INTERNATIONAL LIBRARY OF TECHNOLOGY

A SERIES OF TEXTBOOKS FOR PERSONS ENGAGED IN THE ENGINEERING PROFESSIONS AND TRADES OR FOR THOSE WHO DESIRE INFORMATION CONCERNING THEM. FULLY ILLUSTRATED AND CONTAINING NUMEROUS PRACTICAL EXAMPLES AND THEIR SOLUTIONS

BRIDGE SPECIFICATIONS
DESIGN OF PLATE GIRDERS
DESIGN OF A HIGHWAY TRUSS BRIDGE
DESIGN OF A RAILROAD TRUSS BRIDGE
WOODEN BRIDGES
ROOF TRUSSES
BRIDGE PIERS AND ABUTMENTS
BRIDGE DRAWING

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PREFACE

The International Library of Technology is the outgrowth of a large and increasing demand that has arisen for the Reference Libraries of the International Correspondence Schools on the part of those who are not students of the Schools. As the volumes composing this Library are all printed from the same plates used in printing the Reference Libraries above mentioned, a few words are necessary regarding the scope and purpose of the instruction imparted to the students of—and the class of students taught by—these Schools, in order to afford a clear understanding of their salient and unique features.

The only requirement for admission to any of the courses offered by the International Correspondence Schools, is that the applicant shall be able to read the English language and to write it sufficiently well to make his written answers to the questions asked him intelligible. Each course is complete in itself, and no textbooks are required other than those prepared by the Schools for the particular course selected. The students themselves are from every class, trade, and profession and from every country; they are, almost without exception, busily engaged in some vocation, and can spare but little time for study, and that usually outside of their regular working hours. The information desired is such as can be immediately applied in practice, so that the student may be enabled to exchange his present vocation for a more congenial one, or to rise to a higher level in the one he now pursues. Furthermore, he wishes to obtain a good working knowledge of the subjects treated in the shortest time and in the most direct manner possible.

In meeting these requirements, we have produced a set of books that in many respects, and particularly in the general plan followed, are absolutely unique. In the majority of subjects treated the knowledge of mathematics required is limited to the simplest principles of arithmetic and mensuration, and in no case is any greater knowledge of mathematics needed than the simplest elementary principles of algebra, geometry, and trigonometry, with a thorough, practical acquaintance with the use of the logarithmic table. To effect this result, derivations of rules and formulas are omitted, but thorough and complete instructions are given regarding how, when, and under what circumstances any particular rule, formula, or process should be applied; and whenever possible one or more examples, such as would be likely to arise in actual practice—together with their solutions—are given to illustrate and explain its application.

In preparing these textbooks, it has been our constant endeavor to view the matter from the student's standpoint, and to try and anticipate everything that would cause him trouble. The utmost pains have been taken to avoid and correct any and all ambiguous expressions—both those due to faulty rhetoric and those due to insufficiency of statement or explanation. As the best way to make a statement, explanation, or description clear is to give a picture or a diagram in connection with it, illustrations have been used almost without limit. The illustrations have in all cases been adapted to the requirements of the text, and projections and sections or outline, partially shaded, or full-shaded perspectives have been used, according to which will best produce the desired results. Half-tones have been used rather sparingly, except in those cases where the general effect is desired rather than the actual details.

It is obvious that books prepared along the lines mentioned must not only be clear and concise beyond anything heretofore attempted, but they must also possess unequaled value for reference purposes. They not only give the maximum of information in a minimum space, but this information is so ingeniously arranged and correlated, and the indexes are so full and complete, that it can at once be made available to the reader. The numerous examples and explanatory remarks, together with the absence of long demonstrations and abstruse mathematical calculations, are of great assistance in helping one select the proper formula, method, or process and in teaching him how and when it should be used.

This volume treats of the design and construction of bridges and roof trusses. As an introduction to the subject, a full set of bridge specifications is given. These specifications have been carefully selected and compiled from those representing the best modern practice among bridge engineers. The principles of design are fully illustrated by the complete design, including all details, of several plate-girder bridges, a highway bridge, and a railroad bridge. The part on wooden bridges is a very simple and most convenient presentation of a subject on which there is almost no easily accessible literature. The design and construction of bridge piers and abutments, which forms so important a part of bridge engineering, is treated with the thoroughness that its importance demands. Several bridge-drawing plates—complete working drawings—are given and fully described in connection with the design work dealt with in the text.

The method of numbering the pages, cuts, articles, etc. is such that each subject or part, when the subject is divided into two or more parts, is complete in itself; hence, in order to make the index intelligible, it was necessary to give each subject or part a number. This number is placed at the top of each page, on the headline, opposite the page number; and to distinguish it from the page number it is preceded by the printer's section mark (§). Consequently, a reference such as § 16, page 26, will be readily found by looking along the inside edges of the headlines until § 16 is found, and then through § 16 until page 26 is found.

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BRIDGE SPECIFICATIONS

INTRODUCTION

1. In the early days of bridge trusses, when wood, being plentiful and cheap, was used to a great extent, bridges were invariably built without plans, and in a great many cases without previously calculating any stresses or designing any members. They were built by men called *bridge carpenters*, and the experience of the foreman or superintendent of the gang usually enabled him to decide on the sizes of members to be used. As the loads were then light and timber was much more plentiful than it is now, the errors were generally on the side of safety; that is, the bridges were made more than strong enough for the loads.

