57	6.08	6.58	7.09	7.60	8.10	8.61	9.
	5.18	5.70	6.22	6.74	7.26	7.77	8.29	8.81	9.
	5.30	5.83	6.36	6.89	7.42	7.95	8.48	9.01	9.
	5.41	5.95	6.50	7.04	7.58	8.12	8.66	9.20	9.
	5.52	6.08	6.63	7.18	7.73	8.29	8.84	9.39	9.
	5.63	6.20	6.76	7.32	7.89	8.45	9.01	9.58	10.
	5.74	6.32	6.89	7.46	8.04	8.61	9.19	9.76	10.
	5.85	6.43	7.02	7.60	8.18	8.77	9.35	9.94	10.
D a.	10″	11″	12″	13″	14″	15″	16″	17″	18

TABLE XXXV.

…BLE PRESSURES ON ROLLERS FOR BRIDGES OF CLASSES B AND C.

…the pressure in tons per lineal inch of roller, and *d* the diameter of roller in inches. The first and last vertical lines …ower lines the length of rollers. The intermediate spaces contain the permissible pressures on the rollers.

15″	16″	17″	18″	19″	20″	21″	22″	23″	24″	25″	26″	27″	28″	29″	30″	Dia.
5.74	6.12	6.51	6.89	7.27	7.66	8.04	8.42	8.80	9.19	9.57	9.95	10.34	10.72	11.10	11.48	1½″
5.98	6.37	6.77	7.17	7.57	7.97	8.37	8.76	9.16	9.56	9.96	10.36	10.76	11.16	11.55	11.95	1⅝″
6.20	6.61	7.03	7.44	7.85	8.27	8.68	9.09	9.51	9.92	10.34	10.75	11.16	11.58	11.99	12.40	1¾″
6.42	6.85	7.27	7.70	8.13	8.56	8.99	9.41	9.84	10.27	10.70	11.13	11.55	11.98	12.41	12.84	1⅞″
6.63	7.07	7.51	7.96	8.40	8.84	9.28	9.72	10.17	10.61	11.05	11.49	11.93	12.38	12.82	13.26	2″
6.83	7.29	7.74	8.20	8.65	9.11	9.57	10.02	10.48	10.93	11.39	11.84	12.30	12.75	13.21	13.67	2⅛″
7.03	7.50	7.97	8.44	8.91	9.38	9.84	10.31	10.78	11.25	11.72	12.19	12.66	13.13	13.60	14.06	2¼″
7.22	7.71	8.19	8.67	9.15	9.63	10.11	10.60	11.08	11.56	12.04	12.52	13.00	13.48	13.97	14.45	2⅜″
7.41	7.91	8.40	8.89	9.39	9.88	10.38	10.87	11.36	11.86	12.35	12.85	13.34	13.83	14.33	14.82	2½″
7.60	8.10	8.61	9.12	9.62	10.13	10.63	11.14	11.65	12.15	12.66	13.17	13.67	14.18	14.69	15.19	2⅝″
7.77	8.29	8.81	9.33	9.85	10.37	10.88	11.40	11.92	12.44	12.96	13.48	13.99	14.51	15.03	15.55	2¾″
7.95	8.48	9.01	9.54	10.07	10.60	11.13	11.66	12.19	12.72	13.25	13.78	14.31	14.84	15.37	15.90	2⅞″
8.12	8.66	9.20	9.74	10.28	10.83	11.37	11.91	12.45	12.99	13.53	14.07	14.62	15.16	15.70	16.24	3″
8.29	8.84	9.39	9.94	10.50	11.05	11.60	12.15	12.71	13.26	13.81	14.36	14.91	15.47	16.02	16.57	3⅛″
8.45	9.01	9.58	10.14	10.70	11.27	11.83	12.39	12.96	13.52	14.09	14.65	15.21	15.78	16.34	16.90	3¼″
8.61	9.19	9.76	10.33	10.91	11.48	12.06	12.63	13.20	13.78	14.35	14.93	15.50	16.07	16.65	17.22	3⅜″
8.77	9.35	9.94	10.52	11.11	11.69	12.28	12.86	13.45	14.03	14.62	15.20	15.78	16.37	16.95	17.54	3½″
15″	16″	17″	18″	19″	20″	21″	22″	23″	24″	25″	26″	27″	28″	29″	30″	Dia.

.BL

RIV

C

Beari

3"	
688"	
578	
836	
094	
352	
609	
867	
125	
383	
641	

.BLE X

RIVET T

CLASS

Bearing-stresse

$\frac{1}{8}''$	$\frac{23}{32}''$
.688″	0.719″
578	2.696
836	2.966
094	3.235
352	3.505
609	3.774
867	4.044
125	4.313
383	4.583
641	4.852

$1\frac{1}{8}''$	$1\frac{5}{32}''$	$1\frac{3}{16}''$	[illegible]	$1\frac{1}{4}''$	Diameters.
1.125″	1.156″	1.188″	1.210″	1.250″	
6.750					[illegible]
7.172					$1\frac{1}{16}''$
7.594	7.805	8.016	8.227	8.438	[illegible]

Diameters.	Bending-Moments in Inch Tons	$\frac{1}{4}''$ 0.250"	$\frac{9}{32}''$ 0.281"	$\frac{5}{16}''$ 0.313"	$\frac{11}{32}''$ 0.344"	$\frac{3}{8}''$ 0.375"	$\frac{13}{32}''$ 0.406"	$\frac{7}{16}''$ 0.438"	$\frac{15}{32}''$ 0.469"
[illegible]	0.092	0.750	0.844	0.938	1.032	1.125	1.219	1.313	1.407
[illegible]	0.131	0.844	0.950	1.055	1.161	1.266	1.372	1.477	1.583
[illegible]	0.180	0.938	1.055	1.172	1.289	1.406	1.524	1.641	1.758
[illegible]	0.239	1.032	1.161	1.290	1.419	1.547	1.677	1.805	1.934
[illegible]	0.311	1.125	1.266	1.407	1.548	1.688	1.829	1.969	2.110
[illegible]	0.395	1.219	1.372	1.524	1.677	1.829	1.981	2.133	2.286
[illegible]	0.494	1.312	1.477	1.641	1.805	1.969	2.133	2.297	2.461
[illegible]	0.607	1.406	1.582	1.758	1.934	2.110	2.286	2.461	2.637
1"	0.736	1.500	1.687	1.875	2.063	2.250	2.438	2.625	2.813
$1\frac{1}{16}''$	0.883	1.594	1.793	1.993	2.192	2.391	2.590	2.789	2.989
$1\frac{1}{8}''$	1.049	1.687	1.898	2.110	2.321	2.531	2.742	2.953	3.164

TABLE XX

RIVET TA

CLASS A

Bearing-stresses i

3/8″	13/32″	7/16″	15/32″	1/2″	17/32″	9/16″	19/32″	5/8″	21/32″	11/16″	23/32″	3/4″
0.375″	0.406″	0.438″	0.469″	0.500″	0.531″	0.563″	0.594″	0.625″	0.656″	0.688″	0.719″	0.75
1.125	1.219	1.313	1.407	1.500	1.594	1.688	1.782	1.875				
1.266	1.372	1.477	1.583	1.688	1.794	1.899	2.005	2.110				
1.406	1.524	1.641	1.758	1.875	1.993	2.110	2.227	2.344	2.461	2.578	2.696	2.81
1.547	1.677	1.805	1.934	2.063	2.192	2.321	2.450	2.579	2.708	2.836	2.966	3.09
1.688	1.829	1.969	2.110	2.250	2.391	2.532	2.673	2.813	2.954	3.094	3.235	3.37
1.829	1.981	2.133	2.286	2.438	2.590	2.742	2.895	3.047	3.200	3.352	3.505	3.65
1.969	2.133	2.297	2.461	2.625	2.789	2.952	3.117	3.281	3.445	3.609	3.774	3.93
2.110	2.286	2.461	2.637	2.813	2.989	3.164	3.340	3.516	3.692	3.867	4.044	4.21
2.250	2.438	2.625	2.813	3.000	3.188	3.376	3.563	3.750	3.938	4.125	4.313	4.50
2.391	2.590	2.789	2.989	3.188	3.387	3.587	3.786	3.985	4.184	4.383	4.583	4.78
2.531	2.742	2.953	3.164	3.375	3.586	3.797	4.008	4.219	4.430	4.641	4.852	5.06

BLE

RIVET

ASSES

Bearing-s

"	$\frac{23}{32}$
38"	0.71
23	3.37
46	3.70
68	4.04
90	4.38
1	4.71
34	5.05
56	5.39
79	5.72
01	6.06

3LE X

RIVET TA

ASSES B

Bearing-stresses								
"	$\frac{23}{32}$"	$\frac{3}{4}$	$1\frac{1}{8}$"	$1\frac{5}{32}$"	$1\frac{3}{16}$"	$1\frac{7}{32}$"	$1\frac{1}{4}$"	Diameters.
38"	0.719"	0.7	1.125"	1.156"	1.188"	1.219"	1.250"	
23	3.370	3.5						
40	3.707	3.8						
58	4.044	4.2						
00	4.380	4.5						
1	4.716	4.9						
31	5.054	5.2						
56	5.391	5.6	8.438					1"
79	5.728	5.9	8.966					$1\frac{1}{16}$"
01	6.065	6.3	9.493	9.757	[illegible]	10.284	10.548	[illegible]

TABLE XXXV

RIVET TABLE.

CLASSES B AND C

Diameters.	Bending-moments in inch tons	Bearing-stresses in tons.																
		1/4″	9/32″	5/16″	11/32″	3/8″	13/32″	7/16″	15/32″	1/2″	17/32″	9/16″	19/32″	5/8″	21/32″	11/16″	23/32″	3/4″	25/32″
		0.250″	0.281″	0.313″	0.344″	0.375″	0.406″	0.438″	0.469″	0.500″	0.531″	0.563″	0.594″	0.625″	0.656″	0.688″	0.719″	0.750″	0.781
1/2″	0.115	0.938	1.055	1.172	1.289	1.406	1.523	1.641	1.758	1.875	1.988	2.100	2.222	2.344					
9/16″	0.164	1.055	1.187	1.319	1.451	1.582	1.714	1.846	1.978	2.110	2.240	2.369	2.503	2.637					
5/8″	0.225	1.172	1.319	1.465	1.612	1.758	1.905	2.051	2.198	2.344	2.491	2.638	2.784	2.930	3.077	3.223	3.370	3.516	
11/16″	0.299	1.289	1.451	1.612	1.771	1.929	2.093	2.256	2.418	2.579	2.740	2.902	3.063	3.223	3.585	3.546	3.707	3.868	
3/4″	0.389	1.406	1.583	1.759	1.930	2.100	2.281	2.461	2.637	2.813	2.989	3.165	3.341	3.516	3.692	3.868	4.044	4.219	4.395
13/16″	0.494	1.523	1.715	1.905	2.093	2.281	2.474	2.666	2.857	3.047	3.238	3.428	3.619	3.809	3.999	4.190	4.380	4.570	4.761
7/8″	0.617	1.640	1.846	2.051	2.256	2.461	2.666	2.871	3.076	3.281	3.486	3.691	3.896	4.101	4.306	4.511	4.716	4.921	5.126
15/16″	0.759	1.758	1.978	2.198	2.418	2.637	2.857	3.076	3.296	3.516	3.736	3.956	4.175	4.395	4.614	4.834	5.054	5.273	5.493
1″	0.920	1.875	2.110	2.344	2.579	2.813	3.047	3.281	3.516	3.750	3.985	4.220	4.454	4.688	4.922	5.156	5.391	5.625	5.860
1 1/16″	1.104	1.992	2.242	2.491	2.740	2.989	3.238	3.486	3.736	3.985	4.235	4.484	4.733	4.981	5.230	5.479	5.728	5.977	6.226
1 1/8″	1.311	2.109	2.374	2.638	2.901	3.164	3.428	3.691	3.955	4.219	4.483	4.748	5.011	5.274	5.538	5.801	6.065	6.328	6.592

TABLE XXXVII.

RIVET TABLE.

CLASSES B AND C.

Bearing-stresses in tons.

$\frac{11}{16}''$	$\frac{23}{32}''$	$\frac{3}{4}''$	$\frac{25}{32}''$	$\frac{13}{16}''$	$\frac{27}{32}''$	$\frac{7}{8}''$	$\frac{29}{32}''$	$\frac{15}{16}''$	$\frac{31}{32}''$	$1''$	$1\frac{1}{32}''$	$1\frac{1}{16}''$	$1\frac{3}{32}''$	$1\frac{1}{8}''$	$1\frac{5}{32}''$	$1\frac{3}{16}''$	$1\frac{7}{32}''$	$1\frac{1}{4}''$	Diameters.
0.688″	0.719″	0.750″	0.781″	0.813″	0.844″	0.875″	0.906″	0.938″	0.969″	1.000″	1.031″	1.063″	1.094″	1.125″	1.156″	1.188″	1.219″	1.250″	
																			$\frac{1}{2}''$
																			$\frac{9}{16}''$
3.223	3.370	3.516																	$\frac{5}{8}''$
3.546	3.707	3.868																	$\frac{11}{16}''$
3.868	4.044	4.219	4.395	4.570	4.747	4.923													$\frac{3}{4}''$
4.190	4.380	4.570	4.761	4.951	5.142	5.333													$\frac{13}{16}''$
4.511	4.716	4.921	5.126	5.331	5.537	5.743	5.948	6.153	6.358	6.562									$\frac{7}{8}''$
4.834	5.054	5.273	5.493	5.713	5.933	6.153	6.373	6.592	6.812	7.031									$\frac{15}{16}''$
5.156	5.391	5.625	5.860	6.094	6.329	6.563	6.797	7.031	7.266	7.500	7.735	7.969	8.204	8.438					$1''$
5.479	5.728	5.977	6.226	6.475	6.725	6.973	7.222	7.471	7.720	7.969	8.219	8.336	8.717	8.966					$1\frac{1}{16}''$
5.801	6.065	6.328	6.592	6.856	7.120	7.383	7.647	7.910	8.174	8.438	8.702	8.967	9.229	9.493	9.757	10.020	10.284	10.548	$1\frac{1}{8}''$

Span
40′
50′
60′
70′
80′
90′
100′
110′
120′
130′
[illegible]
[illegible]
[illegible]
[illegible]
[illegible]
[illegible]
[illegible]
210′
[illegible]
[illegible]
240′
250′
[illegible]
[illegible]
[illegible]
[illegible]
300′

Span.	12′ Roadway
40′	17
50′	19
60′	24
70′	30
80′	40
90′	50
100′	63
110′	75
120′	87
130′	104
140′	120
150′	
160′	
170′	
180′	
190′	
200′	
210′	
220′	
230′	
240′	
250′	
260′	
270′	
280′	
290′	
300′	

Span
40′
50′
60′
70′
80′
90′
100′
110′
120′
130′
140′
150′
160′
170′
180′
190′
200′
210′
220′
230′
240′
250′
260′
270′
280′
290′
300′

TABLE XXXVIII.

LABOR IN ERECTION.

Span.	12′ Roadway.	14′ Roadway.	16′ Roadway.	18′ Roadway.	20′ Roadway.	22′ Roadway.	24′ Roadway.
40′	17	18	19	20	21	22	23
50′	19	21	23	25	27	29	31
60′	24	26	28	30	32	34	36
70′	39	41	[illegible]	45	47	49	51
80′	46	49	52	55	58	61	64
90′	56	59	62	65	68	71	74
100′	63	67	71	75	79	83	87
110′	75	80	85	90	95	100	105
120′	87	93	99	105	111	117	123
130′	104	111	118	125	132	139	146
140′	120	134	142	150	158	166	174
150′		160	170	180	190	200	210
160′		191	203	215	227	239	251
170′		222	236	250	264	278	292
180′		253	269	285	301	317	333
190′		284	302	320	338	356	374
200′			335	355	375	395	415
210′			373	395	417	439	461
220′			416	440	464	488	512
230′			463	490	517	544	571
240′			510	540	570	600	630
250′			557	590	623	656	689
260′			614	650	686	722	758
270′				716	[illegible]	788	827
280′				[illegible]	[illegible]	856	900
290′				83[illegible]	[illegible]	922	965
300′				8[illegible]	[illegible]	990	1040

TABLE

TONS OF

king-stress, *A* t

Length	$10\frac{1}{2}''$	$11''$
8'	51.64	57.96
$8\frac{1}{2}'$	50.02	56.27
9'	48.42	54.59
$9\frac{1}{2}'$	46.83	52.90
10'	45.26	51.24
$10\frac{1}{2}'$	43.72	49.59
11'	42.22	47.99
$11\frac{1}{2}'$	40.75	46.42
12'	39.32	44.87
$12\frac{1}{2}'$	37.94	43.37
13'	36.59	41.91
$13\frac{1}{2}'$	35.30	40.49
14'	34.04	39.13
$14\frac{1}{2}'$	32.84	37.79
15'	31.68	36.52
$15\frac{1}{2}'$	30.56	35.28
16'	29.47	34.08
$16\frac{1}{2}'$	28.45	32.93

TABLE XXXIX

TABLE OF WORKING-STRESSES, IN TONS OF 2000 POU

Calculated by the formula $P = \frac{0.625A}{1 + 0.004\frac{L^2}{D^2}}$, where P is equal to the working-stress, A the area in □ i