2. When wood was replaced by wrought iron, it became necessary to manufacture in the shops most of the members, and designs and plans were made. There were no systematic scientific methods of design, however; the details, instead of being proportioned according to the forces they were to resist, were designed and arranged as seemed most convenient. Many bridges at that time were designed by shop foremen that had no knowledge of stresses; the members were laid out full size on the shop floors, and made so as to utilize the material on hand in the stock yard. Very few, if any, bridges were designed to allow an increase in the loads they were to carry in the future.

3. Later, engineers began to realize that both safety and economy required that bridges should be designed according to scientific principles and constructed in conformity with fixed rules derived from both experience and theoretical investigations. Such rules, when assembled together for the guidance of the designer, builder, or contractor, are called **specifications.** Specifications differed greatly at first, but after a short time they began to approach each other, and today the points in which the various specifications differ from one another are comparatively few. It is not unlikely that standard specifications for the construction of all bridges of the same type will be adopted in the near future. The introduction of such uniform system will greatly facilitate bridge design and construction.

4. When the earlier bridges were finished, the plans, if any, that had been used in the design and construction were either destroyed or lost, as the importance of saving them for future reference was apparently not fully realized. As a result, there are at the present time many bridges in use for which no plans can be found; when it is desired to know if they can support with safety heavier loads than they have been carrying, it is difficult and very expensive to calculate their strength, for it is first necessary to measure accurately the span, panel length, and depth of girders, and trusses, the cross-sections of stringers, floorbeams, girders, and truss members, and the details of all connections. For this reason, it has become the custom to keep on file detail plans of every new bridge; these plans show the location of every rivet and the size of every piece of metal in the structure, and are of great value for future reference.

5. In the following articles are given bridge specifications agreeing with the best practice in the United States at the present time. The clauses are those that actual practice has shown to be most suitably adapted to the purposes stated. Some of these specifications will be discussed at the end of this Section; others, such as those relating to plate girders, will be given in the Sections on design. It will be sufficient for the student to read these specifications very carefully, so as to get a good idea of their contents; it is not necessary to memorize them.

The bridges in the following Sections will be designed according to these specifications. In case it is necessary to design bridges according to other specifications, as is usually the case when a designer works for a bridge or railroad company, it will simply be necessary to read over the other specifications and design the work accordingly.

SPECIFICATIONS FOR THE DESIGN OF STEEL BRIDGES

PLANS AND PROPOSALS

6. Engineer and Contractor.—The term **Engineer**, where used in these specifications, refers to the Chief or Consulting Engineer in charge of the design and construction of the bridge, and to his duly authorized assistants or representatives. The term **Contractor** refers to the bidder to whom the contract for the work has been awarded, and to his duly authorized representatives. The decision of the Engineer shall be authoritative in all cases of uncertainty.

7. Letter of Invitation.—A copy of these specifications will be furnished each bidder; in addition, he will be given **a letter of invitation** to bid, stating the general description of the work for which bids are desired and any additional facts that may be necessary. In case the requirements given in the letter of invitation conflict in any way with those in the specifications, those in the letter of invitation will rule.

8. Bids.—Bids shall state the total sum for which the work, as described in the letter of invitation, will be done, the estimated weight, and price per pound, of each class of material, and the amount of time required to complete the work. They shall be made with the understanding that the Engineer reserves the right to make such changes in the plans, before the commencement of work, as may be considered advisable by him to render the bridge a satisfactory piece of work. The increase or decrease in price due to such change shall be estimated from the pound price of the original bid, and shall be added to or deducted from the contract price.

9. **Extension of Time.**—The Contractor shall be responsible for damages on account of delay from any cause during the progress of the work. If any unforeseen delay shall arise, it will entitle him to an extension of time, to be granted in writing by the Engineer at the time of the delay.

10. **Patent Devices.**—The Contractor shall assume all responsibility for the use of patent devices in any part of the bridge, or in connection with the work of construction.

11. **Subcontractors.**—No part of the work shall be sublet, nor shall the contract for the whole or any part of the work be assigned by the Contractor, without the written consent of the Engineer. No part of the work shall be done in a shop not properly equipped with modern facilities. These specifications shall be binding on subcontractors in every respect.

12. **Plans and Stress Sheets.**—As a rule, the Engineer will provide each bidder with plans and stress sheets, showing the loads assumed, the resulting stresses, the proposed sizes and sectional areas of the members, and the style of the details and connections, as well as lengths, heights, and clearances. The bidder shall verify the plans before he submits his bid, and he alone shall be responsible for any errors, except as to general layout. He shall return the Engineer's plans if his bid is not accepted. If the Engineer does not furnish plans and stress sheets as described, the bidder shall furnish them with his bid, if requested to do so by the Engineer.