Length.	3″	3½″	4″	4½″	5″	5½″	6″	6½″	7″	7½″	8″	8½″	9″	9½″	10″	10½″	11″	11½″	12″
8′	1.10	1.31	3.02	4.48	6.31	8.53	11.11	14.10	17.42	21.24	25.38	29.90	34.79	40.05	45.66	51.64	57.96	64.64	71.65
8½′	1.00	1.74	2.78	4.14	5.86	7.95	10.43	13.30	16.56	20.20	24.24	28.65	33.48	38.61	44.13	50.02	56.27	62.87	69.82
9′	0.91	1.59	2.55	3.83	5.44	7.45	9.80	12.55	15.69	19.22	23.13	27.44	32.12	37.18	42.62	48.42	54.59	61.10	67.97
9½′	0.83	1.46	2.35	3.55	5.07	6.95	9.21	11.84	14.86	18.27	22.07	26.26	30.84	35.79	41.12	46.83	52.90	59.33	66.13
10′	0.76	1.34	2.17	3.29	4.73	6.51	8.65	11.17	14.07	17.37	21.05	25.13	29.59	34.45	39.66	45.26	51.24	57.58	64.28
10½′	0.70	1.23	2.01	3.06	4.41	6.10	8.14	10.55	13.34	16.51	20.08	24.03	28.38	33.11	38.23	43.72	49.59	55.84	62.45
11′	0.64	1.14	1.86	2.85	4.13	5.72	7.66	9.97	12.64	15.70	19.15	22.99	27.21	31.83	36.83	42.22	47.99	54.13	60.64
11½′	0.59	1.06	1.74	2.66	3.87	5.38	7.22	9.42	11.99	14.93	18.26	21.98	26.04	30.59	35.48	40.75	46.42	52.45	58.86
12′	0.55	0.99	1.62	2.48	3.62	5.05	6.81	8.91	11.38	14.21	17.42	21.02	25.01	29.39	34.16	39.32	44.87	50.80	57.11
12½′	0.51	0.92	1.51	2.33	3.39	4.75	6.43	8.44	10.79	13.52	16.62	20.11	23.98	28.25	32.89	37.94	43.37	49.19	55.38
13′	0.48	0.85	1.41	2.18	3.19	4.48	6.08	7.99	10.26	12.87	15.86	19.24	22.99	27.14	31.67	36.59	41.91	47.61	53.70
13½′	0.44	0.80	1.32	2.05	3.01	4.23	5.75	7.58	9.74	12.27	15.15	18.41	22.05	26.07	30.49	35.30	40.49	46.08	52.05
14′	0.42	0.75	1.24	1.92	2.83	3.99	5.44	7.19	9.27	11.69	14.47	17.62	21.15	25.06	29.36	34.04	39.12	44.59	50.45
14½′	0.39	0.70	1.17	1.81	2.67	3.78	5.16	6.83	8.82	11.15	13.83	16.88	20.29	24.09	28.27	32.84	37.79	43.14	48.88
15′	0.37	0.66	1.10	1.71	2.53	3.58	4.89	6.49	8.40	10.64	13.22	16.16	19.47	23.16	27.22	31.68	36.52	41.75	47.37
15½′	0.34	0.62	1.04	1.62	2.39	3.39	4.64	6.18	8.01	10.17	12.65	15.49	18.70	22.27	26.26	30.56	35.28	40.39	45.89
16′	0.32	0.59	0.98	1.53	2.27	3.22	4.42	5.88	7.64	9.71	12.11	14.85	17.95	21.41	25.26	29.47	34.08	39.08	44.47
16½′	0.30	0.55	0.93	1.45	2.15	3.06	4.20	5.60	7.29	9.29	11.60	14.24	17.24	20.60	24.34	28.45	32.93	37.82	43.08
17′	0.29	0.52	0.88	1.37	2.04	2.91	3.98	5.35	6.96	8.88	11.11	13.67	16.57	19.83	23.46	27.45	31.84	36.59	41.75
17½′	0.27	0.50	0.83	1.30	1.94	2.77	3.81	5.10	6.65	8.50	10.65	13.12	15.93	19.09	22.61	26.50	30.77	35.41	40.45
18′	0.26	0.47	0.79	1.24	1.85	2.64	3.64	4.87	6.36	8.15	10.21	12.60	15.32	18.38	21.81	25.59	29.75	34.28	39.20
18½′	0.25	0.45	0.75	1.18	1.76	2.52	3.47	4.66	6.10	7.80	9.80	12.11	14.74	17.71	21.04	24.72	28.76	33.19	37.99
19′	0.23	0.43	0.71	1.12	1.68	2.40	3.32	4.46	5.84	7.49	9.41	11.64	14.19	17.07	20.29	23.88	27.82	32.13	36.82
19½′	0.22	0.41	0.68	1.07	1.60	2.29	3.18	4.27	5.60	7.18	9.05	11.20	13.67	16.46	19.59	23.07	26.91	31.12	35.70
20′	0.21	0.39	0.65	1.02	1.53	2.19	3.04	4.09	5.38	6.90	8.70	10.78	13.17	15.88	18.92	22.30	26.04	30.15	34.61
20½′	0.20	0.37	0.62	0.98	1.46	2.10	2.91	3.92	5.16	6.62	8.36	10.38	12.69	15.32	18.27	21.56	25.21	29.20	33.57
21′		0.35	0.59	0.93	1.40	2.01	2.80	3.77	4.95	6.37	8.05	10.00	12.24	14.79	17.65	20.86	24.40	28.30	32.56
21½′		0.34	0.57	0.89	1.34	1.92	2.68	3.62	4.76	6.13	7.75	9.64	11.81	14.28	17.06	20.18	23.63	27.42	31.59
22′		0.32	0.54	0.86	1.29	1.85	2.57	3.47	4.57	5.90	7.47	9.29	11.40	13.79	16.50	19.53	22.88	26.59	30.65
22½′			0.52	0.82	1.24	1.78	2.47	3.34	4.41	5.69	7.20	8.97	11.01	13.33	15.96	18.90	22.18	25.79	29.75
23′			0.50	0.79	1.18	1.71	2.37	3.21	4.24	5.47	6.94	8.65	10.63	12.89	15.44	18.31	21.49	25.02	28.88
23½′			0.48	0.76	1.14	1.64	2.29	3.09	4.09	5.30	6.70	8.36	10.27	12.47	14.95	17.74	20.84	24.37	28.05
24′			0.46	0.73	1.09	1.58	2.20	2.99	3.95	5.10	6.46	8.08	9.93	12.06	14.47	17.19	20.21	23.55	27.24
24½′				0.70	1.05	1.52	2.12	2.87	3.80	4.92	6.25	7.81	9.61	11.67	14.02	16.66	19.60	22.87	26.46
25′				0.67	1.01	1.47	2.05	2.77	3.65	4.75	6.04	7.55	9.30	11.31	13.60	16.15	19.02	22.21	25.71
25½′				0.65	0.98	1.41	1.97	2.68	3.54	4.59	5.84	7.30	9.00	10.95	13.17	15.67	18.47	21.57	24.99
26′					0.94	1.36	1.90	2.58	3.42	4.44	5.65	7.06	8.71	10.61	12.77	15.20	17.93	20.95	24.30
26½′					0.91	1.32	1.84	2.50	3.31	4.29	5.47	6.84	8.44	10.29	12.39	14.76	17.41	20.36	23.63
27′					0.88	1.27	1.78	2.42	3.20	4.15	5.29	6.63	8.18	10.00	12.02	14.33	16.92	19.79	22.99
27½′					0.85	1.23	1.72	2.33	3.10	4.02	5.12	6.42	7.94	9.68	11.67	13.92	16.44	19.25	22.36
28′						1.19	1.66	2.26	3.00	3.89	4.97	6.23	7.69	9.39	11.33	13.52	15.99	18.73	21.76
28½′						1.15	1.61	2.19	2.90	3.77	4.81	6.04	7.47	9.12	11.00	13.14	15.54	18.22	21.18
29′						1.11	1.56	2.12	2.81	3.66	4.67	5.86	7.25	8.85	10.70	12.77	15.12	17.73	20.62
29½′							1.51	2.05	2.72	3.55	4.53	5.69	7.04	8.62	10.39	12.42	14.71	17.25	20.08
30′							1.46	1.99	2.64	3.44	4.40	5.52	6.84	8.36	10.11	12.10	14.30	16.80	19.57
30½′							1.42	1.93	2.57	3.34	4.27	5.37	6.65	8.13	9.83	11.76	13.93	16.36	19.06
31′							1.37	1.87	2.49	3.24	4.15	5.21	6.46	7.91	9.57	11.44	13.58	15.94	18.58
31½′								1.82	2.42	3.15	4.03	5.07	6.28	7.69	9.31	11.14	13.21	15.53	18.11
32′								1.77	2.35	3.06	3.92	4.93	6.11	7.48	9.06	10.84	12.88	15.14	17.67
32½′								1.71	2.28	2.98	3.81	4.79	5.95	7.29	8.82	10.51	12.54	14.76	17.22
33′									2.22	2.89	3.70	4.66	5.79	7.09	8.59	10.30	12.23	14.39	16.80
33½′									2.16	2.81	3.60	4.54	5.64	6.91	8.37	10.04	11.92	14.04	16.40
34′									2.10	2.74	3.51	4.42	5.49	6.73	8.16	9.79	11.63	13.69	16.00
34½′									2.04	2.67	3.42	4.31	5.35	6.56	7.96	9.55	11.34	13.36	15.62
35′										2.60	3.33	4.19	5.21	6.40	7.76	9.31	11.07	13.05	15.25
35½′										2.53	3.24	4.09	5.08	6.24	7.57	9.09	10.80	12.74	14.89
36′										2.46	3.16	3.98	4.97	6.08	7.38	8.87	10.55	12.41	14.55
36½′											3.08	3.90	4.83	5.94	7.21	8.66	10.30	12.15	14.22
37′											3.00	3.79	4.72	5.79	7.03	8.45	10.06	11.87	13.89
37½′											2.93	3.70	4.60	5.65	6.87	8.26	9.83	11.60	13.58
38′											2.86	3.61	4.49	5.52	6.71	8.06	9.60	11.34	13.27
38½′												3.52	4.39	5.39	6.55	7.88	9.39	11.09	13.12
39′												3.44	4.28	5.27	6.40	7.70	9.18	10.84	12.70
39½′												3.36	4.19	5.15	6.26	7.53	8.97	10.60	12.43
40′													4.09	5.03	6.12	7.36	8.78	10.37	12.10

ABLE XXXIX.

ONS OF 2000 POUNDS, FOR SQUARE WOODEN PILLARS.

g-stress, A the area in □ inches, L the length in inches, and D the length of side of square in inenes.

10½"	11"	11½"	12"	12½"	13"	13½"	14"	14½"	15"	15½"	16"	16½"	17"	17½"	18"	18½"	19"	19½"	20"
51.64	57.96	64.04	71.65	79.01	86.71	94.74	103.10	111.81	120.83	130.18	139.86	149.86	160.19	170.85	181.81	193.11	204.72	216.67	228.91
50.02	56.27	62.87	69.82	77.12	84.75	92.73	101.04	109.69	118.69	127.99	137.63	147.60	157.87	168.52	179.45	190.71	202.28	214.22	226.45
48.42	54.59	61.10	67.97	75.20	82.78	90.69	98.95	107.54	116.48	125.74	135.34	145.26	155.52	166.11	177.01	188.24	199.81	211.68	223.89
46.83	52.90	59.33	66.13	73.28	80.78	88.63	96.92	105.36	114.23	123.45	133.00	142.88	153.09	163.63	174.51	185.69	197.22	209.07	221.29
45.26	51.24	57.58	64.28	71.35	78.77	86.55	94.67	103.15	111.96	121.12	130.61	140.45	150.61	161.10	171.94	183.09	194.58	206.40	218.53
43.72	49.59	55.84	62.45	69.44	76.77	84.48	92.52	100.92	109.67	118.76	128.20	137.97	148.09	158.54	169.31	180.43	191.87	203.65	215.76
42.22	47.99	54.13	60.64	67.54	74.78	82.39	90.36	98.69	107.37	116.39	125.77	135.47	145.53	155.93	166.65	177.72	189.12	200.84	212.91
40.75	46.42	52.45	58.86	65.65	72.82	80.33	88.22	96.46	105.06	114.01	123.31	132.97	142.96	153.28	163.95	174.98	186.28	197.99	210.01
39.32	44.87	50.80	57.11	63.79	70.58	78.28	86.06	94.24	102.75	111.62	120.84	130.43	140.34	150.62	161.22	172.19	183.48	195.10	207.07
37.94	43.37	49.19	55.38	61.97	68.92	76.25	83.95	92.02	100.44	109.23	118.35	127.88	137.73	147.97	158.49	169.37	180.60	192.17	204.08
36.59	41.91	47.61	53.70	60.17	67.02	74.25	81.85	89.82	98.16	106.85	115.92	125.35	135.12	145.25	155.72	166.54	177.71	189.22	201.07
35.30	40.49	46.08	52.05	58.41	65.16	72.27	79.75	87.64	95.89	104.50	113.47	122.81	132.50	142.55	152.94	163.69	174.79	186.23	198.03
34.04	39.12	44.59	50.45	56.69	63.32	70.33	77.73	85.50	93.64	102.15	111.03	120.29	129.86	139.84	150.19	160.86	171.88	183.25	194.97
32.84	37.79	43.14	48.88	55.02	61.52	68.43	75.71	83.38	91.42	99.83	108.62	117.77	127.29	137.17	147.40	158.00	168.95	180.25	191.91
31.68	36.52	41.75	47.37	53.39	59.78	66.57	73.74	81.29	89.23	97.54	106.23	115.28	124.70	133.87	144.64	155.16	166.02	177.25	188.82
30.56	35.28	40.39	45.89	51.79	58.07	64.75	71.81	79.25	87.07	95.28	103.86	112.81	122.14	131.84	141.89	152.32	163.13	174.25	185.75
29.47	34.08	39.08	44.47	50.24	56.41	62.96	69.91	77.29	84.95	93.05	101.52	110.35	119.60	129.20	139.16	149.50	160.19	171.26	182.66
28.45	32.93	37.82	43.08	48.74	54.78	61.23	68.06	75.27	82.87	90.85	99.22	107.96	117.09	126.59	136.45	146.69	157.29	168.26	179.06
27.45	31.84	36.59	41.75	47.28	53.21	59.53	66.25	73.34	80.82	88.69	96.96	105.59	114.61	124.01	133.77	143.91	154.17	165.30	176.54
26.50	30.77	35.41	40.45	45.87	51.67	57.88	64.47	71.46	78.83	86.59	94.73	103.25	112.16	121.45	131.12	141.15	151.57	162.34	173.49
25.59	29.75	34.28	39.20	44.50	50.20	56.28	62.76	69.62	76.87	84.51	92.53	100.95	109.75	118.93	128.49	138.43	148.67	159.42	170.47
24.72	28.76	33.19	37.99	43.18	48.76	54.72	61.07	67.81	74.96	82.48	90.40	98.69	107.38	116.44	125.90	135.72	145.93	156.51	167.47
23.88	27.82	32.13	36.82	41.90	47.35	53.20	59.44	66.07	73.08	80.49	88.29	96.47	105.04	114.01	123.34	133.06	143.16	153.64	164.49
23.07	26.91	31.12	35.70	40.16	46.00	51.28	57.85	64.36	71.26	78.55	86.23	94.30	102.75	111.60	120.82	130.44	140.42	150.79	161.55
22.30	26.04	30.15	34.61	39.47	44.69	50.31	56.31	62.69	69.49	76.65	84.21	92.16	100.51	109.23	118.31	127.84	137.72	147.99	158.63
21.56	25.21	29.20	33.57	38.31	43.42	48.92	54.81	61.08	67.74	74.80	82.24	90.07	98.29	106.91	115.91	125.29	135.06	145.21	155.75
20.86	24.40	28.30	32.56	37.19	42.20	47.58	53.35	59.51	66.05	73.00	80.32	88.02	96.13	104.65	113.51	122.78	132.44	142.48	152.90
20.18	23.63	27.42	31.59	36.12	41.01	46.28	51.94	57.98	64.41	71.23	78.43	86.02	94.01	102.41	111.15	120.31	129.85	139.72	150.09
19.53	22.88	26.59	30.65	35.07	39.87	45.03	50.57	56.49	62.81	69.50	76.59	84.07	91.94	100.19	108.85	117.88	127.31	137.12	147.32
18.90	22.18	25.79	29.75	34.07	38.75	43.81	49.24	55.05	61.24	67.83	74.80	82.16	89.91	98.06	106.58	115.50	124.81	134.51	144.59
18.31	21.49	25.02	28.88	33.10	37.68	42.63	47.95	53.65	59.73	66.20	73.05	80.29	87.93	95.95	104.36	113.16	122.35	131.93	141.91
17.74	20.84	24.37	28.05	32.17	36.64	41.49	46.70	52.29	58.26	64.61	71.35	78.48	85.98	93.88	102.18	110.87	119.94	129.41	139.26
17.19	20.21	23.55	27.24	31.27	35.65	40.39	45.49	50.97	56.83	63.07	69.69	76.70	84.09	91.88	100.05	108.64	117.57	126.92	136.11
16.66	19.60	22.87	26.46	30.40	34.67	39.32	44.32	49.69	55.44	61.56	68.07	74.96	82.24	89.91	97.96	106.41	115.25	124.48	134.10
16.15	19.02	22.21	25.71	29.56	33.75	38.28	43.18	48.45	54.10	60.10	66.49	73.27	80.43	87.98	95.92	104.25	112.97	122.07	131.58
15.67	18.47	21.57	24.99	28.75	32.84	37.28	42.08	47.25	52.77	58.68	64.96	71.62	78.67	86.10	93.92	102.14	110.74	119.73	129.11
15.20	17.93	20.95	24.30	27.97	31.97	36.32	41.02	46.08	51.50	57.30	63.47	70.02	76.95	84.26	91.97	100.06	108.54	117.42	126.68
14.76	17.41	20.36	23.63	27.21	31.13	35.38	40.00	44.94	50.27	55.95	62.02	68.45	75.27	82.47	90.06	98.04	106.40	115.16	124.30
14.33	16.92	19.79	22.99	26.48	30.31	34.48	38.98	43.84	49.05	54.64	60.60	66.93	73.64	80.72	88.19	96.06	104.31	112.94	121.96
13.92	16.44	19.25	22.36	25.78	29.52	33.59	38.01	42.78	47.89	53.37	59.23	65.44	72.04	79.02	86.37	94.12	102.25	110.76	119.67
13.52	15.99	18.73	21.76	25.10	28.76	32.75	37.08	41.74	46.76	52.14	57.88	64.00	70.48	77.35	84.59	92.22	100.25	108.64	117.43
13.14	15.54	18.22	21.18	24.45	28.03	31.93	36.16	40.75	45.67	50.95	56.59	62.59	68.97	75.72	82.86	90.37	98.26	106.55	115.23
12.77	15.12	17.73	20.62	23.82	27.32	31.14	35.29	39.77	44.60	49.78	55.32	61.22	67.49	74.14	81.16	88.[illegible]6	96.35	104.51	113.06
12.42	14.71	17.25	20.08	23.21	26.63	30.37	34.43	38.83	43.57	48.65	54.09	59.89	66.06	72.59	79.50	86.79	94.46	102.52	110.95
12.10	14.30	16.80	19.57	22.60	25.97	29.63	33.61	37.92	42.57	47.55	52.89	58.50	64.66	71.08	77.88	85.06	92.62	100.56	108.88
11.76	13.93	16.36	19.06	22.05	25.33	28.91	32.81	37.03	41.59	46.48	51.73	57.33	63.29	69.62	76.30	83.38	90.82	98.65	106.86
11.44	13.58	15.94	18.58	21.49	24.71	28.21	32.03	36.17	40.65	45.45	50.59	56.09	61.95	68.18	74.77	81.72	89.06	96.77	104.87
11.14	13.21	15.53	18.11	20.97	24.12	27.54	31.28	35.34	39.72	44.44	49.49	54.90	60.66	66.79	73.26	80.12	87.34	94.95	102.93
10.84	12.88	15.14	17.67	20.45	23.56	26.89	30.55	34.53	38.84	43.46	48.42	53.74	59.39	65.42	72.26	78.55	85.66	93.16	101.03
10.51	12.54	14.76	17.22	19.95	22.96	26.26	29.85	33.75	37.97	42.50	47.39	52.60	58.17	64.09	70.37	77.01	84.02	91.39	99.17
10.30	12.23	14.39	16.80	19.47	22.42	25.65	29.17	32.99	37.12	41.59	46.37	51.50	56.97	62.79	68.97	75.51	82.42	89.69	97.35
10.04	11.92	14.04	16.40	19.01	21.89	25.05	28.50	32.25	36.31	40.68	45.39	50.42	55.80	61.53	67.61	74.04	80.85	88.02	95.56
9.79	11.63	13.69	16.00	18.56	21.38	24.48	27.85	31.53	35.52	39.81	44.43	49.38	54.66	60.32	66.28	72.62	79.32	86.38	93.82
9.55	11.34	13.36	15.62	18.12	20.88	23.92	27.23	30.84	34.75	38.97	43.50	48.36	53.56	59.10	64.98	71.22	77.83	84.79	92.11
9.31	11.07	13.05	15.25	17.70	20.41	23.38	26.63	30.17	34.00	38.14	42.59	47.37	52.4[illegible]	57.93	63.72	69.87	76.36	83.23	[illegible]
9.09	10.80	12.74	14.89	17.29	19.94	22.86	26.05	29.51	33.28	37.34	41.72	46.41	51.43	56.79	62.48	68.54	74.94	81.69	88.8[illegible]
8.87	10.55	12.44	14.55	16.90	19.49	22.35	25.47	28.88	32.57	36.56	40.85	45.47	50.41	55.63	61.[illegible]	67.24	73.55	80.20	87.22
8.66	10.30	12.15	14.22	16.52	19.06	21.86	24.92	28.26	31.88	35.80	40.00	44.56	49.41	54.50	60.12	65.98	72.21	78.74	85.[illegible]
8.45	10.06	11.87	13.89	16.15	18.64	21.38	24.38	27.66	31.22	35.07	39.21	43.67	[illegible]	53.54	[illegible]	[illegible]	[illegible]	77.32	84.[illegible]
8.26	9.83	11.60	13.58	15.79	18.23	20.92	23.87	27.08	30.57	34.35	38.38	42.80	[illegible]	52.71	[illegible]	[illegible]	[illegible]	75.93	82.64
8.06	9.60	11.34	13.27	15.44	17.84	20.47	23.36	26.51	29.94	33.63	37.60	41.96	46.57	51.51	56.76	62.35	68.28	74.56	81.12
7.88	9.39	11.09	13.12	15.11	17.45	20.04	22.87	25.96	29.33	32.97	36.90	41.14	45.08	50.53	55.70	61.21	[illegible]	73.23	79.76
7.70	9.18	10.84	12.70	14.78	17.08	19.61	22.39	25.43	28.73	32.31	36.18	40.34	44.84	49.58	54.67	60.08	65.[illegible]	71.02	78.36
7.53	8.97	10.60	12.43	14.46	16.71	19.20	21.93	24.91	28.15	31.67	35.47	39.56	43.95	48.64	53.66	58.94	64.00	70.65	77.00
7.36	8.78	10.37	12.16	14.15	16.36	18.80	21.48	24.41	27.59	31.05	34.78	39.03	43.12	47.74	52.07	57.92	[illegible]	69.41	75.6[illegible]