13. **Working Drawings.**—After a contract has been awarded and before any material is ordered or work commenced, the Contractor shall submit to the Engineer three complete sets of working drawings, including erection diagrams. When satisfactory, one set of such drawings and diagrams will be approved and returned to the Contractor, and all work shall be done in accordance with them. The

Contractor alone shall be responsible for the correctness of these drawings, even if they have been approved by the Engineer. No changes shall be made on the drawings after they have been approved, unless authorized in writing by the Engineer. On the completion of the work, the Contractor shall furnish the Engineer one complete set of tracings of the working drawings, which will be permanently filed in the office of the Engineer. The Contractor shall, when required, furnish also the necessary plans for designing the masonry.

Drawings shall preferably be not more than 24 in. × 36 in., with details drawn to a scale of $\frac{3}{4}$ or 1 inch to 1 foot.

DESIGN OF RAILROAD BRIDGES

GENERAL DIMENSIONS

14. Kinds of Bridges.—The following kinds of bridges shall preferably be used:

For spans less than 25 feet in length, rolled beams.

For spans from 25 to 100 feet in length, plate girders.

For spans from 100 to 150 feet in length, riveted trusses.

For spans over 150 feet in length, pin-connected trusses.

If, for any reason, it is desired to depart more than 10 feet from these limits, permission in writing must be obtained from the Engineer.

Deck bridges will have the preference wherever the conditions permit their use.

15. Panel Lengths and Depths.—The depth of plate girders shall preferably be one-eighth, and in no case less than one-twelfth, of the span. The depth of trusses shall preferably be not less than one-sixth of the span. Panel lengths shall preferably be from 10 to 25 feet, and in truss bridges shall be so chosen that the angle between diagonal ... and the lower chord shall be not less than 50°.

... **gers and Deck Girders.**— ... at 6 inches center to center.

Deck girders less than 70 feet long shall be spaced 6 feet 6 inches center to center; deck girders over 70 feet long shall be spaced 6 inches farther apart for each 10 feet increase in length. In bridges on curves, the center line between stringers, and between deck girders, shall be parallel to the chord of the curve between abutments, and at a distance from it equal to two-thirds the middle ordinate.

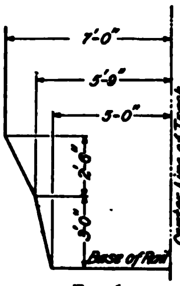

FIG. 1

17. Half-Through Bridges.—Half-through truss bridges shall be avoided when possible. Where used, the flanges and brackets shall not come closer to the center line of track than shown in outline in Fig. 1.

18. Through Bridges.—In through bridges on straight track, no part of the structure shall come closer to the center line of the nearest track than shown in outline in Fig. 2. In bridges on curves, there shall be provided 1 inch additional clearance on each side of the track for each degree of curvature, and 2½ inches additional clearance on the inside of the curve for each inch of superelevation of track.

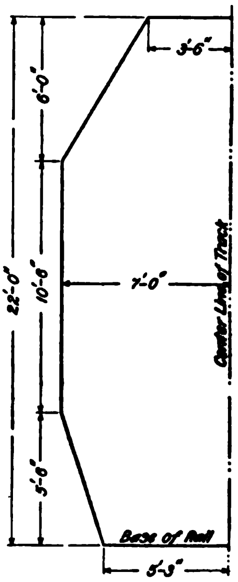

FIG. 2

19. Spacing and Gauge of Tracks.—Tracks shall be spaced 13 feet center to center, unless otherwise specified. The gauge of track is 4 feet 8½ inches.

20. Spacing of Trusses.—The distance center to center of trusses shall preferably be not less than one-twelfth the span, and in no case less than one-half the depth of trusses.

21. Double-Track Spans.—Double-track spans shall preferably have only two trusses or girders. Where, on account of thin floors, three-truss bridges are advisable, the tracks shall be spread so as to provide the proper clearance between the trusses for each track.

LOADING

22. Loads.—Bridges shall be designed to resist properly the stresses caused by the following forces: *dead load; live, or moving, load; impact and vibration; centrifugal force; wind pressure;* and *the longitudinal force due to suddenly stopping trains.*

23. Dead Load.—The dead load shall consist of the estimated weight of the entire structure. The weight of ties, guard timbers, and rails shall be taken as 400 pounds per linear foot of track, of timber as 4½ pounds per board foot, and of ballast as 120 pounds per cubic foot. In truss bridges, two-thirds of the dead load shall, in general, be assumed as applied at the loaded chord, and one-third at the unloaded chord.

24. Live Load.—The live load on each track shall consist of two engines followed by a uniform load of 5,000 pounds per linear foot, as represented in Fig. 3, or a loading

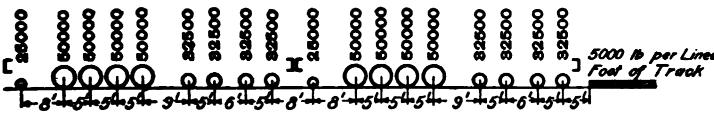

FIG. 3

having the same spacing of wheels and derived from the former by multiplying each load by the same number.

25. Impact and Vibration.—To provide for impact and vibration, an amount I is to be added to the stress or bending moment in each member, as given in the following formulas, in which

S = maximum live-load stress or bending moment in the member;

L = length, in feet, of single track that must be loaded in order to obtain the value S.