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TABLED AS STRUTS IN LATERAL

Length in feet	4" 10# I Sideways.	4" 10# I Edgeways.	4" 8# I. Sideways.	4" 8# I. Edgeways.	Length of Strut in feet.		
1	13.5	13.5	10.8	10.8	1		
2	12.7	13.2	10.2	10.6	2	1.5	1
4	11.2	13.0	9.0	10.4	4	3.0	2
6	9.2	12.7	7.4	10.2	6	4.5	3
8	7.4	12.5	5.9	10.0	8	6.0	4
10	6.0	12.2	4.8	9.8	10	7.5	5
12	5.0	11.6	4.0	9.3	12	9.0	6
14	4.0	11.0	3.2	8.8	14	10.5	7
16	3.2	10.5	2.5	8.4	16	12.0	8
18	2.5	10.0	2.0	8.0	18	13.5	9
20	2.2	9.5	1.8	7.5	20	15.0	10
22	2.0	8.8	1.6	7.0	22	16.5	11
24	1.7	8.2	1.4	6.6	24	18.0	12
26	1.5	7.7	1.2	6.2	26	19.5	13
28	1.3	7.2	1.0	5.8	28	21.0	14
30	1.1	6.7	0.9	5.3	30	22.5	15
32	0.9	6.3	0.8	5.0	32	24.0	16
34	0.8	6.0	0.7	4.8	34	25.5	17
36	0.8	5.6	0.6	4.5	36	27.0	18
38	0.7	5.2	0.6	4.2	38	28.5	19
40	0.6	5.0	0.5	4.0	40	30.0	20

TABLE OF APPROXIMATE WOR

Length of Strut in feet.			7″ 20# I.		7″ 18# I.	
I	○	○ ○	SIDEWAYS.	EDGEWAYS.	SIDEWAYS.	EDGEWAYS.
1			26.7	27.0	24.2	24.5
2	1.5	1	26.4	26.9	24.0	24.4
4	3.	2	25.0	26.7	22.7	24.3
6	4.5	3	23.0	26.5	21.0	24.2
8	6.0	4	20.5	26.2	18.7	24.0
10	7.5	5	18.2	26.0	16.5	23.7
12	9.0	6	15.7	25.7	14.5	23.5
14	10.5	7	13.7	25.2	12.5	23.1
16	12.0	8	12.0	24.7	11.2	22.7
18	13.5	9	10.5	24.2	9.7	22.2
20	15.0	10	9.0	23.7	8.5	21.7
22	16.5	11	8.0	23.0	7.5	21.2
24	18.0	12	7.0	22.3	6.5	20.7
26	19.5	13	6.2	21.7	5.7	20.2
28	21.0	14	5.5	21.1	5.0	19.5
30	22.5	15	5.0	20.5	4.5	18.7
32	24.0	16	4.5	19.8	4.0	18.0
34	25.5	17	4.0	19.2	3.5	17.5
36	27.0	18	3.5	18.6	3.2	17.0
38	28.5	19	3.2	18.0	3.0	16.5
40	30.0	20	3.0	17.4	2.7	16.0

TABLE XL.

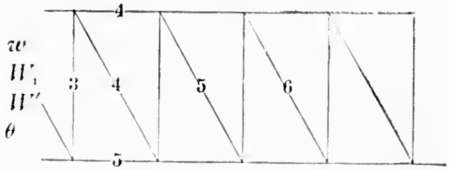
PROXIMATE WORKING LOADS FOR I-BEAMS USED AS STRUTS IN LATERAL SYSTEMS OR SWAY BRACING.

20# I.	7″ 18# I.		6″ 16# I.		6″ 13.5# I.		5″ 12# I.		5″ 10# I.		4″ 10# I.		4″ 8# I.		Length of Strut in feet.		
Edgeways.	Sideways.	Edgeways.	Sideways.	Edgeways.	Sideways.	Edgeways.	Sideways.	Edgeways.	Sideways.	Edgeways.	Sideways.	Edgeways.	Sideways.	Edgeways.			
27.0	24.2	24.5	21.5	21.7	18.2	18.2	16.0	16.0	13.8	13.8	13.5	13.5	10.8	10.8	1		
26.9	24.0	24.4	21.2	21.6	17.7	18.1	15.5	15.9	13.0	13.0	12.7	13.2	10.2	10.6	2	1.5	1
26.7	22.7	24.3	20.0	21.5	16.5	18.0	13.7	15.7	11.1	12.8	11.2	13.0	9.0	10.4	4	3.0	2
26.5	21.0	24.2	18.0	21.2	15.0	17.9	11.7	15.5	9.7	12.6	9.2	12.7	7.4	10.2	6	4.5	3
26.2	18.7	24.0	15.8	21.0	13.2	17.7	9.7	15.2	8.1	12.4	7.4	12.5	5.9	10.0	8	6.0	4
26.0	16.5	23.7	13.7	20.7	11.5	17.5	8.0	14.9	6.7	12.2	6.0	12.2	4.8	9.8	10	7.5	5
25.7	14.5	23.5	11.7	20.2	10.4	17.1	6.5	14.5	5.4	12.0	5.0	11.6	4.0	9.3	12	9.0	6
25.2	12.5	23.1	10.0	19.7	9.2	16.7	5.4	14.0	4.5	11.7	4.0	11.0	3.2	8.8	14	10.5	7
24.7	11.2	22.7	8.7	19.2	8.0	16.2	4.4	13.5	3.7	11.3	3.2	10.5	2.5	8.4	16	12.0	8
24.2	9.7	22.2	7.5	18.7	6.7	15.7	3.6	13.0	3.0	10.8	2.5	10.0	2.0	8.0	18	13.5	9
23.7	8.5	21.7	6.5	18.1	5.5	15.2	3.0	12.5	2.5	10.4	2.2	9.5	1.8	7.5	20	15.0	10
23.0	7.5	21.2	5.7	17.5	5.0	14.7	2.6	12.0	2.2	10.0	2.0	8.8	1.6	7.0	22	16.5	11
22.3	6.5	20.7	5.0	17.0	4.5	14.2	2.3	11.5	2.0	9.6	1.7	8.2	1.4	6.6	24	18.0	12
21.7	5.7	20.2	4.2	16.2	4.0	13.7	2.0	11.0	1.8	9.2	1.5	7.7	1.2	6.2	26	19.5	13
21.1	5.0	19.5	3.7	15.5	3.5	13.2	1.8	10.4	1.6	8.7	1.3	7.2	1.0	5.8	28	21.0	14
20.5	4.5	18.7	3.5	15.0	3.1	12.7	1.6	9.7	1.5	8.1	1.1	6.7	0.9	5.3	30	22.5	15
19.8	4.0	18.0	3.0	14.5	2.7	12.2	1.4	9.0	1.3	7.5	0.9	6.3	0.8	5.0	32	24.0	16
19.2	3.5	17.5	2.7	14.0	2.5	11.7	1.2	8.2	1.1	6.8	0.8	6.0	0.7	4.8	34	25.5	17
18.6	3.2	17.0	2.5	13.5	2.2	11.2	1.1	7.5	1.0	6.2	0.8	5.6	0.6	4.5	36	27.0	18
18.0	3.0	16.5	2.2	13.0	2.0	10.7	1.0	6.8	0.9	5.7	0.7	5.2	0.6	4.2	38	28.5	19
17.4	2.7	16.0	2.0	12.5	1.7	10.2	0.9	6.1	0.8	5.1	0.6	5.0	0.5	4.0	40	30.0	20

ME

Top Chor
" "
" "
" "
Bottom C
"
"
"
"
Batter Bra
Diagonal
"
"
"
"
"
Post (Thro
" "
" "
Post (Deck
" "
" "

'RUSSES.



MEMI	8 Panel.		9 Panel.		Multiply by
	w	W_1	w	W_1	
Top Chord	6	6	7	7	tan θ
" "	$7\frac{1}{2}$	$7\frac{1}{2}$	9	9	
" "	8	8	10	10	
" "			10	10	
Bottom Chor	$3\frac{1}{2}$	$3\frac{1}{2}$	4	4	
" "	$3\frac{1}{2}$	$3\frac{1}{2}$	4	4	
" "	6	6	7	7	
" "	$7\frac{1}{2}$	$7\frac{1}{2}$	9	9	
" "			10	10	
Batter Brace	$\frac{28}{8}$	$3\frac{1}{2}$	$\frac{36}{9}$	4	sec θ.
Diagonal .	$\frac{21}{8}$	$2\frac{1}{2}$	$\frac{28}{9}$	3	
" .	$\frac{15}{8}$	$1\frac{1}{2}$	$\frac{21}{9}$	2	
" .	$\frac{10}{8}$	$\frac{1}{2}$	$\frac{15}{9}$	1	
" .	$\frac{6}{8}$	$-\frac{1}{2}$	$\frac{10}{9}$	0	
" .	$\frac{3}{8}$	$-1\frac{1}{2}$	$\frac{6}{9}$	-1	
" .			$\frac{3}{9}$	-2	
Post (Throu	$\frac{15}{8}$	$1\frac{1}{2}$	$\frac{21}{9}$	2	To the stress on each post must be added W''.
" "	$\frac{10}{8}$	$\frac{1}{2}$	$\frac{15}{9}$	1	
" "	$\frac{6}{8}$	$-\frac{1}{2}$	$\frac{10}{9}$	0	
Post (Deck I	$\frac{21}{8}$	$1\frac{1}{2}$	$\frac{28}{9}$	2	
" "	$\frac{15}{8}$	$\frac{1}{2}$	$\frac{21}{9}$	1	
" "	$\frac{10}{8}$	$-\frac{1}{2}$	$\frac{15}{9}$	0	

STRESSES IN SI

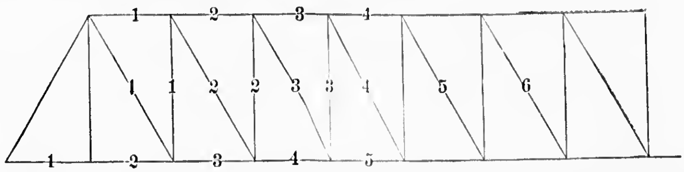
w = panel live load on one truss,
W_1 = panel dead load on one truss,
W'' = upper panel dead load on one truss,
θ = inclination of diagonal to vertical.

MEMBER.	3 Panel.		4 Panel.		5
	w	W_1	w	W_1	w
Top Chord 1	1	1	2	2	3
" " 2					3
" " 3					
" " 4					
Bottom Chord 1	1	1	$1\frac{1}{2}$	$1\frac{1}{2}$	2
" " 2	1	1	$1\frac{1}{2}$	$1\frac{1}{2}$	2
" " 3					3
" " 4					
" " 5					
Batter Brace	$\frac{4}{3}$	1	$\frac{6}{4}$	$1\frac{1}{2}$	$\frac{10}{5}$
Diagonal 1	$\frac{1}{3}$	0	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{6}{5}$
" 2			$\frac{1}{4}$	$-\frac{1}{2}$	$\frac{3}{5}$
" 3					$\frac{1}{5}$
" 4					
" 5					
" 6					
Post (Through-Bridge) 1			$\frac{1}{4}$	$-\frac{1}{2}$	$\frac{3}{5}$
" " " . 2					
" " " . 3					
Post (Deck-Bridge) . 1			$\frac{7}{4}$	$-\frac{1}{2}$	$\frac{3}{5}$
" " " . 2					
" " " . 3					

TABLE XLI.

STRESSES IN SINGLE-INTERSECTION TRUSSES.

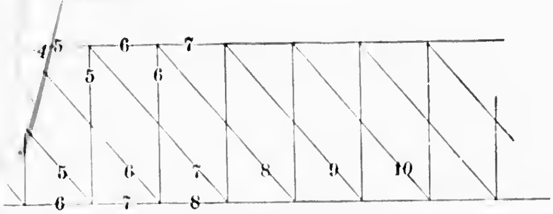
l on one truss,
d on one truss,
ead load on one truss,
diagonal to vertical.



anel.	4 Panel.		5 Panel.		6 Panel.		7 Panel.		8 Panel.		9 Panel.		Multiply by
W_1	w	W_1	w	W_1	w	W_1	w	W_1	w	W_1	w	W_1	
1	2	2	3	3	4	4	5	5	6	6	7	7	
			3	3	$4\frac{1}{2}$	$4\frac{1}{2}$	6	6	$7\frac{1}{2}$	$7\frac{1}{2}$	9	9	
							6	6	8	8	10	10	
											10	10	
1	$1\frac{1}{2}$	$1\frac{1}{2}$	2	2	$2\frac{1}{2}$	$2\frac{1}{2}$	3	3	$3\frac{1}{2}$	$3\frac{1}{2}$	4	4	$\tan\theta$
1	$1\frac{1}{2}$	$1\frac{1}{2}$	2	2	$2\frac{1}{2}$	$2\frac{1}{2}$	3	3	$3\frac{1}{2}$	$3\frac{1}{2}$	4	4	
			3	3	4	4	5	5	6	6	7	7	
							6	6	$7\frac{1}{2}$	$7\frac{1}{2}$	9	9	
											10	10	
1	$\frac{6}{4}$	$1\frac{1}{2}$	$\frac{10}{5}$	2	$\frac{15}{6}$	$2\frac{1}{2}$	$\frac{21}{7}$	3	$\frac{28}{8}$	$3\frac{1}{2}$	$\frac{36}{9}$	4	
0	$\frac{3}{4}$	$\frac{1}{2}$	$\frac{6}{5}$	1	$\frac{10}{6}$	$1\frac{1}{2}$	$\frac{15}{7}$	2	$\frac{21}{8}$	$2\frac{1}{2}$	$\frac{28}{9}$	3	
	$\frac{1}{4}$	$-\frac{1}{2}$	$\frac{3}{5}$	0	$\frac{6}{6}$	$\frac{1}{2}$	$\frac{10}{7}$	1	$\frac{15}{8}$	$1\frac{1}{2}$	$\frac{21}{9}$	2	
			$\frac{1}{5}$	-1	$\frac{3}{6}$	$-\frac{1}{2}$	$\frac{6}{7}$	0	$\frac{10}{8}$	$\frac{1}{2}$	$\frac{15}{9}$	1	$\sec\theta$.
							$\frac{3}{7}$	-1	$\frac{6}{8}$	$-\frac{1}{2}$	$\frac{10}{9}$	0	
									$\frac{3}{8}$	$-1\frac{1}{2}$	$\frac{6}{9}$	-1	
											$\frac{3}{9}$	-2	
	$\frac{1}{4}$	$-\frac{1}{2}$	$\frac{3}{5}$	0	$\frac{6}{6}$	$\frac{1}{2}$	$\frac{10}{7}$	1	$\frac{15}{8}$	$1\frac{1}{2}$	$\frac{21}{9}$	2	
					$\frac{3}{6}$	$-\frac{1}{2}$	$\frac{6}{7}$	0	$\frac{10}{8}$	$\frac{1}{2}$	$\frac{15}{9}$	1	
									$\frac{6}{8}$	$-\frac{1}{2}$	$\frac{10}{9}$	0	To the stress on each post must be added W'.
	$\frac{3}{4}$	$-\frac{1}{2}$	$\frac{5}{5}$	0	$\frac{12}{6}$	$\frac{1}{2}$	$\frac{17}{7}$	1	$\frac{23}{8}$	$1\frac{1}{2}$	$\frac{30}{9}$	2	
					$\frac{9}{6}$	$-\frac{1}{2}$	$\frac{13}{7}$	0	$\frac{18}{8}$	$\frac{1}{2}$	$\frac{24}{9}$	1	
									$\frac{14}{8}$	$-\frac{1}{2}$	$\frac{19}{9}$	0	

13

USSES.



13 Panel.		14 Panel.		15 Panel.		Multiply by
w	W_1	w	W_1	w	W_1	
$\frac{172}{13}$	$\frac{172}{13}$	$14\frac{1}{2}$	$14\frac{1}{2}$	$\frac{238}{15}$	$\frac{238}{15}$	
$\frac{218}{13}$	$\frac{218}{13}$	$18\frac{1}{2}$	$18\frac{1}{2}$	$\frac{304}{15}$	$\frac{304}{15}$	
$\frac{250}{13}$	$\frac{250}{13}$	$21\frac{1}{2}$	$21\frac{1}{2}$	$\frac{358}{15}$	$\frac{358}{15}$	
$\frac{270}{13}$	$\frac{270}{13}$	$23\frac{1}{2}$	$23\frac{1}{2}$	$\frac{394}{15}$	$\frac{394}{15}$	
$\frac{276}{13}$	$\frac{276}{13}$	$24\frac{1}{2}$	$24\frac{1}{2}$	$\frac{418}{15}$	$\frac{418}{15}$	
$\frac{276}{13}$	$\frac{276}{13}$	$24\frac{1}{2}$	$24\frac{1}{2}$	$\frac{424}{15}$	$\frac{424}{15}$	
				$\frac{424}{15}$	$\frac{424}{15}$	
$\frac{78}{13}$	$\frac{78}{13}$	$6\frac{1}{2}$	$6\frac{1}{2}$	$\frac{105}{15}$	$\frac{105}{15}$	tan a.
$\frac{78}{13}$	$\frac{78}{13}$	$6\frac{1}{2}$	$6\frac{1}{2}$	$\frac{105}{15}$	$\frac{105}{15}$	
$\frac{114}{13}$	$\frac{114}{13}$	$9\frac{1}{2}$	$9\frac{1}{2}$	$\frac{154}{15}$	$\frac{154}{15}$	
$\frac{172}{13}$	$\frac{172}{13}$	$14\frac{1}{2}$	$14\frac{1}{2}$	$\frac{238}{15}$	$\frac{238}{15}$	
$\frac{218}{13}$	$\frac{218}{13}$	$18\frac{1}{2}$	$18\frac{1}{2}$	$\frac{304}{15}$	$\frac{304}{15}$	
$\frac{250}{13}$	$\frac{250}{13}$	$21\frac{1}{2}$	[illegible]	358	358	

STRESSES IN D

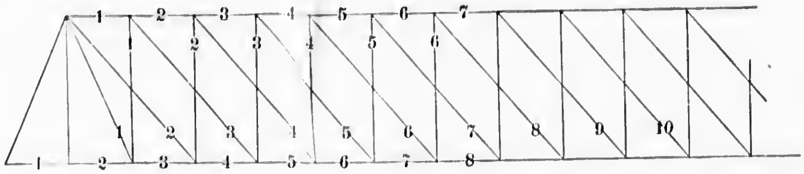
w = panel live load on one truss,
W_1 = panel dead load on one truss,
W'' = upper panel dead load on one truss,
α = inclination of short diagonal to vertical,
β = inclination of long diagonal to vertical.

MEMBER.	7 Panel.		8 Panel.		9 Panel.	
	w	W_1	w	W_1	w	W_1
Top Chord 1	$\frac{40}{7}$	$\frac{40}{7}$	7	7	[illegible]	$\frac{71}{9}$
" " 2	$\frac{44}{7}$	$\frac{44}{7}$	8	8	[illegible]	$\frac{88}{9}$
" " 3	$\frac{44}{7}$	$\frac{44}{7}$	8	8	$\frac{92}{9}$	$\frac{92}{9}$
" " 4					$\frac{92}{9}$	$\frac{92}{9}$
" " 5						
" " 6						
" " 7						
Bottom Chord 1	$\frac{21}{7}$	$\frac{21}{7}$	$3\frac{1}{2}$	$3\frac{1}{2}$	[illegible]	$\frac{36}{9}$
" " 2	$\frac{21}{7}$	$\frac{21}{7}$	$3\frac{1}{2}$	$3\frac{1}{2}$	[illegible]	$\frac{36}{9}$
" " 3	$\frac{30}{7}$	$\frac{30}{7}$	5	5	$\frac{52}{9}$	$\frac{52}{9}$
" " 4	$\frac{36}{7}$	$\frac{36}{7}$	7	7	$\frac{71}{9}$	$\frac{71}{9}$
" " 5					[illegible]	$\frac{84}{9}$
" " 6						
" " 7						
" " 8						
Batter Brace	$\frac{21}{7}$	$\frac{21}{7}$	$\frac{28}{8}$	$3\frac{1}{2}$	$\frac{36}{9}$	$\frac{36}{9}$
Diagonal 1	$\frac{9}{7}$	$\frac{9}{7}$	$\frac{12}{8}$	$1\frac{1}{2}$	$\frac{16}{9}$	$\frac{16}{9}$
" 2	$\frac{6}{7}$	$\frac{5}{7}$	$\frac{9}{8}$	1	$\frac{12}{9}$	$\frac{11}{9}$
" 3	$\frac{4}{7}$	$\frac{5}{7}$	$\frac{6}{8}$	$\frac{1}{2}$	[illegible]	$\frac{7}{9}$
" 4	$\frac{2}{7}$	$-\frac{2}{7}$	$\frac{4}{8}$	o	$\frac{6}{9}$	$\frac{2}{9}$
" 5			$\frac{2}{8}$	$-\frac{1}{2}$	$\frac{4}{9}$	$-\frac{2}{9}$
" 6					[illegible]	$-\frac{7}{9}$
" 7						
" 8						
" 9						
" 10						
Post (Through Bridge). 1	$\frac{4}{7}$	$\frac{2}{7}$	$\frac{6}{8}$	$\frac{1}{2}$	$\frac{9}{9}$	$\frac{7}{9}$
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	$\frac{3}{9}$	$\frac{2}{9}$
" . 3			$\frac{2}{8}$	$-\frac{1}{2}$	$\frac{1}{9}$	$-\frac{2}{9}$
" " " 4						
" " " 5						
" " " 6						
Post (Deck Bridge) . 1	$\frac{11}{7}$	$\frac{2}{7}$	$\frac{14}{8}$	$\frac{1}{2}$	$\frac{18}{9}$	$\frac{7}{9}$
" " " . 2	$\frac{9}{7}$	$-\frac{2}{7}$	$\frac{12}{8}$	o	$\frac{15}{9}$	$\frac{2}{9}$
" " " . 3			$\frac{10}{8}$	$-\frac{1}{2}$	$\frac{13}{9}$	$-\frac{2}{9}$
" " " . 4						
" " " . 5						
" " " . 6						

TABLE XLII.

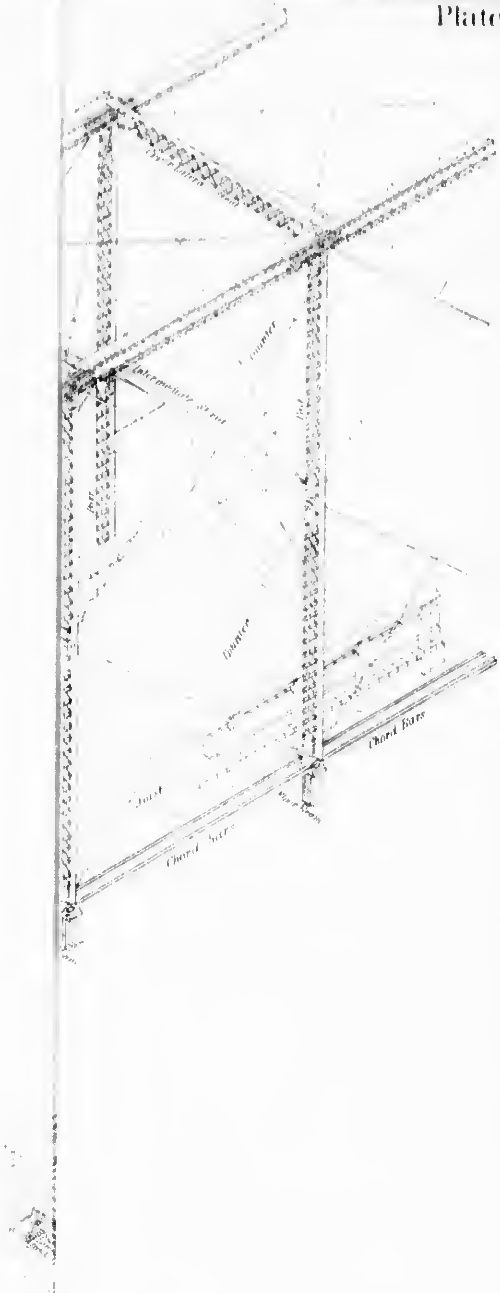
STRESSES IN DOUBLE-INTERSECTION TRUSSES.

truss,
truss,
on one truss,
gonal to vertical,
gonal to vertical.



8 Panel.		9 Panel.		10 Panel.		11 Panel.		12 Panel.		13 Panel.		14 Panel.		15 Panel.		Multiply by
w	W_1	w	W_1	w	W_1	w	W_1	w	W_1	w	W_1	w	W_1	w	W_1	
	7	$\frac{71}{9}$	$\frac{71}{9}$	$9\frac{1}{2}$	$9\frac{1}{2}$	$\frac{118}{11}$	$\frac{118}{11}$	12	12	$\frac{172}{13}$	$\frac{172}{13}$	$14\frac{1}{2}$	$14\frac{1}{2}$	$\frac{236}{15}$	$\frac{236}{15}$	tan a.
	8	[illegible]	[illegible]	$11\frac{1}{2}$	$11\frac{1}{2}$	$\frac{146}{11}$	$\frac{146}{11}$	15	15	$\frac{218}{13}$	$\frac{218}{13}$	$18\frac{1}{2}$	$18\frac{1}{2}$	$\frac{304}{15}$	$\frac{304}{15}$	
	8	$\frac{92}{9}$	$\frac{92}{9}$	$12\frac{1}{2}$	$12\frac{1}{2}$	$\frac{162}{11}$	$\frac{162}{11}$	17	17	$\frac{250}{13}$	$\frac{250}{13}$	$21\frac{1}{2}$	$21\frac{1}{2}$	$\frac{356}{15}$	$\frac{356}{15}$	
		$\frac{92}{9}$	$\frac{92}{9}$	$12\frac{1}{2}$	$12\frac{1}{2}$	$\frac{168}{11}$	$\frac{168}{11}$	18	18	$\frac{270}{13}$	$\frac{270}{13}$	$23\frac{1}{2}$	$23\frac{1}{2}$	$\frac{394}{15}$	$\frac{394}{15}$	
						$\frac{168}{11}$	$\frac{168}{11}$	18	18	$\frac{276}{13}$	$\frac{276}{13}$	$24\frac{1}{2}$	$24\frac{1}{2}$	$\frac{418}{15}$	$\frac{418}{15}$	
										$\frac{276}{13}$	$\frac{276}{13}$	$24\frac{1}{2}$	$24\frac{1}{2}$	$\frac{424}{15}$	$\frac{424}{15}$	
														$\frac{424}{15}$	$\frac{424}{15}$	
$3\frac{1}{2}$	$3\frac{1}{2}$	$\frac{36}{9}$	$\frac{36}{9}$	$4\frac{1}{2}$	$4\frac{1}{2}$	$\frac{55}{11}$	$\frac{55}{11}$	$5\frac{1}{2}$	$5\frac{1}{2}$	$\frac{78}{13}$	$\frac{78}{13}$	$6\frac{1}{2}$	$6\frac{1}{2}$	$\frac{105}{15}$	$\frac{105}{15}$	
$3\frac{1}{2}$	$3\frac{1}{2}$	$\frac{36}{9}$	$\frac{36}{9}$	$4\frac{1}{2}$	$4\frac{1}{2}$	$\frac{55}{11}$	$\frac{55}{11}$	$5\frac{1}{2}$	$5\frac{1}{2}$	$\frac{78}{13}$	$\frac{78}{13}$	$6\frac{1}{2}$	$6\frac{1}{2}$	$\frac{105}{15}$	$\frac{105}{15}$	
5	5	$\frac{52}{9}$	$\frac{52}{9}$	$6\frac{1}{2}$	$6\frac{1}{2}$	$\frac{80}{11}$	$\frac{80}{11}$	8	8	$\frac{114}{13}$	$\frac{114}{13}$	$9\frac{1}{2}$	$9\frac{1}{2}$	$\frac{154}{15}$	$\frac{154}{15}$	
[illegible]	7	$\frac{71}{9}$	$\frac{71}{9}$	$9\frac{1}{2}$	$9\frac{1}{2}$	$\frac{118}{11}$	$\frac{118}{11}$	12	12	$\frac{172}{13}$	$\frac{172}{13}$	$14\frac{1}{2}$	$14\frac{1}{2}$	$\frac{236}{15}$	$\frac{236}{15}$	
		[illegible]	[illegible]	$11\frac{1}{2}$	$11\frac{1}{2}$	$\frac{146}{11}$	$\frac{146}{11}$	15	15	$\frac{218}{13}$	$\frac{218}{13}$	$18\frac{1}{2}$	$18\frac{1}{2}$	$\frac{304}{15}$	$\frac{304}{15}$	
						$\frac{156}{11}$	$\frac{156}{11}$	17	17	$\frac{250}{13}$	$\frac{250}{13}$	$21\frac{1}{2}$	$21\frac{1}{2}$	$\frac{356}{15}$	$\frac{356}{15}$	
										$\frac{264}{13}$	$\frac{264}{13}$	$23\frac{1}{2}$	$23\frac{1}{2}$	$\frac{394}{15}$	$\frac{394}{15}$	
														$\frac{408}{15}$	$\frac{408}{15}$	
[illegible]	$3\frac{1}{2}$	[illegible]	$\frac{36}{9}$	$\frac{45}{10}$	$4\frac{1}{2}$	$\frac{55}{11}$	$\frac{55}{11}$	$\frac{66}{12}$	$5\frac{1}{2}$	$\frac{78}{13}$	$\frac{78}{13}$	$\frac{91}{14}$	$6\frac{1}{2}$	$\frac{105}{15}$	$\frac{105}{15}$	sec a.
$\frac{12}{8}$	$1\frac{1}{2}$	[illegible]	$\frac{16}{9}$	$\frac{20}{10}$	2	$\frac{25}{11}$	$\frac{25}{11}$	$\frac{30}{12}$	$2\frac{1}{2}$	$\frac{36}{13}$	$\frac{36}{13}$	$\frac{42}{14}$	3	$\frac{49}{15}$	$\frac{49}{15}$	
[illegible]	1	[illegible]	$\frac{11}{9}$	$\frac{16}{10}$	$1\frac{1}{2}$	$\frac{20}{11}$	$\frac{19}{11}$	$\frac{25}{12}$	2	$\frac{30}{13}$	$\frac{29}{13}$	$\frac{36}{14}$	$2\frac{1}{2}$	$\frac{42}{15}$	$\frac{41}{15}$	sec β.
[illegible]	$\frac{1}{2}$	[illegible]	$\frac{7}{9}$	$\frac{12}{10}$	1	$\frac{16}{11}$	$\frac{14}{11}$	$\frac{20}{12}$	$1\frac{1}{2}$	$\frac{25}{13}$	$\frac{23}{13}$	$\frac{30}{14}$	2	$\frac{36}{15}$	$\frac{34}{15}$	
[illegible]	0	[illegible]	$\frac{2}{9}$	$\frac{9}{10}$	$\frac{1}{2}$	$\frac{12}{11}$	$\frac{8}{11}$	$\frac{16}{12}$	1	$\frac{20}{13}$	$\frac{17}{13}$	$\frac{25}{14}$	$1\frac{1}{2}$	$\frac{30}{15}$	$\frac{26}{15}$	
[illegible]	$-\frac{1}{2}$	$\frac{4}{9}$	$-\frac{2}{9}$	$\frac{6}{10}$	0	$\frac{9}{11}$	$\frac{3}{11}$	$\frac{12}{12}$	$\frac{1}{2}$	$\frac{16}{13}$	$\frac{10}{13}$	$\frac{20}{14}$	1	$\frac{25}{15}$	$\frac{19}{15}$	
		[illegible]	$-\frac{7}{9}$	$\frac{1}{10}$	$-\frac{1}{2}$	$\frac{6}{11}$	$-\frac{3}{11}$	$\frac{9}{12}$	0	$\frac{12}{13}$	$\frac{3}{13}$	$\frac{16}{14}$	$\frac{1}{2}$	$\frac{20}{15}$	$\frac{11}{15}$	
				$\frac{2}{10}$	-1	$\frac{1}{11}$	$-\frac{8}{11}$	$\frac{6}{12}$	$-\frac{1}{2}$	$\frac{9}{13}$	$-\frac{3}{13}$	$\frac{12}{14}$	0	$\frac{16}{15}$	$\frac{4}{15}$	
								$\frac{4}{12}$	-1	$\frac{6}{13}$	$-\frac{10}{13}$	$\frac{9}{14}$	$-\frac{1}{2}$	$\frac{12}{15}$	$-\frac{4}{15}$	
										$\frac{4}{13}$	$-\frac{16}{13}$	$\frac{6}{14}$	-1	$\frac{9}{15}$	$-\frac{11}{15}$	
														$\frac{6}{15}$	$-\frac{19}{15}$	
$\frac{6}{8}$	$\frac{1}{2}$	$\frac{9}{9}$	$\frac{7}{9}$	$\frac{12}{10}$	1	$\frac{16}{11}$	$\frac{14}{11}$	$\frac{20}{12}$	$1\frac{1}{2}$	$\frac{25}{13}$	$\frac{23}{13}$	$\frac{30}{14}$	2	$\frac{36}{15}$	$\frac{34}{15}$	To the stress on each post must be added W'.
[illegible]	[illegible]	[illegible]	[illegible]	$\frac{9}{10}$	$\frac{1}{2}$	$\frac{12}{11}$	$\frac{8}{11}$	$\frac{16}{12}$	1	$\frac{20}{13}$	$\frac{17}{13}$	$\frac{25}{14}$	$1\frac{1}{2}$	$\frac{30}{15}$	$\frac{26}{15}$	
$\frac{2}{8}$	$-\frac{1}{2}$	$\frac{4}{9}$	$-\frac{2}{9}$	$\frac{6}{10}$	0	$\frac{9}{11}$	$\frac{3}{11}$	$\frac{12}{12}$	$\frac{1}{2}$	$\frac{16}{13}$	$\frac{10}{13}$	$\frac{20}{14}$	1	$\frac{25}{15}$	$\frac{19}{15}$	
				$\frac{1}{10}$	$-\frac{1}{2}$	$\frac{6}{11}$	$-\frac{3}{11}$	$\frac{9}{12}$	0	$\frac{12}{13}$	$\frac{3}{13}$	$\frac{16}{14}$	$\frac{1}{2}$	$\frac{20}{15}$	$\frac{11}{15}$	
								$\frac{6}{12}$	$-\frac{1}{2}$	$\frac{9}{13}$	$-\frac{3}{13}$	$\frac{12}{14}$	0	$\frac{16}{15}$	$\frac{4}{15}$	
												$\frac{9}{14}$	$-\frac{1}{2}$	$\frac{12}{15}$	$-\frac{4}{15}$	
$\frac{14}{8}$	$\frac{1}{2}$	$\frac{19}{9}$	$\frac{7}{9}$	$\frac{22}{10}$	1	$\frac{27}{11}$	$\frac{14}{11}$	$\frac{32}{12}$	$1\frac{1}{2}$	$\frac{38}{13}$	$\frac{23}{13}$	$\frac{44}{14}$	2	$\frac{51}{15}$	$\frac{34}{15}$	
$\frac{12}{8}$	0	$\frac{15}{9}$	$\frac{2}{9}$	$\frac{19}{10}$	$\frac{1}{2}$	$\frac{24}{11}$	$\frac{8}{11}$	$\frac{28}{12}$	1	$\frac{33}{13}$	$\frac{17}{13}$	$\frac{39}{14}$	$1\frac{1}{2}$	$\frac{45}{15}$	$\frac{26}{15}$	
$\frac{10}{8}$	$-\frac{1}{2}$	$\frac{13}{9}$	$-\frac{2}{9}$	$\frac{16}{10}$	0	$\frac{20}{11}$	$\frac{3}{11}$	$\frac{24}{12}$	$\frac{1}{2}$	$\frac{29}{13}$	$\frac{10}{13}$	$\frac{34}{14}$	1	$\frac{40}{15}$	$\frac{19}{15}$	
				$\frac{14}{10}$	$-\frac{1}{2}$	$\frac{17}{11}$	$-\frac{3}{11}$	$\frac{21}{12}$	0	$\frac{25}{13}$	$\frac{3}{13}$	$\frac{30}{14}$	$\frac{1}{2}$	$\frac{35}{15}$	$\frac{11}{15}$	
								$\frac{18}{12}$	$-\frac{1}{2}$	$\frac{22}{13}$	$-\frac{3}{13}$	$\frac{26}{14}$	0	$\frac{31}{15}$	$\frac{4}{15}$	
												$\frac{23}{14}$	$-\frac{1}{2}$	$\frac{27}{15}$	$-\frac{4}{15}$	

Plate I.