For counters, hip verticals, subverticals, short diagonals, floor members and connections, and members subject to reversal of stress,

$$I = S$$

For all other members,

$$I = \frac{300}{L + 300} S$$

26. Centrifugal Force.—In bridges on curves, the centrifugal force F shall be found from the formula

$$F = \frac{(4.5 - .2\,D)}{100} D\,W$$

in which D = degree of curvature;
W = live load.

Centrifugal force shall be assumed to act 6 feet above the rail.

27. Wind Pressure.—Wind pressure shall be assumed as 300 pounds per linear foot on a train, applied 7 feet above the top of the rail, and 30 pounds per square foot on the exposed area of one girder in girder bridges, one truss in through bridges, or two trusses in deck bridges, together with the floor in truss bridges. When 50 pounds per square foot on twice the exposed area of one truss and the exposed area of the floor, with no train on the bridge, produces greater stresses than the above, the greater stresses shall be used.

28. Suddenly Stopping Trains.—The longitudinal force due to the friction between the rails and the wheels of suddenly stopping trains shall be taken as one-fifth of the live load on the structure.

DESIGN OF MEMBERS

29. Working Stresses.—All parts shall be so designed that the sum of the maximum stresses in any part shall not cause the intensity of stress to exceed the following values in pounds per square inch:

Tension on net section, 16,000.

Compression on gross section,

$$\frac{16,000}{1 + \dfrac{l^2}{18,000\, r^2}}$$

in which l = unsupported length of member, in inches;
r = least radius of gyration, in inches.

In half-through truss bridges, the entire length of the upper chord shall be considered unsupported laterally.

Shear on net section of web-plates,

$$\frac{12,000}{1 + \dfrac{d^2}{3,000\, t^2}}$$

in which t = thickness of web, in inches;
d = clear distance, in inches, between stiffeners or flange angles, whichever is the smaller.

The intensity of the shearing stress found by dividing the total vertical shear by the gross area of cross-section of the web shall in no case exceed 9,000.

Shear on shop rivets and pins, 11,000.

Shear on field rivets and turned bolts, 9,000.

Bending on pins, 22,000.

Bending on rolled sections and plate girders:

Tension, 16,000.

Compression, when unsupported length l of compression flange is not greater than twenty times the width w, 16,000.

Compression, when l is greater than $20\,w$,

$$20,000 - 200\frac{l}{w}$$

Bearing on shop rivets and pins, 22,000.

Bearing on field rivets, turned bolts, and ends of stiffeners, 18,000.

Bearing on masonry:

Sandstone and limestone, 300.

Cement concrete, 400.

Granite, 500.

Bearing on rollers shall not exceed $600\,D$ pounds per linear inch of roller, D being diameter of roller in inches.

30. Data.—The following general dimensions shall first be calculated or assumed:

Length of trusses, center to center of end pins or pedestals.

Length of girders, center to center of bearings.

Length of floorbeams, center to center of girders or trusses.

Length of stringers, center to center of floorbeams.

Depth of trusses and girders, center to center of gravity of chords or flanges.

31. Floor Members.—Solid floor sections, I beams, and channels shall be designed by their moments of inertia, or section moduli. The load on each axle shall be assumed to be distributed over a length of 3 feet in designing solid floor sections. In bridges on curves, stringers and deck girders shall be designed for the increase in load due to the eccentricity of the track.

32. Compression Members.—Pin and bolt holes shall be deducted from the gross section of compression members. The value of $\frac{l}{r}$ shall preferably be from 40 to 60, and must not exceed 100 for main members, nor 120 for members of lateral systems. Splices in compression members shall have sufficient rivets to fully develop the stresses in the members.

33. Tension Members.—The net section of a riveted tension member shall be determined by deducting from the gross section the area of cross-section of the greatest number of pin, bolt, or rivet holes that can be cut by a plane at right angles to the member. In addition, for rivets $\frac{7}{8}$ inch in diameter and larger, there shall be deducted each hole whose center lies within $\frac{3}{4}$ inch of the cutting plane, and a proportionate part of each hole whose center lies within $2\frac{3}{4}$ inches; and for rivets $\frac{3}{4}$ inch in diameter and smaller, each hole whose center lies within $\frac{1}{2}$ inch of the cutting plane, and a proportionate part of each hole whose center lies within 2 inches. Rivet holes shall be taken $\frac{1}{8}$ inch larger in diameter than the rivets.

34. Reversal of Stress.—Members subject to both tension and compression shall be designed to resist each stress plus eight-tenths of the other stress.

35. Combined Stresses.—Members subject to transverse stresses in addition to the direct stresses shall be designed for both.

36. Bearing Values of Rivets.—In calculating the bearing value of a rivet, the area subjected to stress shall be taken as equal to the product of the thickness of the plate and the diameter of the rivet before driving, that is, the nominal diameter. The value of countersunk rivets shall not be counted.

GENERAL DETAILS

37. General Requirements for Details and Connections.—Special attention shall be given to all details and connections; they shall always be of greater strength than the body of the member. All details shall be accessible for inspection, cleaning, and painting. Details that permit the collection of water shall be avoided if possible; if used, they shall be provided with drainage holes or filled with cement concrete.