DOUBLE INTER

WROUGHT IRON HIG

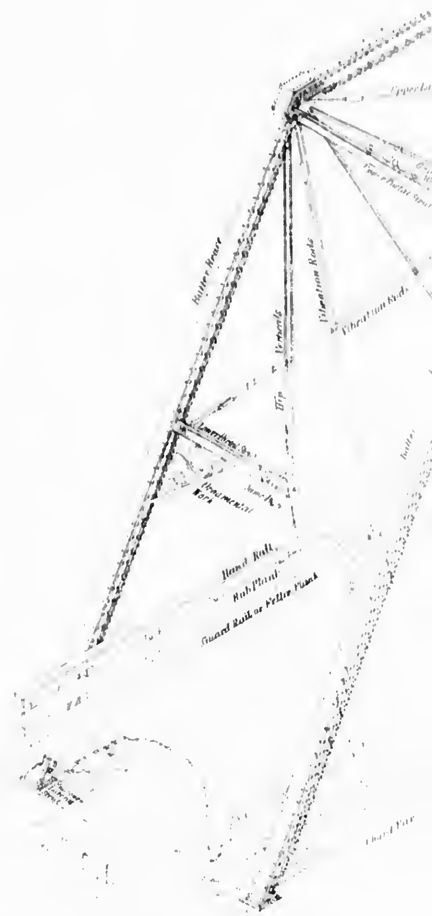


Plate I.

DOUBLE INTERSECTION
OUGHT IRON HIGHWAY BRIDGE.

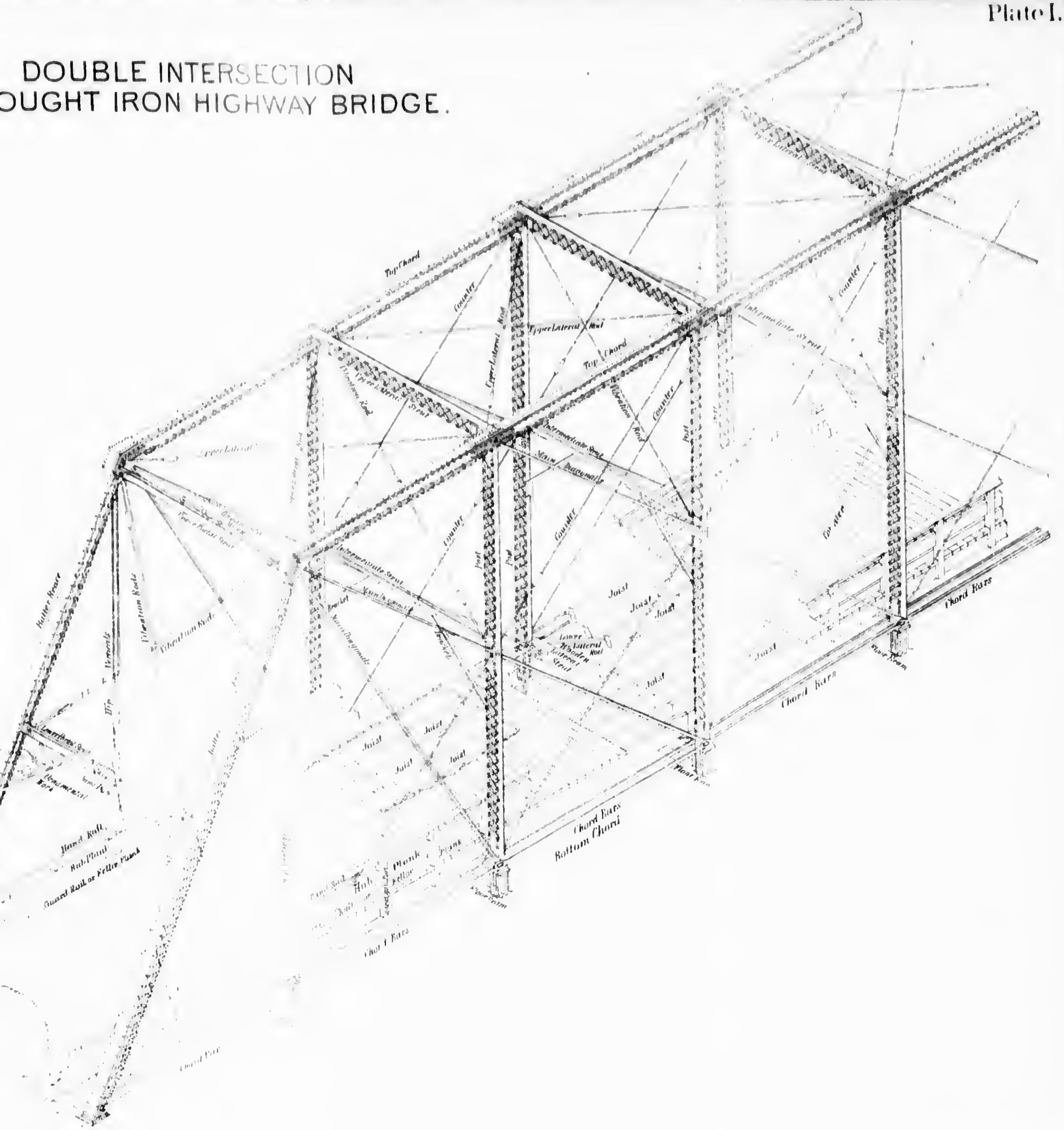


Plate II

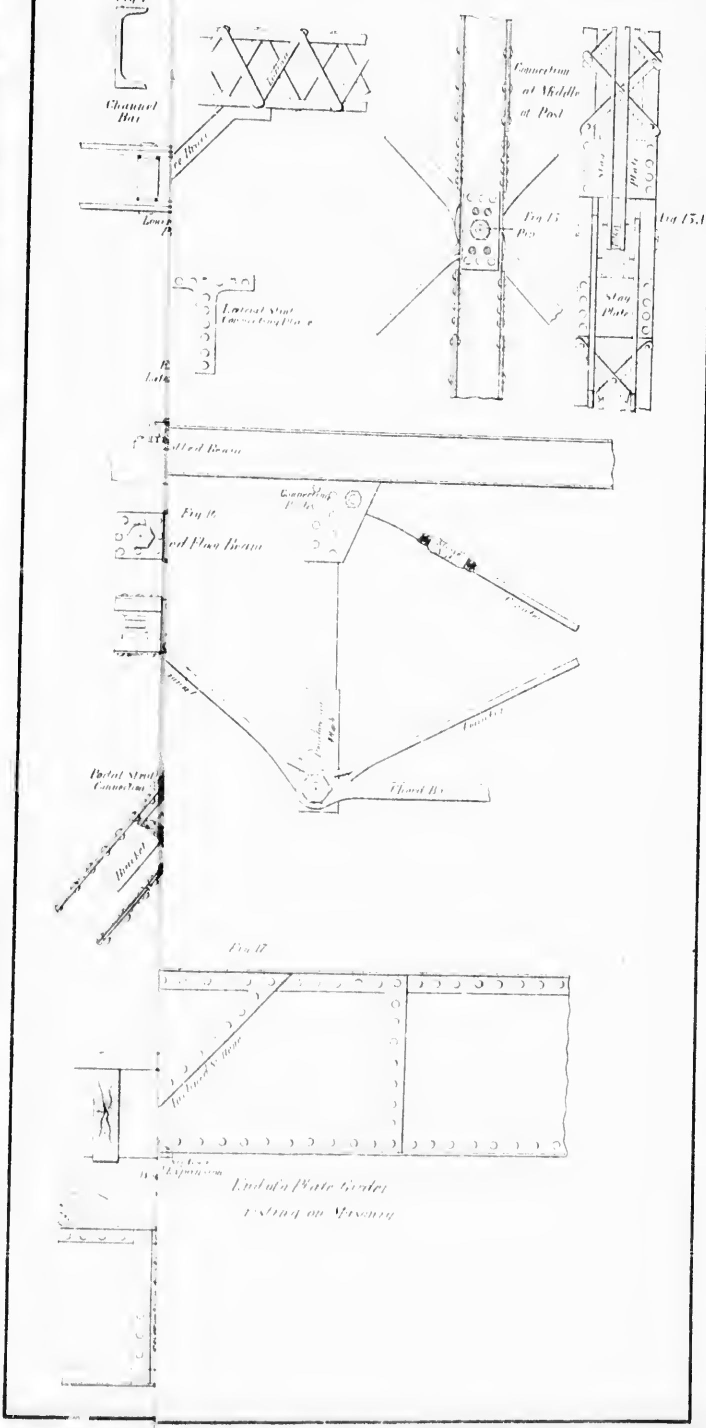
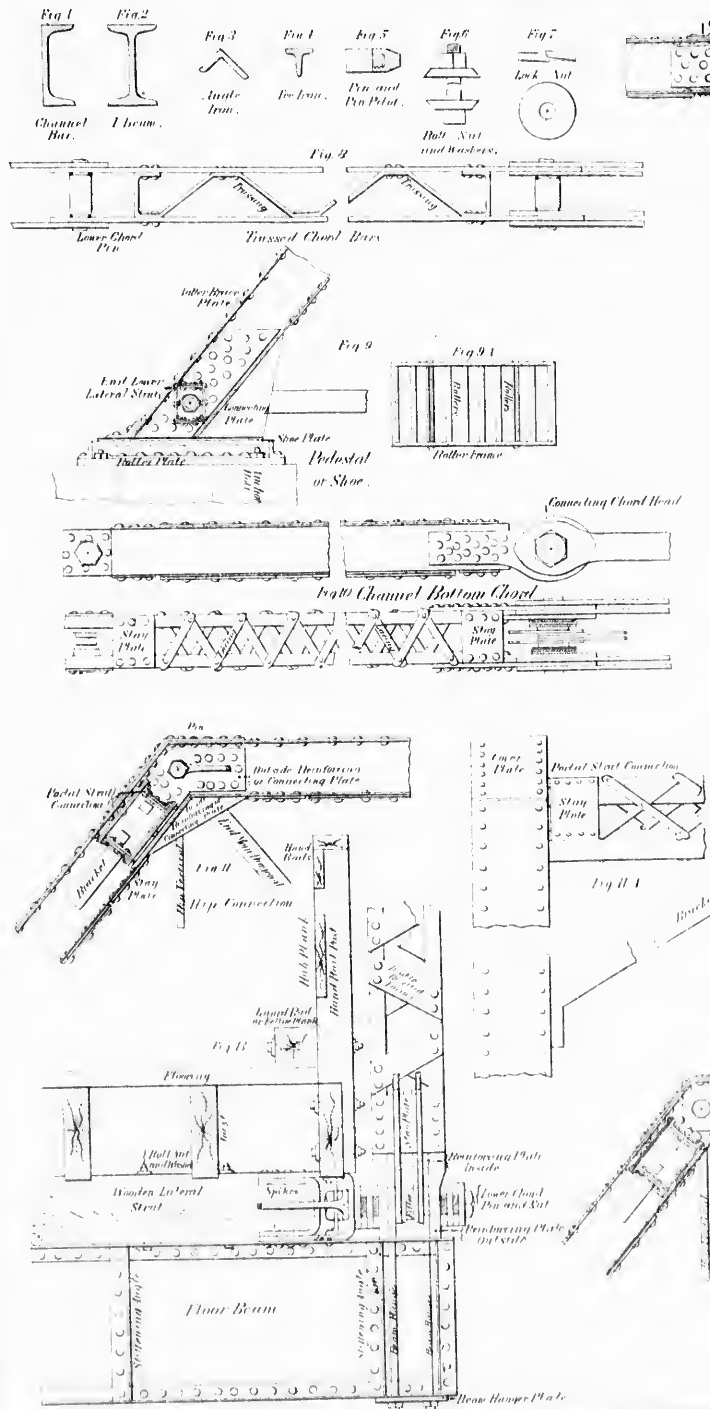


Fig 1
Channel Bar
Connection at Middle of Post
Fig 15
Pin
Fig 15 A
Stay Plate
Lateral Strut Connecting Plate
Floor Beam
Fig 16
Connecting Plate
Portal Strut Connection
Bracket
Chord Bar
Fig 17
End of a Plate Girder resting on Masonry

GENERAL DESCRI



GENERAL DESCRIPTIVE PLATE OF DETAILS

Plate II

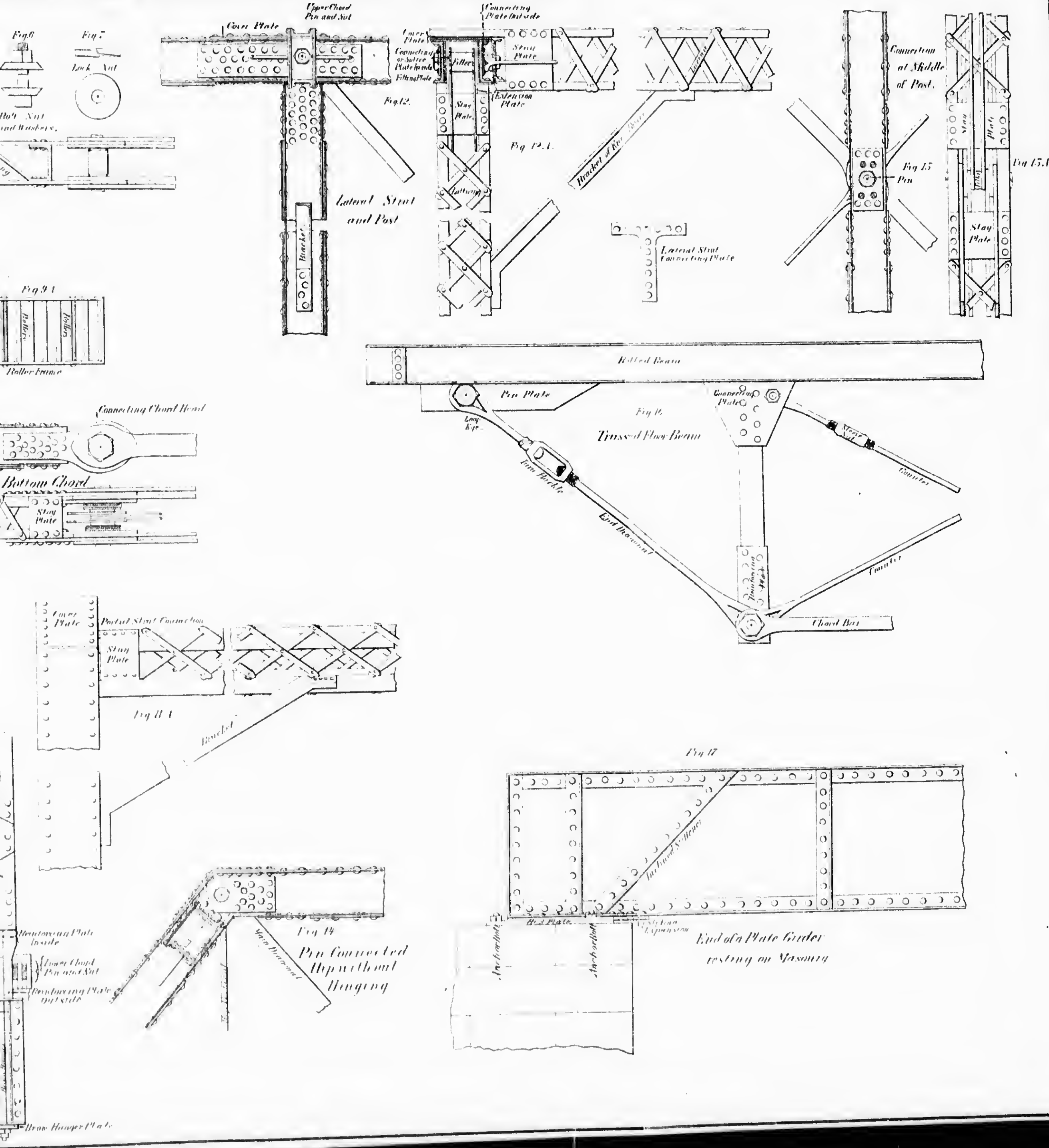
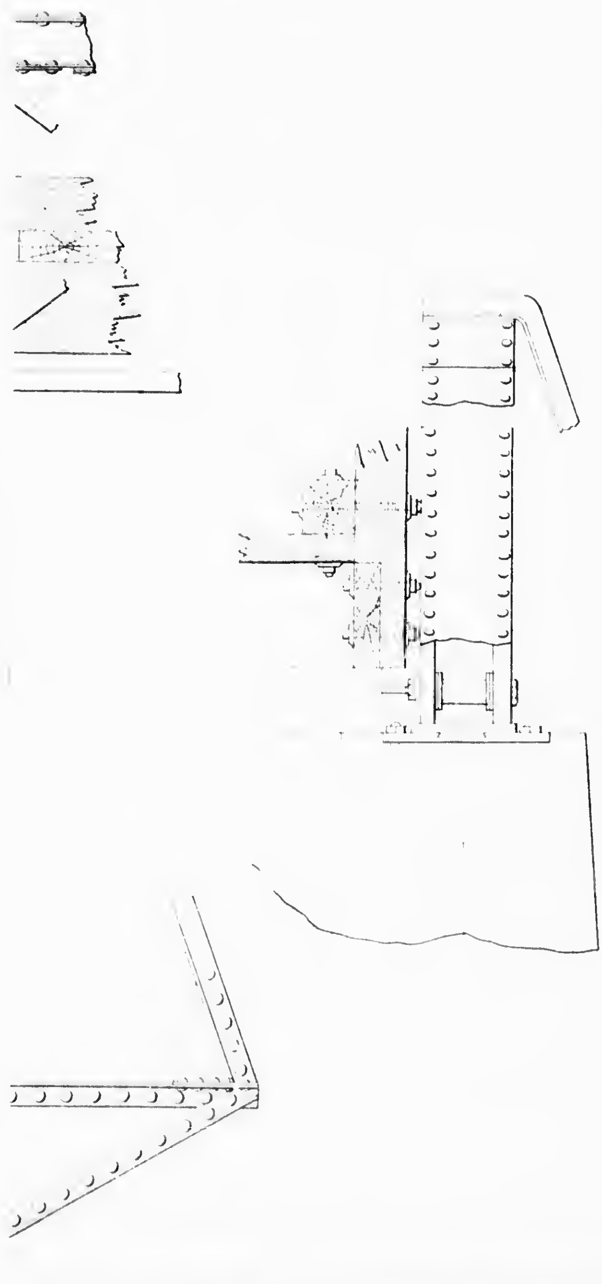


Plate III.



DETAILS FOR
IRON HIG

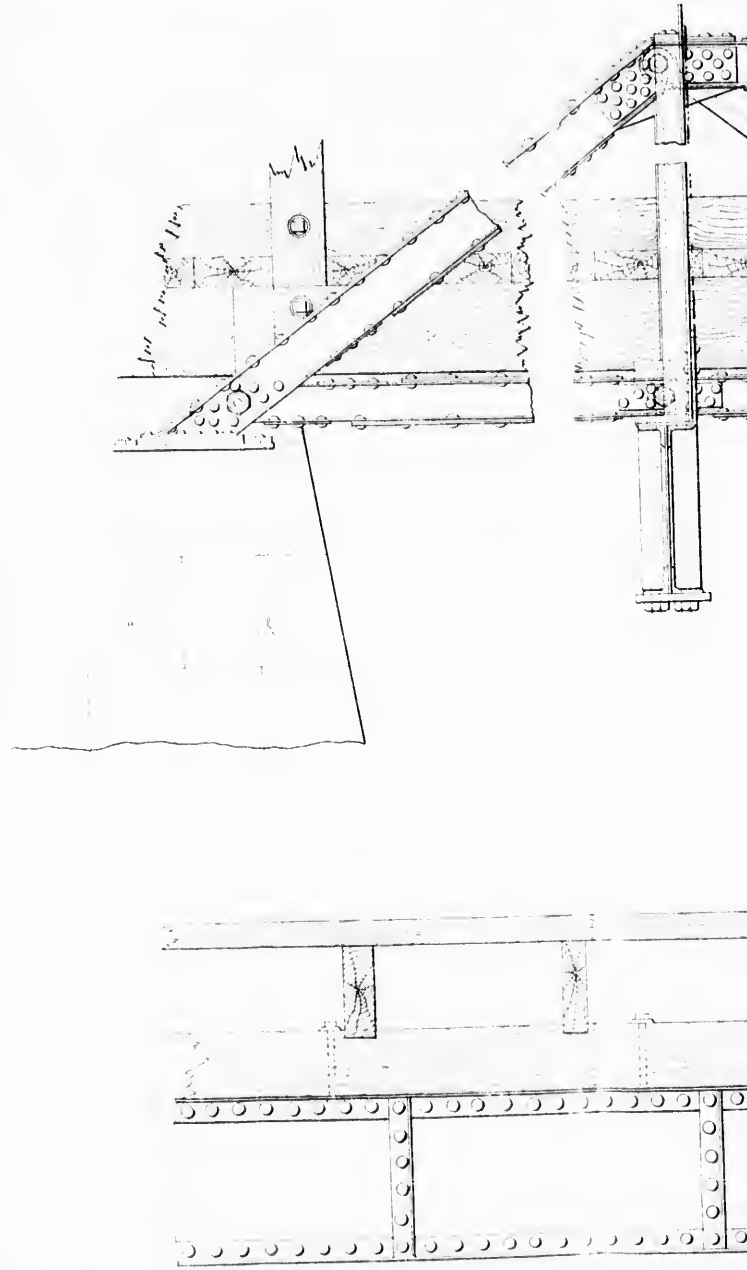
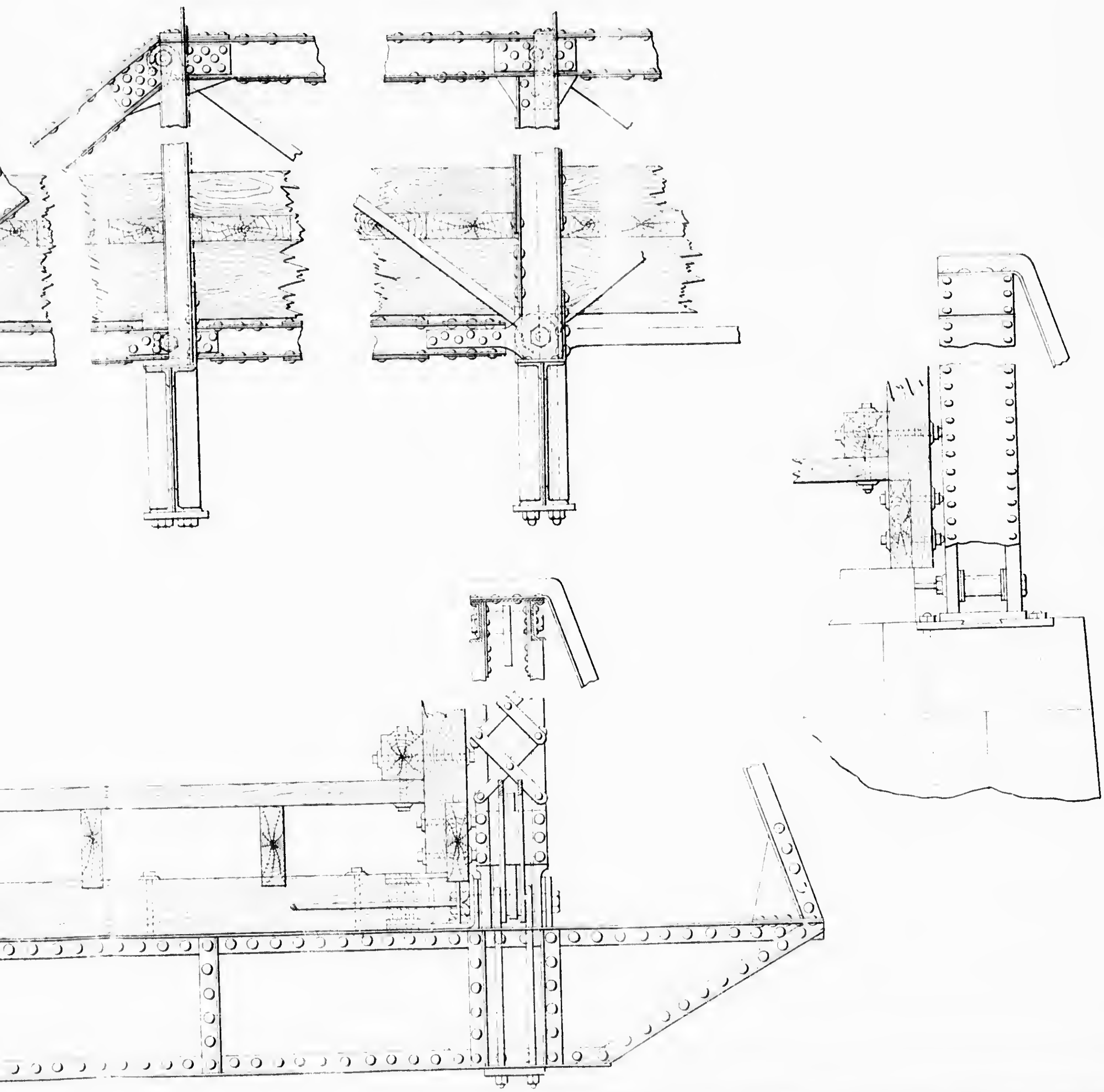
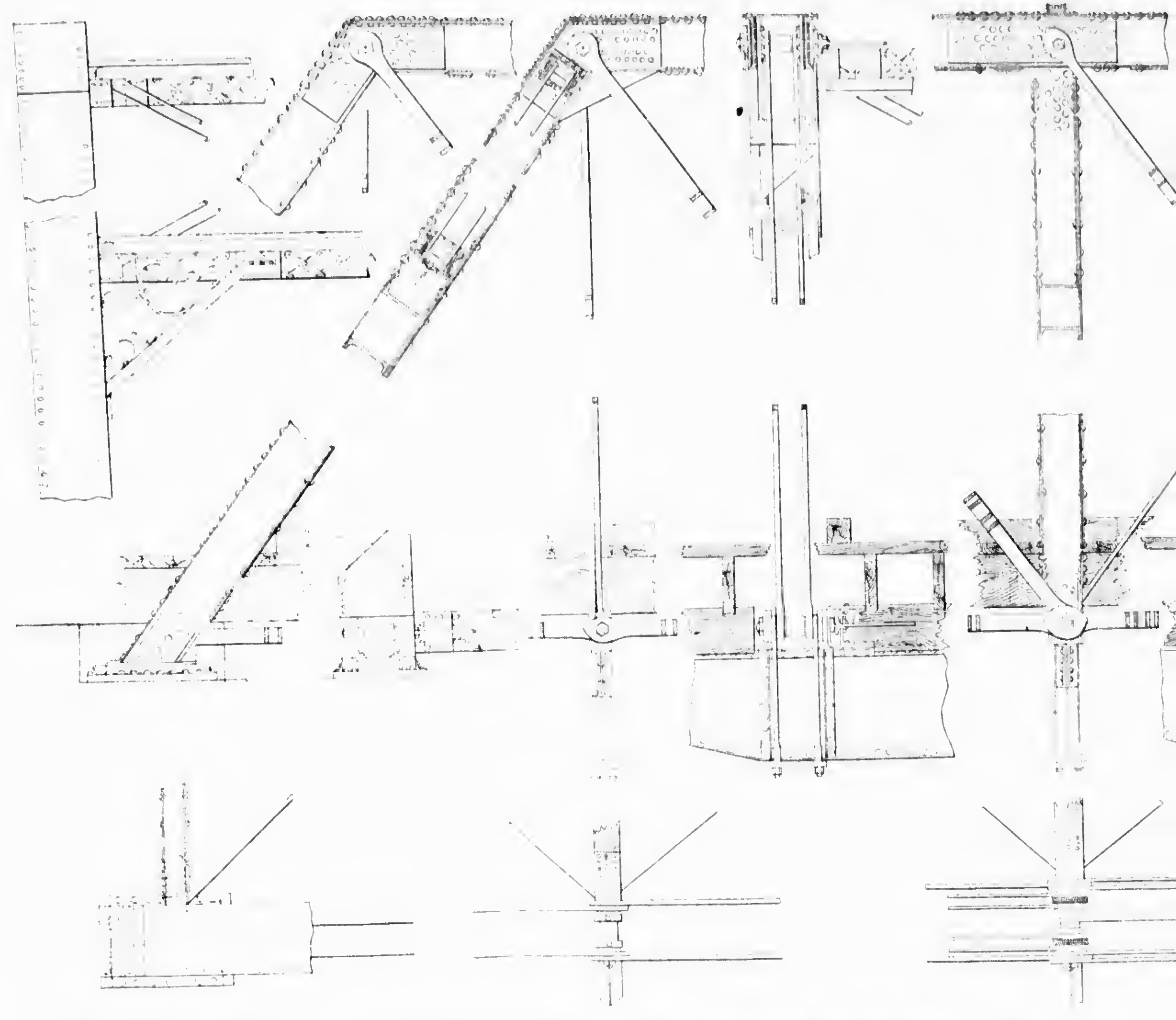


Plate III.

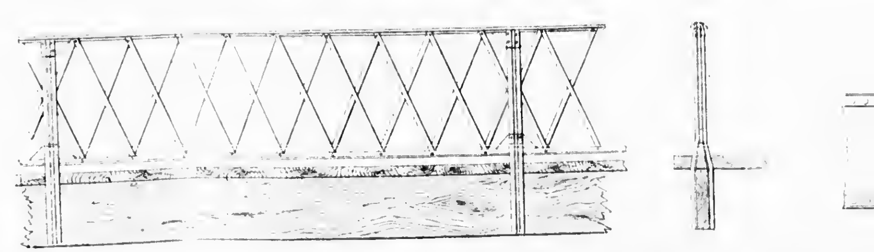
DETAILS FOR A PONY TRUSS IRON HIGHWAY BRIDGE.



DETAILS FOR A SINGLE INT
WITH SIDE

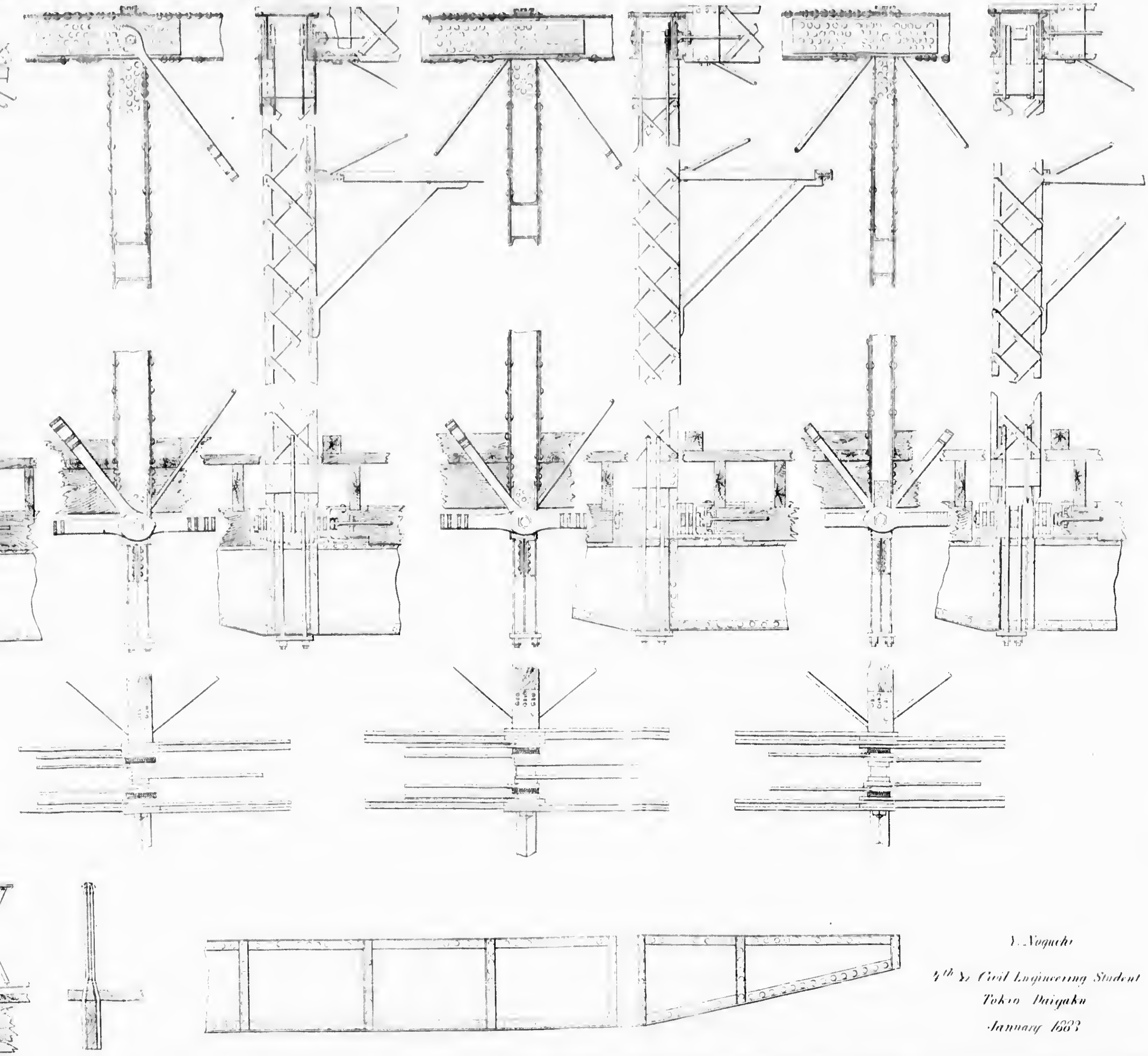


東京大學
工學科第卯年生
堅口嘉茂
明治十六年一月



SINGLE INTERSECTION BRIDGE
ITH SIDE WALKS.

Plate IV

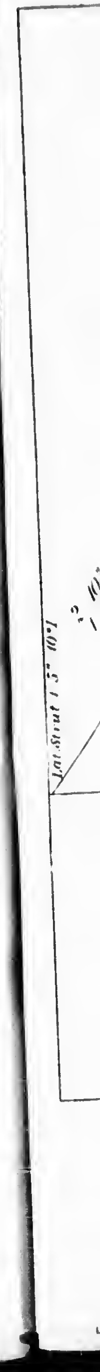


Y. Noguchi

4th Yr. Civil Engineering Student

Tokio Daigaku

January 1883



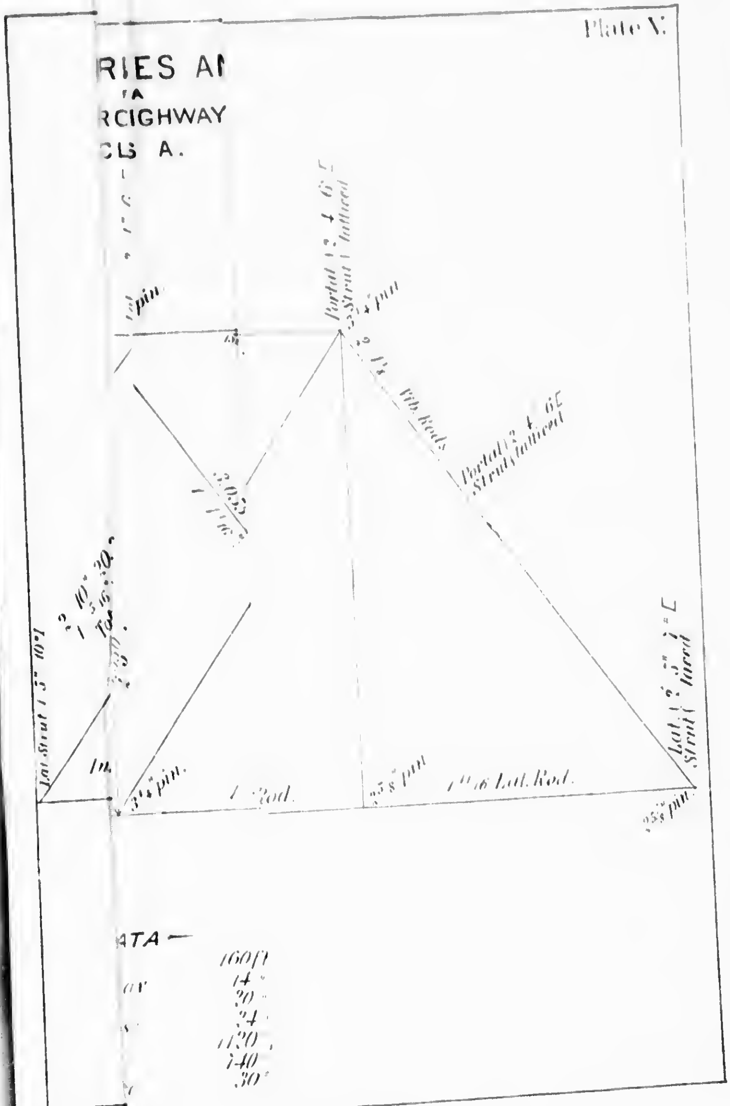


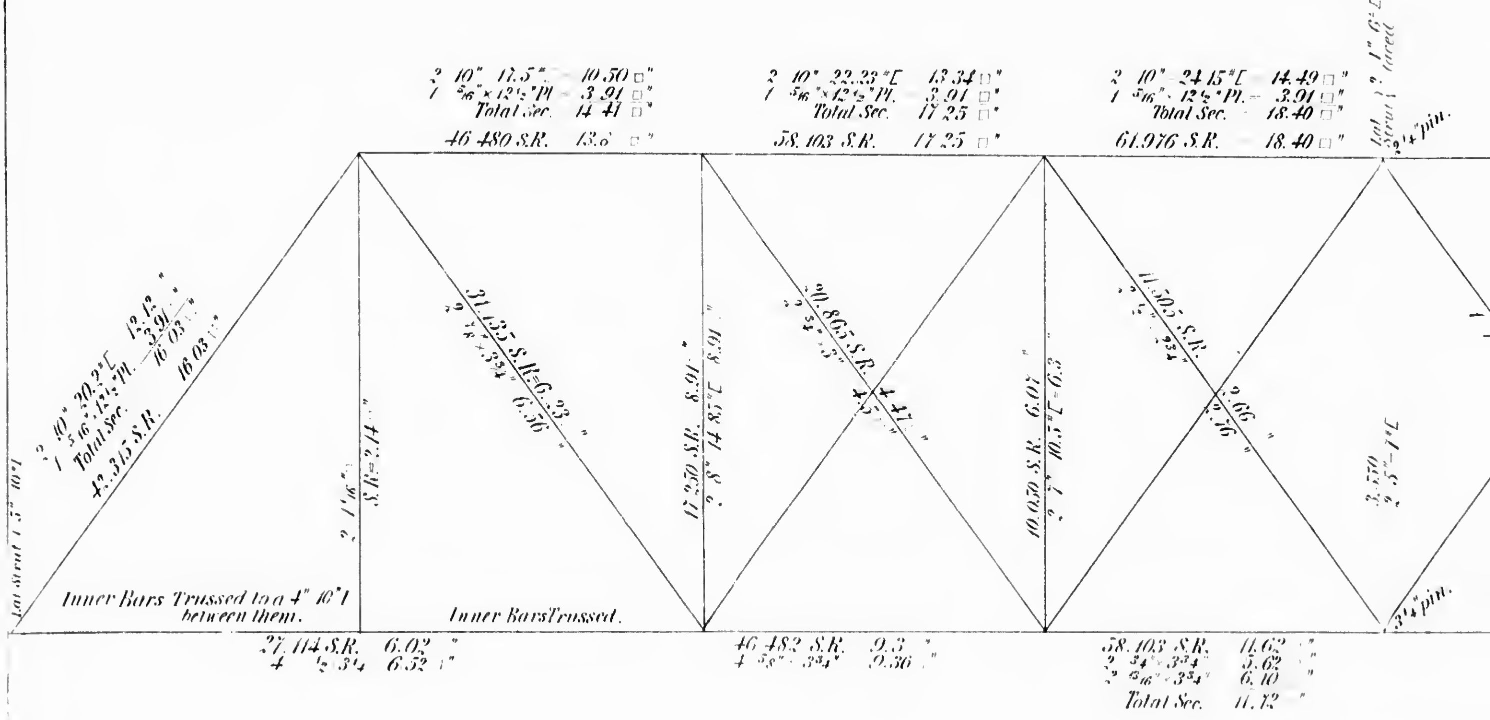
Plate V.
RIES AN
RCIGHWAY
CLS A.
Portal
Strut latticed
Portal
Strut latticed
Lat. Strut
pin.
Rod.
Lat. Rod.
ATA —
160ft
14"
20"
24"
120
140
30°