38. Minimum Thickness.—No material less than $\frac{3}{8}$ inch thick shall be used except for latticing and fillers.

39. Single Angles.—Members, or sides of members, composed of single angles shall have both legs of each angle connected at the ends, or only 75 per cent. of the section shall be counted. No angle shall be smaller than 3 in. $\times$ 3 in. $\times$ $\frac{3}{8}$ in., nor be connected by less than four rivets, except for unimportant details.

40. Size of Rivets.—Rivets shall generally be $\frac{7}{8}$ inch and $\frac{3}{4}$ inch in diameter. The diameter of the rivet shall not be greater than one-fourth the width of the bar or angle through which the rivet passes, except for unimportant details, where $\frac{7}{8}$-inch rivets may be used in 3-inch angles, and $\frac{3}{4}$-inch rivets in $2\frac{1}{2}$-inch angles.

41. Spacing of Rivets.—Rivets $\frac{7}{8}$ inch in diameter shall be spaced not more than 6 nor less than 3 inches center to center, and placed not closer than $1\frac{3}{4}$ inches to any sheared edge nor closer than $1\frac{1}{2}$ inches to any rolled edge—except in special cases to conform to standards, where they may be placed not closer than $1\frac{1}{4}$ inches to a rolled edge. Rivets $\frac{3}{4}$ inch in diameter shall be spaced not more than 6 nor less than $2\frac{1}{2}$ inches center to center, and placed not closer than $1\frac{1}{2}$ inches to any sheared edge nor closer than $1\frac{1}{4}$ inches to any rolled edge—except to conform to standards, where they may be placed not closer than $1\frac{1}{8}$ inches to a rolled edge. The spacing of rivets at the ends of compression members shall not exceed four times the diameter of the rivets for a distance equal to the width of the member.

42. Grip of Rivets.—The grip of rivets shall preferably not exceed five times the diameter of the rivet, and shall in no case exceed 5 inches. When the grip is greater than 4 inches, the calculated number of rivets shall be increased 1 per cent. for each $\frac{1}{16}$ inch increase in grip.

43. Compression Members.—In compression members, the material shall mostly be concentrated at the sides. The unsupported widths of plates shall not exceed thirty times their thickness for web-plates, nor forty times their thickness for cover-plates of chords and end posts. No closed sections will be allowed.

44. Tie-Plates and Lattice Bars.—The open sides of all built-up members shall be stiffened by means of tie-plates and lattice bars. The length of tie-plates shall be not less than $1\frac{1}{4}$ times the width of the member. Double latticing shall preferably make an angle of about 45° with the axis of the member, and the bars shall be riveted where they cross each other. Single latticing shall preferably make an angle of about 60° with the axis of the member. Bars in single latticing shall have a thickness not less than one-fortieth, and in double latticing not less than one-sixtieth, of the length of the bar. Lattice bars shall be not less than $2\frac{1}{4}$ inches wide for members up to 9 inches, not less than

$2\frac{1}{2}$ inches for members from 9 to 15 inches, and not less than 3 inches for members more than 15 inches in width or depth.

45. Expansion.—Provision for expansion and contraction due to changes of temperature shall be made at the rate of 1 inch for every 100 feet.

46. Camber.—All trusses shall be cambered by giving the panels of the top chord an excess of length in the proportion of $\frac{1}{8}$ inch to every 10 feet. Plate girders shall not be cambered.

47. I Beams.—In short deck spans, when more than one I beam is used under a rail, the beams shall be bolted together with cast-iron separators between them, and connected by lateral bracing between the two sets of beams.

DETAILS OF FLOOR SYSTEMS

48. Ties, Guard Timbers, and Rails.—Ties, guard timbers, rails, wooden floors, and ballast, where necessary, will be provided and put in place by the railroad company. Cross-ties are 8 in. × 8 in., 10 feet long, framed to not less than $7\frac{1}{2}$ inches over bearings for stringers and girders 6 feet 6 inches center to center. The depth of the tie is increased 1 inch for each 6 inches additional width of girders. Ties are spaced 12 inches center to center, and every fourth tie is fastened to each stringer by a $\frac{3}{4}$-inch bolt. Guard timbers are 8 inches wide and 6 inches thick, framed to 4 inches over ties, and spaced 4 feet from inner edge to center of track. They are fastened by $\frac{5}{8}$-inch bolts to the ties that are connected to the stringers.

49. Floor Members.—Floor members shall be designed with special reference to stiffness; the depth of stringers shall be not less than one-eighth of the panel length, and that of floorbeams not less than one-sixth of the distance between trusses or girders.

50. Floor Connections.—Stringers shall be at right angles to the floorbeams, and shall be riveted to the floorbeam webs. If possible, they shall also rest on shelf angles riveted to the webs of the floorbeams. Floorbeams shall preferably be at right angles to the girders or trusses. In half-through plate-girder bridges, the beams shall be riveted to the webs of the girders. In through truss bridges, the beams shall be riveted to the vertical posts, or to the web connection plates; if there are no vertical posts, diaphragms shall be riveted in, connecting the web connection plates at the ends of the floorbeams. In deck truss bridges, the beams shall either be riveted to the trusses as in through bridges, or rest on the upper chords.