DIAGRAM OF STRESSES

FOR A

160' SPAN IRON HIGHW

CLASS A.

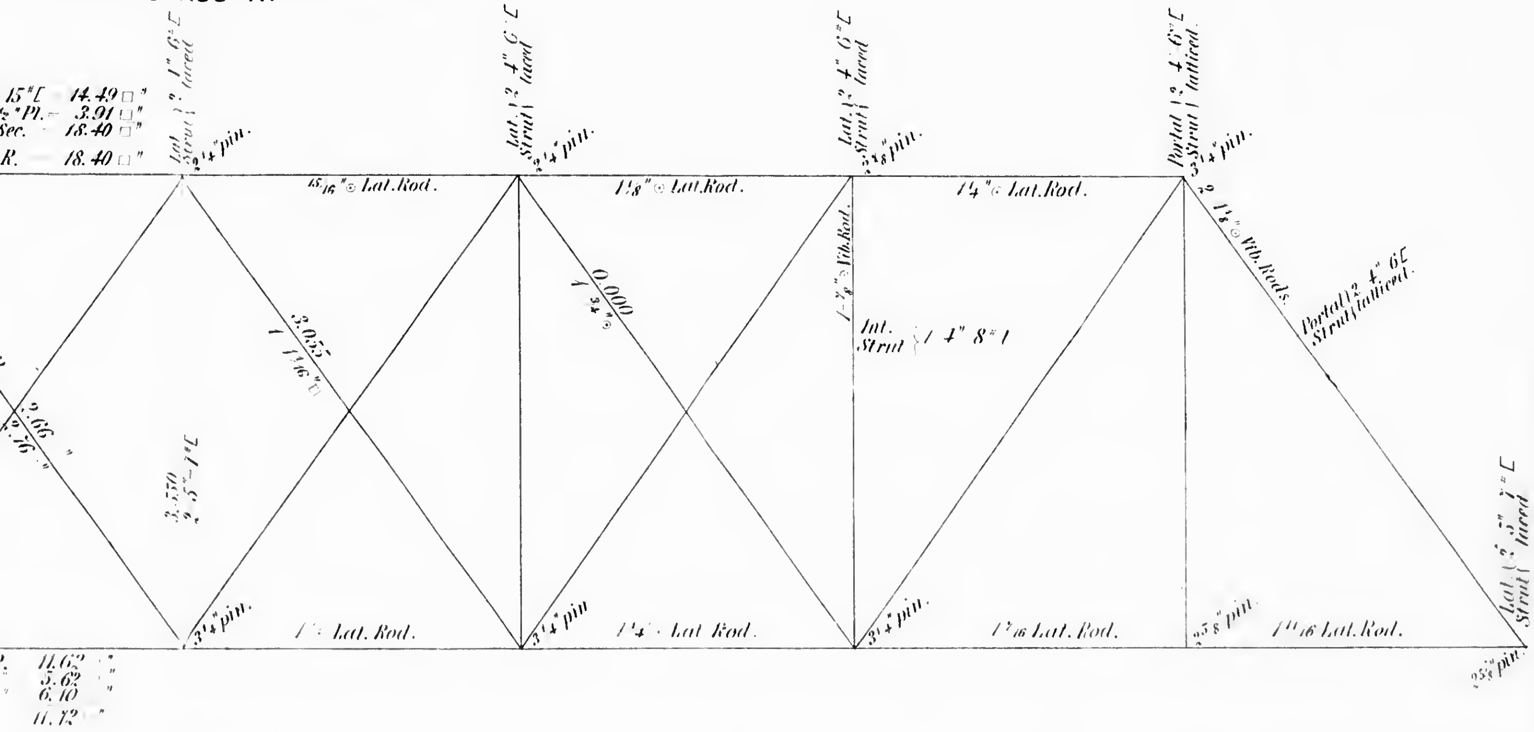


— DATA —

Spain
Clear Roadway
Panel Length
Depth of Truss
Live Load
Dead Load
Wind Pressure

Plate V.

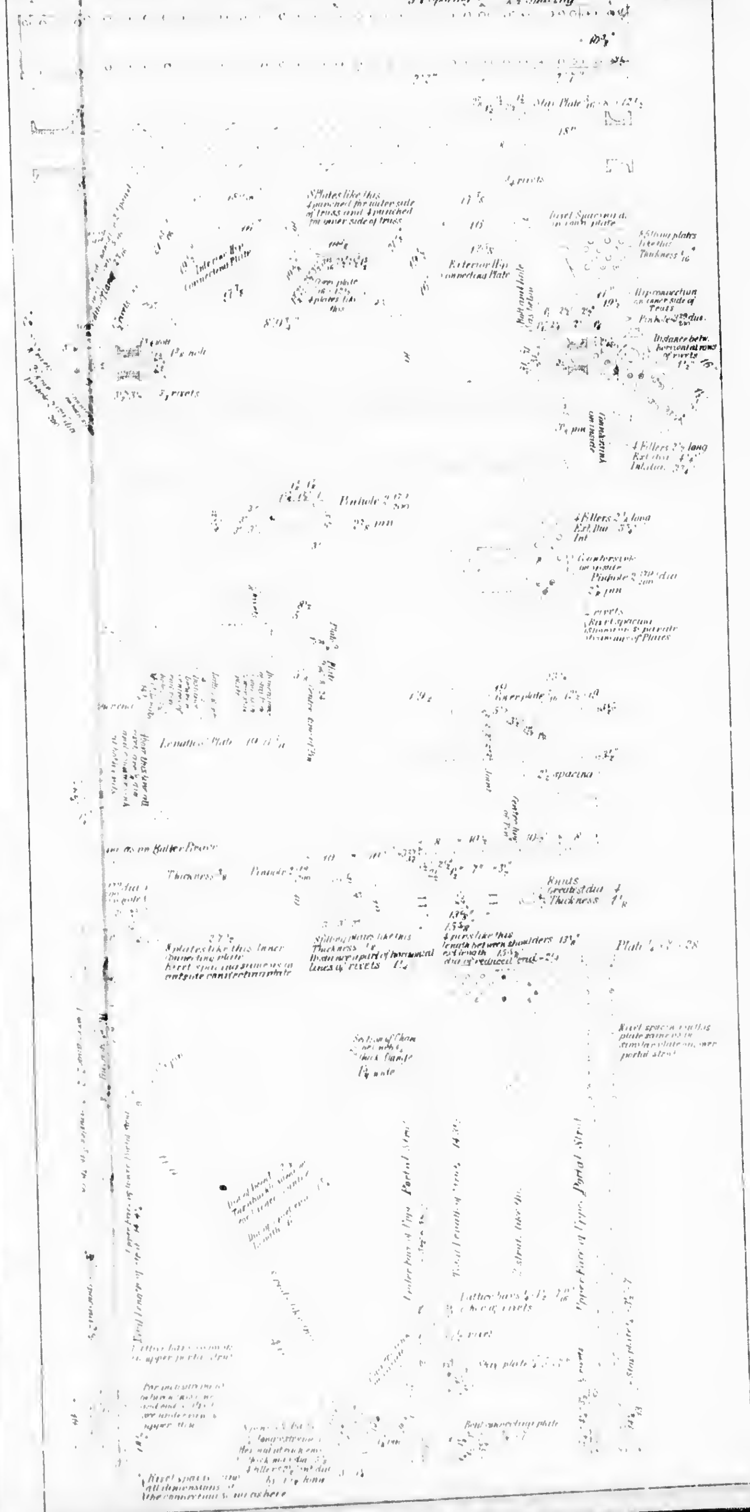
M OF STRESSES AND SECTIONS
FOR A
O' SPAN IRON HIGHWAY BRIDGE.
CLASS A.



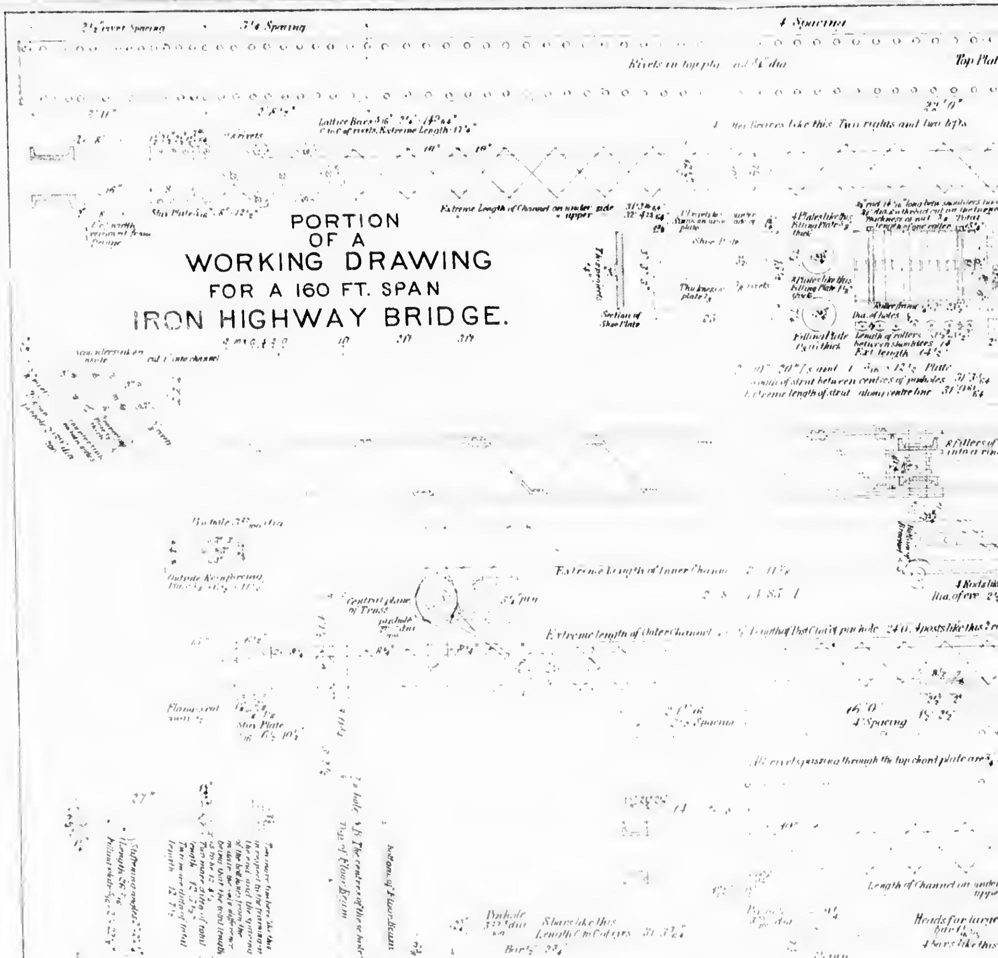
— DATA —

Span	160 ft
Clear Roadway	14 "
Panel Length	20 "
Depth of Truss	24 "
Live Load	1120 # per ft.
Dead Load	740 # " "
Wind Pressure	30 # " sq ft.

Plate VI

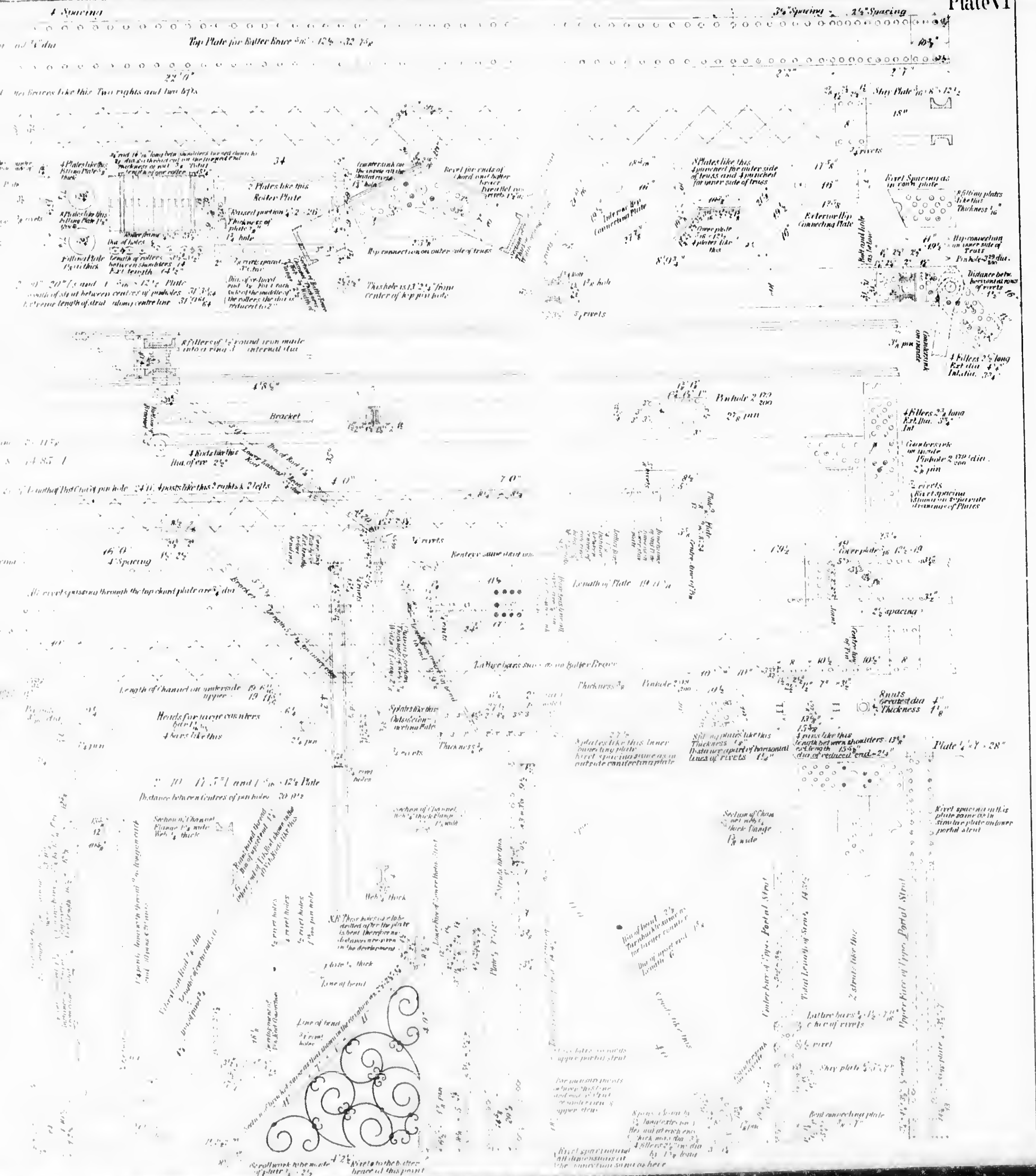


Exterior Hip connecting Plate
Interior Hip connecting Plate
Portal Strut
Portal Strut

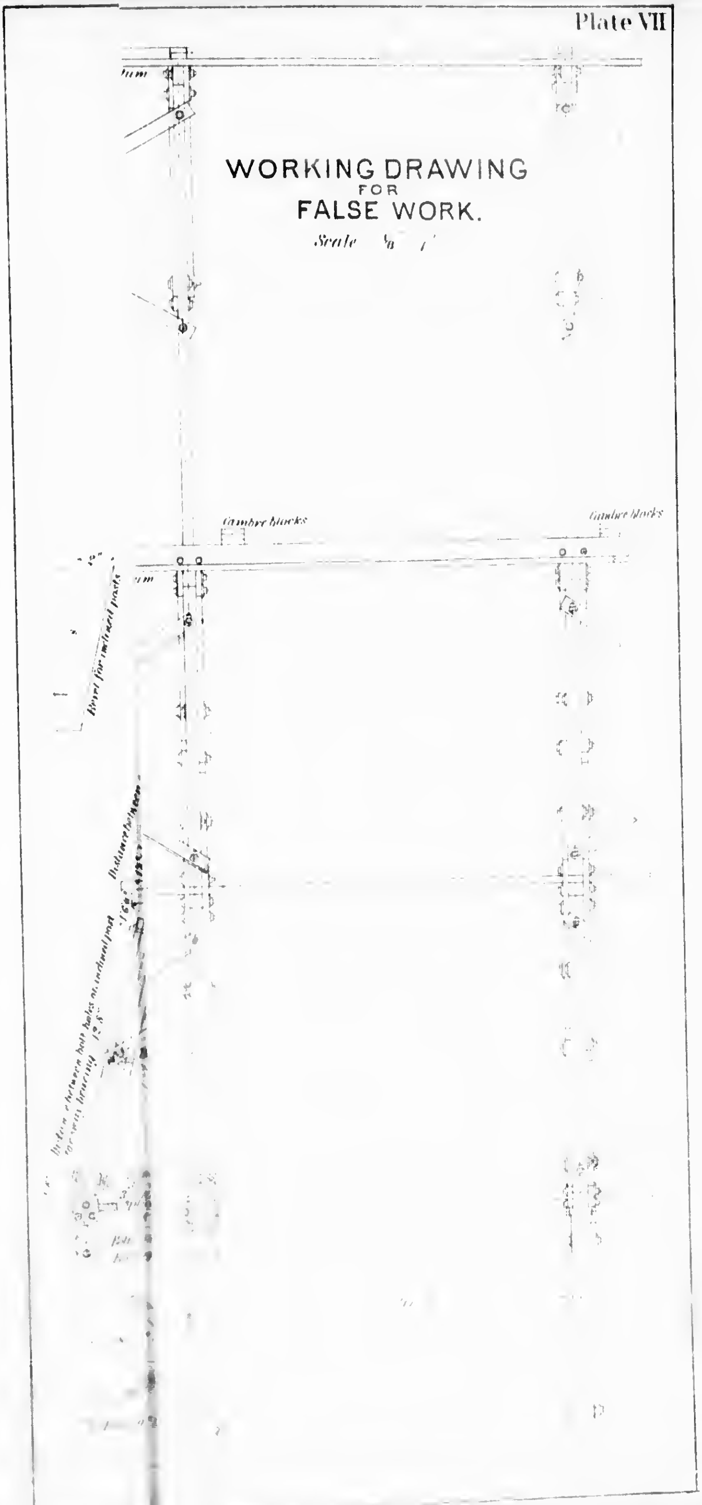


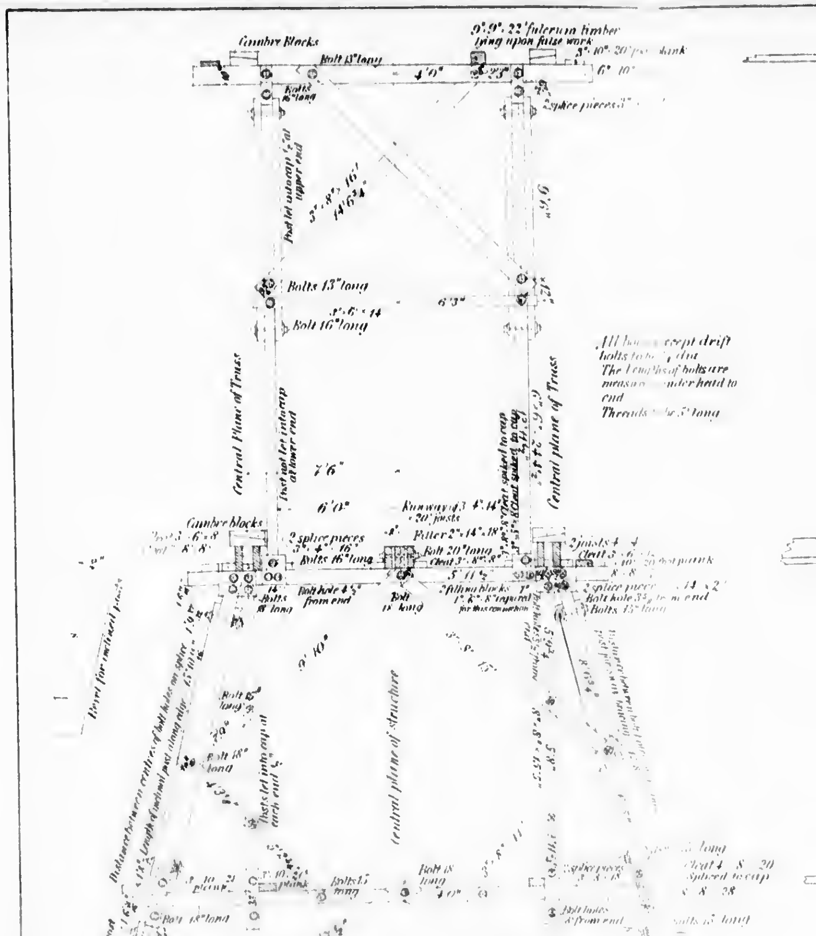
PORTION
OF A
WORKING DRAWING
FOR A 160 FT. SPAN
IRON HIGHWAY BRIDGE.
Top Plate
Section of Shoe Plate
Central plane of Bridge
Top of Floor Beam
Bottom of Floor Beam
Beam hanger

Plate VI



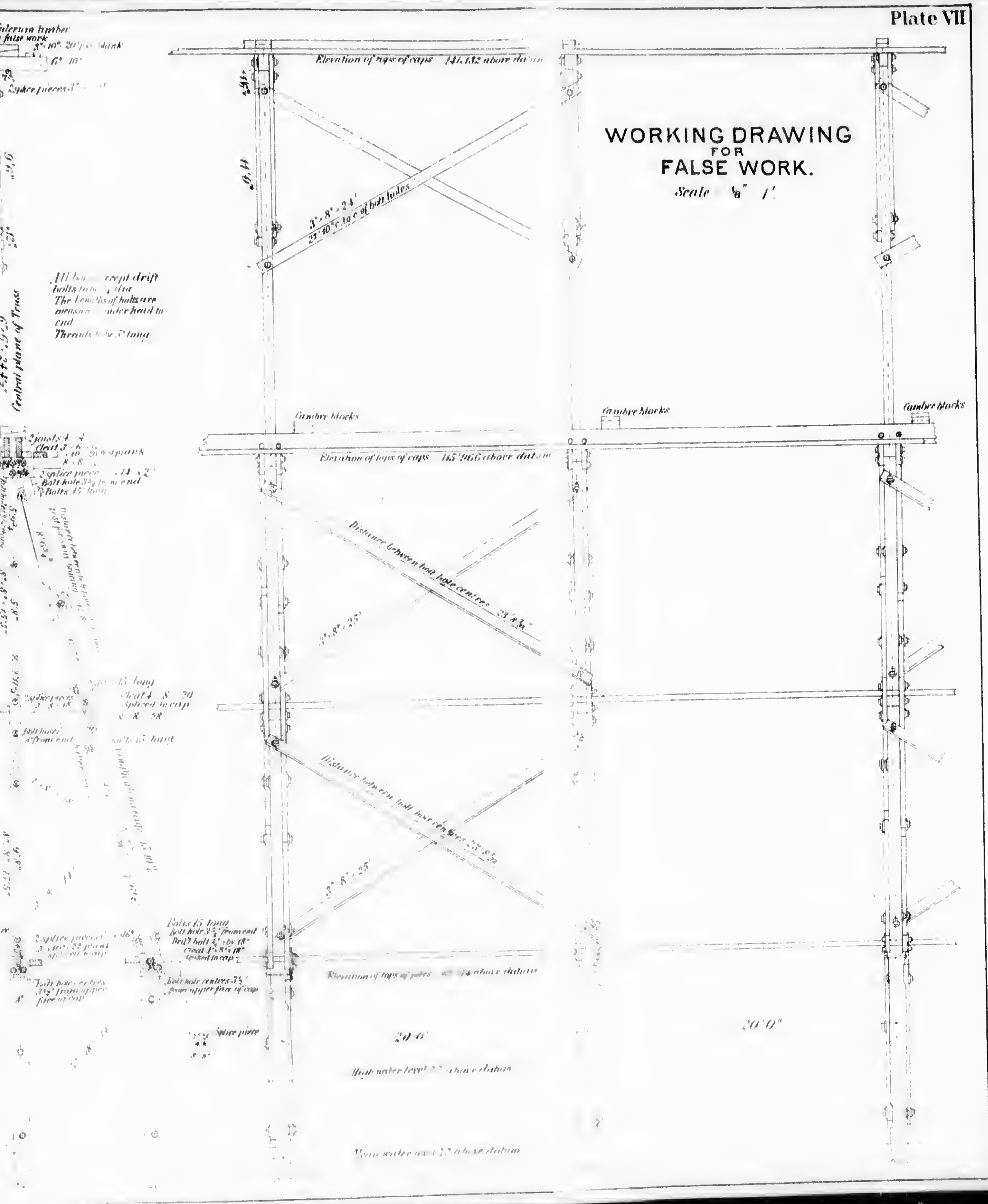
Bracket
4 Spacing
2½ spacing
Lattice bars same as on Batter Brace
Heads for nuts counters


WORKING DRAWING
FOR
FALSE WORK.
Camber blocks
Camber blocks

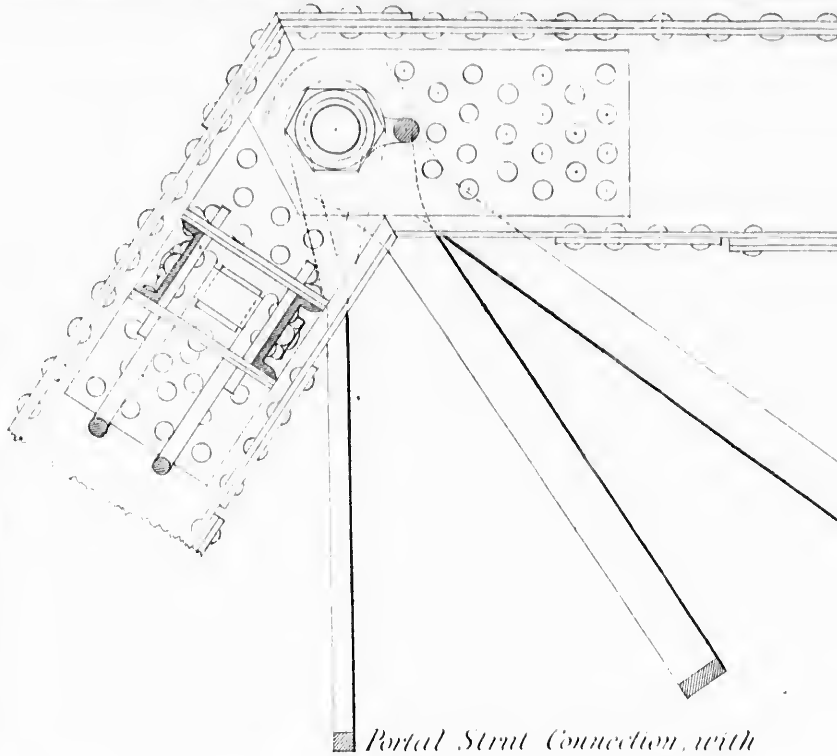


Camber Blocks
Bolt 13" long
4'0"
6" 10"
Bolts 13" long
Bolt 16" long
6'3"
Central Plane of Truss
Central plane of Truss
7'6"
6'0"
Camber blocks
Bolts 16" long
Bolt 20" long
Bolt hole 4½" from end
Central plane of structure
Bevel for inclined posts
Bolt 18" long
Bolt 15" long
Bolt 18" long
Bolt 15" long
Bolts 18" long
Bolt 19" long
Bolt 19" long
Bolt 19 long

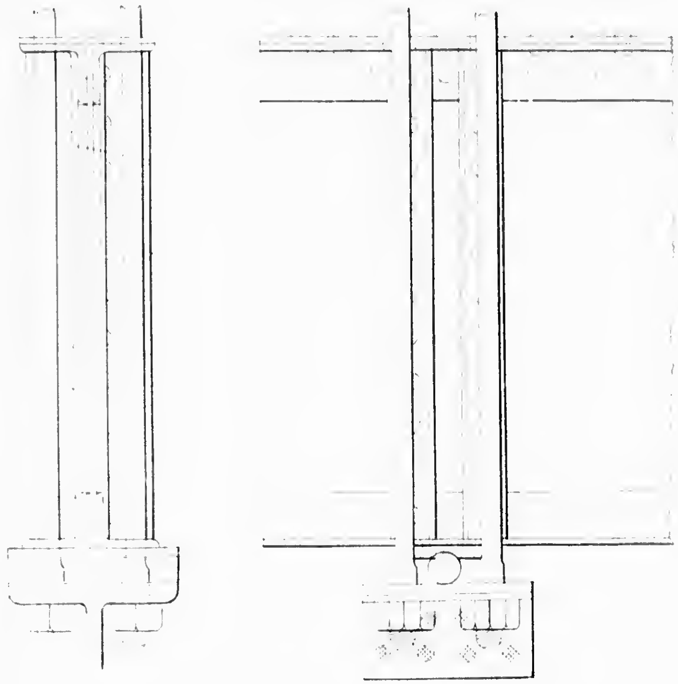
Plate VII



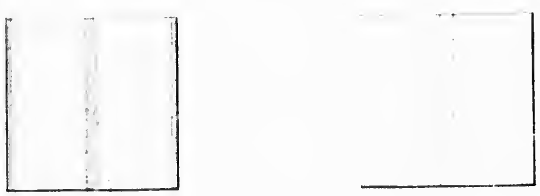
WORKING DRAWING
FOR
FALSE WORK.
Scale ⅛" = 1'

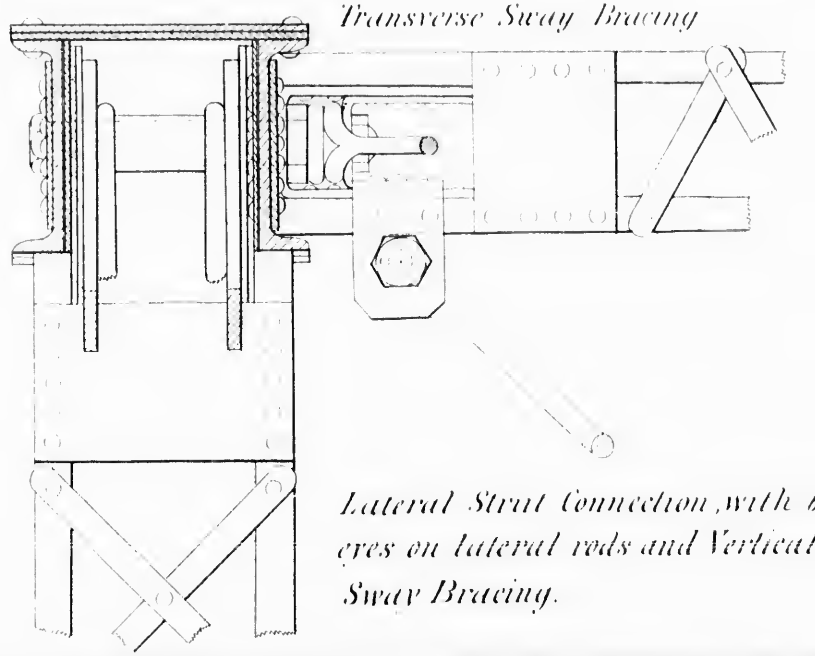


Portal Strut Connection, with Transverse Sway Bracing



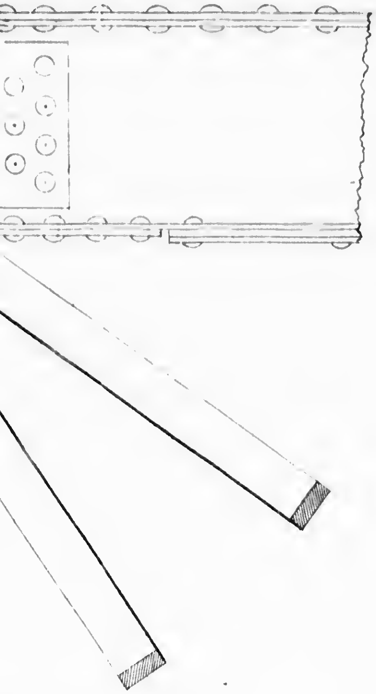
Improved Beam Hanger Plate

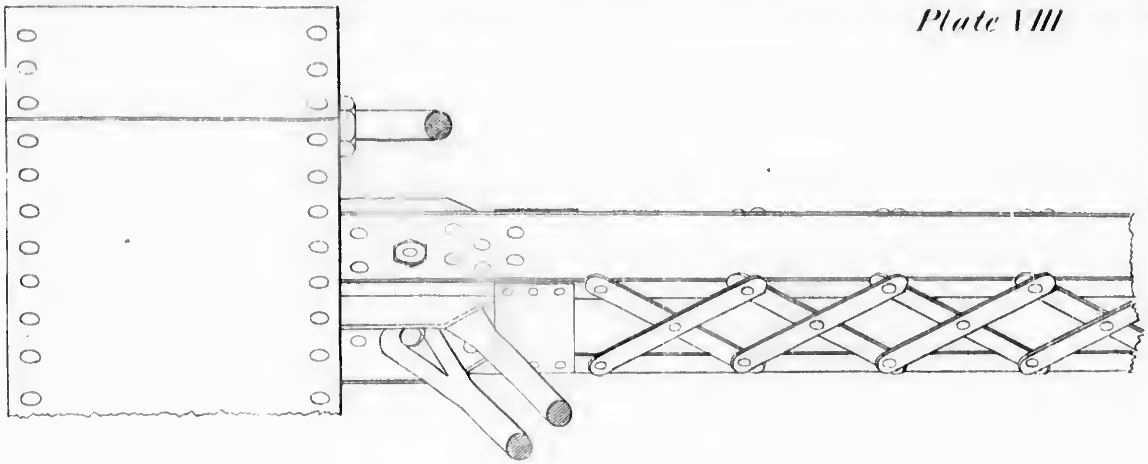




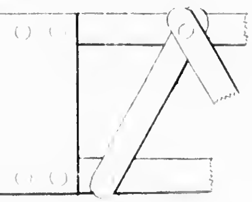
Lateral Strut Connection, with b
eyes on lateral rods and Vertical
Sway Bracing.

Plate VIII

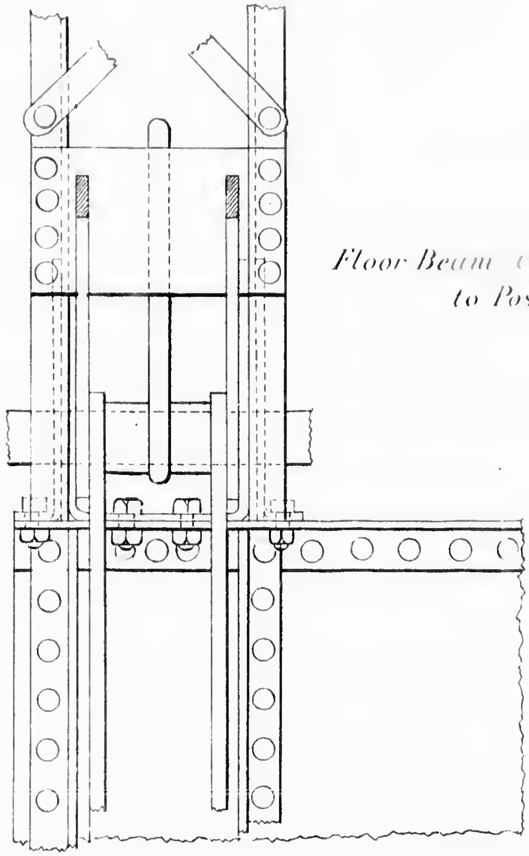




onnection, with
ay Bracing



Connection, with bent
l rods and Vertical
g.



Floor Beam Connection
to Post.