51. Connection Angles.—The connection angles of stringers to floorbeams, and floorbeams to girders and trusses, shall not be smaller than $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{9}{16}$ in. The fillers under connection angles shall be wider than the adjacent leg of the angle, and shall have two-thirds as many rivets through the projecting portion as through the portion under the angle.

52. Deck Bridges.—In deck girder bridges, the ties shall rest directly on the top flange. In deck truss bridges, the ties shall not rest directly on the top chord members; there shall always be a floor system with floorbeams at the panel points.

53. End Floorbeams.—All bridges with floor systems shall preferably be provided with end floorbeams of the same cross-section as intermediate floorbeams. Where this is impossible, the end stringers shall rest on the masonry; end struts, of nearly the same depth as the end stringers, shall be riveted to them and to the girders or trusses.

54. Solid Floors.—Solid floors shall preferably be made of a plate not less than $\frac{7}{16}$ inch thick riveted to the tops of longitudinal I beams supported by floorbeams.

DETAILS OF PLATE GIRDERS

55. Stiffeners.—Webs shall have stiffeners over bearing points, at points of local concentrated loadings, and at intervals not greater than the depth of the girder nor more than 6 feet. Near the ends, the spacing of stiffeners shall be one-third to one-half the depth. They shall be composed of two angles, one on each side of the web, and shall fit tight between the horizontal legs of the flange angles. For girders having flange angles with outstanding or horizontal legs 5 inches in width, stiffeners shall be not less than 4 in. × 3½ in. × ⅜ in.; with outstanding legs 6 inches in width, not less than 5 in. × 3½ in. × ⅜ in.; and with outstanding legs 8 inches in width, not less than 6 in. × 3½ in. × ⅜ in. Rivets in stiffeners shall be spaced not over 3½ inches apart for a distance of 14 inches at each end, and not over 6 inches apart for the remaining distance. When the clear vertical distance between flange angles is less than fifty times the thickness of the web, stiffeners may be omitted—except over bearings, where they shall be designed to resist the reactions.

56. Web Splices.—Web-plates shall be spliced by one or more plates on each side; the splice plates shall have a section equal to at least three-fourths that of the web, and a pair of stiffeners shall be placed outside the plates. There shall be not less than two rows of rivets 3½ inches apart on each side of the splice; rivets in splice plates shall be spaced not over 3½ inches apart for a distance of 14 inches at top and bottom, and not over 4½ inches between. Web splices shall be designed for the same resisting moment as the web.

57. Flange Rivets.—The pitch of rivets connecting the flange angles to the web at any point shall be calculated by the formula

$$p = \frac{K h_r}{V}$$

in which p = pitch, in inches;

K = smallest value of the rivet, in pounds;

h_r = vertical distance, in inches, between centers of rivet lines of flanges;

V = total maximum vertical shear, in pounds, at the section.

When the ties rest on the top flanges of deck girders, the pitch of rivets in the top flange shall be 90 per cent. of the calculated pitch. The rivets connecting flange plates to flange angles shall have the same spacing as, and stagger with, those connecting the flange angles to the web. The spacing of rivets in plate-girder flanges shall in no case exceed 4½ inches.

58. Flanges.—Flanges shall be designed by the net section of the bottom flange and by the gross section of the top flange. One-eighth of the web shall be considered as part of the flange section. Girders with deep webs may have flanges composed of vertical as well as horizontal flange plates and secondary flange angles.

59. Flange Angles.—Flange angles shall preferably be of large sections; in general, not less than one-third to one-half the flange section shall be composed of angles.

60. Flange Plates.—Flange plates shall preferably have a thickness not greater than the angles, nor more than ¾ inch; when two or more plates are used, they shall have the same thickness, or shall diminish in thickness outwards from the angles, except the first plate in the top flange, which shall extend the full length of the flange, and may be thinner than the other plates. Other plates shall extend 12 inches at each end beyond their theoretical ends. Flange plates shall extend beyond the outer lines of rivets not more than 4 inches nor more than eight times the thickness of the thinnest plate.

61. Flange Splices.—Flange members of girders less than [illegible] feet long shall not be spliced. For girders longer than [illegible] feet each flange angle shall be spliced by two splice angles each having a cross-section [illegible] per cent. that of the flange angle and with sufficient rivets on each side of the splice to fully develop the stress in the splice angle. Flange plates shall preferably be spliced without additional splice plates by continuing the inner plates beyond their theoretical ends a sufficient distance to splice the lower plates. When splice plates are not in direct contact with the plates they splice, the calculated number of rivets shall be increased 20 per cent. for each intervening plate. Only one member shall be spliced at any section.

62. Riveting of Girders.—Deck girder bridges less than 70 feet long shall preferably be riveted up complete before shipping.

DETAILS OF RIVETED TRUSSES

63. Chord Members.—The chords and end posts shall be composed of channels, or of vertical plates and flange angles, connected by cover-plates at the top and by tie-plates and lattice bars at the bottom. Gusset plates for the connection of web members to chords shall be riveted to the inside of the chords, and shall be designed to resist the stresses to which they are subjected.

64. Web Members.—Web members shall intersect each other and the chords on lines passing through their centers of gravity, and shall be thoroughly riveted to each other and to the connection plates at every intersection. Web members shall be composed of symmetrical sections, preferably not less than 12 inches in width, connected by web-plates or by tie-plates and lattice bars. The clear distance between gussets shall be not more than $\frac{1}{8}$ inch greater than the width of the web member that connects to them.

65. Connections and Splices.—Splices of chords shall be as close as practicable to panel points. All splices of chords and connections of web members shall have enough rivets to develop fully the stress in the members. If a splice occurs at a joint, that part of the gusset in contact with the chord shall be counted as a splice plate.

66. Tension Members.—Tension members shall be of the same general form as compression members. The use of flat bars alone for riveted tension members will not be allowed.

DETAILS OF PIN-CONNECTED TRUSSES

67. Chord Members.—The top chord and end posts shall be composed of channels, or of vertical plates with flange angles, connected by cover-plates at the top and by tie-plates and lattice bars at the bottom, or by tie-plates and lattice bars at both top and bottom. Splices of top chords shall be as close as practicable to panel points, and shall have enough rivets to develop the stresses fully. The bottom chord shall be composed of eyebars; the inside bars in the two end panels shall be connected to each other by diaphragms or by lattice bars. The eyebars shall be packed on the pins as narrow as possible; those in any panel shall not be in contact, and shall not diverge from the center line of truss by more than $\frac{1}{16}$ inch per foot.

68. Web Members.—Web members shall intersect each other and the chords on lines passing through their centers of gravity, and pins shall be located at the intersections of these lines. Compression web members shall be composed of symmetrical sections, preferably not less than 12 inches in width, connected by web-plates or by tie-plates and lattice bars. Tension web members, except hip verticals and subverticals, shall be composed of eyebars. Hip verticals and subverticals shall be of the same general form as compression members.

69. Counters.—Counters shall be adjustable eyebars with screw ends and open turnbuckles. The area at the root of an upset screw end shall in no case be less than 10 per cent. greater than the body of the bar. No counter shall have a sectional area of less than 3 square inches.

70. Minimum Eyebars.—No eyebar shall be less than 4 inches in width or less than $\frac{3}{4}$ inch in thickness.

71. Pins.—Pins shall be not less than 3 inches in diameter, and shall project $\frac{1}{4}$ inch at each end beyond the outside surfaces of the members.

72. Riveted Tension Members.—Riveted tension members shall have a net section back of pinholes at least equal to the net section of the member, and through pinholes at least 25 per cent. greater.

73. Pin Plates.—Where necessary for section or bearing, members shall be reinforced at pinholes by pin plates. Each plate shall contain sufficient rivets to transmit its proportion of the bearing pressure to the member. One plate on each side shall extend at least 6 inches beyond the end of the tie-plate. The cross-section of a compression member through a pinhole shall be at least equal to that of the member.

DETAILS OF STEEL TRESTLES (VIADUCTS)

74. Towers and Main Spans.—Steel trestles shall consist of riveted spans on trestle bents braced in pairs to form towers. Tower spans shall be not less than 30 feet long, and shall be riveted to the tops of the trestle bents; main spans shall be riveted to the tops of the trestle bents at one end, and bolted to them at the other through expansion holes. In single-track trestles, the girders shall be connected to the tops of the columns; in double-track trestles, the outer lines of girders or trusses shall be connected to the tops of the columns, and the inner lines to cross-girders, the ends of which are connected to the tops of the columns.

75. Trestle Bents.—Trestle bents for single-track trestles shall be not less than 8 feet wide on top, and the batter of each post shall be not less than 1 horizontal to 6 vertical. Trestle bents for double-track trestles shall be not less than 19 feet 6 inches wide on top, and the batter of each post shall be not less than 1 horizontal to 8 vertical. On curves, towers shall be placed so that the center lines of the bents are at right angles to the chord of the curve between bents.

76. Towers.—Towers shall be divided into stories not more than 30 feet in height by horizontal struts and diagonal bracing between the columns.

77. Negative Reactions.—In estimating negative (downward) reactions at the feet of the columns, the weight of train shall be taken as 800 pounds per linear foot.

DETAILS OF BEARINGS

78. Bedplates.—The ends of all spans and the bottoms of columns of trestle bents shall rest on bedplates or pedestals, and shall be held in place by anchor bolts. Bedplates for girders and stringers shall be not less than 1 inch in thickness, and for trusses and columns not less than $1\frac{1}{2}$ inches. Holes for anchor bolts may be $\frac{1}{4}$ inch larger in diameter than the bolts.

79. Anchor Bolts.—Anchor bolts shall be not less than 1 inch in diameter for girders and stringers, nor less than $1\frac{1}{4}$ inches for trusses; they shall be set in holes drilled in the masonry, and the holes shall be filled with cement grout. Anchor bolts for columns having a negative reaction shall be designed to resist the reaction, and shall be built in a mass of masonry the weight of which is not less than twice the estimated reaction. Anchor bolts in expansion ends shall be so placed that the ends can move freely in the direction of expansion, and in no other direction.

80. Pedestals.—Spans over 75 feet in length shall have pin bearings and pedestals at both ends. Pedestals shall be built up of base and web plates not less than $\frac{3}{4}$ inch in thickness. The webs shall be secured to the base plates by angles not less than 6 in. $\times$ 4 in. $\times$ $\frac{1}{2}$ in., with the 6-inch leg vertical, and the webs shall be connected to each other. The pedestals shall be of sufficient height to distribute the load over the bearings.

81. Ends of Columns.—Caps and base plates shall be connected to the tops and bottoms, respectively, of all viaduct columns, by means of horizontal angles not less than 6 in. $\times$ 4 in. $\times$ $\frac{1}{2}$ in., with the 6-inch leg vertical or parallel to the batter of the column.

82. Rollers.—Spans over 75 feet in length shall have rollers at one end. Rollers shall be not less than 3 inches in diameter, and shall be turned down to a groove $\frac{1}{4}$ inch deep to fit guiding strips of this thickness on the bearing plates above and below the rollers. Special attention shall be given to roller bearings, so that they will not hold water, and so that they can be readily cleaned.

83. Adjacent Spans.—When the girders or trusses of two adjacent spans rest on the same pier, the bedplates and pedestals shall be entirely independent for each girder or truss.

84. Spans on Grade.—For spans without rocker bearings, a sole plate of the same size as the bedplate shall be riveted to the bottom of the span at each end; if the track is on a grade, the sole plate shall be planed to bevel, so that the lower surface will be level when the floor of the span is parallel to the grade.

DETAILS OF BRACING

85. Independent Bracing.—All spans shall be independently braced; no bracing shall be used in common for any two adjacent spans.

86. Style of Members.—Members of bracing shall either be built-up members or be composed of simple rolled shapes. They shall intersect each other, and the members to which they connect, on lines passing as nearly as practicable through their centers of gravity, and shall be riveted to each other and to connection plates at every intersection. No member shall be less than $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in., and no connection shall have less than four rivets.

87. Lateral Bracing.—Top and bottom lateral bracing shall be provided in deck and through bridges; bottom lateral bracing in half-through bridges. Lateral bracing shall be riveted to the stringers of the floor system wherever it comes in contact with them. Deck girders shall have the top lateral bracing so arranged that the length of the flange between lateral connections will not exceed twelve times its width. If stringers are longer than twelve times the width of the flange, lateral bracing shall be riveted to their upper flanges.

88. Transverse Bracing.—Deck girder bridges shall have transverse frames, of the same depth as the girders, riveted to the stiffeners near the ends, and at other points at distances apart not greater than 15 feet. If stringers are longer than twenty times the width of the flange, transverse frames shall be riveted to their webs at the intersections of the stringers and lateral bracing. Deck truss bridges shall have sway-bracing, of the same depth as the trusses, at every panel point.

89. Knee Bracing.—Half-through bridges shall have brackets or knee braces riveted to the floorbeams or the tops of solid floors, and to the webs of the girders. Knee braces shall fit tight under the top flange angles of girders, and shall be as wide at the top of the rail as the clearance will allow. They shall be so arranged that the distance between them shall not exceed twelve times the width of the top flange of the girder.

90. Portal Bracing.—Through bridges shall have portals and portal brackets, and intermediate brackets at each transverse strut of the upper lateral bracing. Portals shall be as deep as the specified clearance will allow. Where the headroom above the track is 25 feet or more, sway frames shall be provided at every panel point of the top chord; they shall be as deep as the required headroom will allow.

DESIGN OF HIGHWAY AND STREET-RAILWAY BRIDGES

GENERAL DIMENSIONS

91. Kinds of Bridges.—The following kinds of bridges shall preferably be used:

For spans less than 35 feet in length, rolled beams.

For spans from 35 to 100 feet in length, plate girders.

For spans from 100 to 150 feet in length, riveted trusses.

For spans longer than 150 feet, pin-connected trusses.

If, for any reason, it is desired to depart more than 10 feet from these limits, permission in writing must be obtained from the Engineer.

92. Panel Lengths and Depths.—The depth of girders shall preferably be not less than one-twelfth the span. Panel lengths shall preferably be from 15 to 30 feet, and in truss bridges the panel lengths and depths shall be so chosen that the inclined web members shall make an angle with the lower chord of not less than 50°. The depth of I beams shall in no case be less than one-thirtieth of the span.

93. Spacing of Stringers, Girders, and Trusses. For bridges carrying only a railway track, stringers of floor systems, and deck girders less than 70 feet long, shall be spaced 6 feet 6 inches center to center. Deck girders over 70 feet long shall be spaced 6 inches farther apart for each 10 feet increase in length. Stringers, girders, and trusses in bridges carrying both railways and highways, or highways only, shall be arranged to accommodate the actual traffic, and shall be adapted to local conditions. Trusses shall be spaced not less than one-twentieth of the span.

94. Clearance.—No part of any bridge shall come closer to the center line of the nearest track than is shown in outline in Fig. 4. If a track is on a curve, ½ inch additional clearance for each degree of curvature shall be provided on the outside of the curve, and on the inside of the curve ¼ inch additional clearance for each degree of curvature, and 2 inches for each inch of superelevation of track. All through highway bridges carrying railways shall have a clear headroom of 15 feet at a distance of 3 feet from the wheel-guards; those carrying highways only shall have a clear headroom of not less than 13 feet at a distance of 3 feet from the wheel-guards.

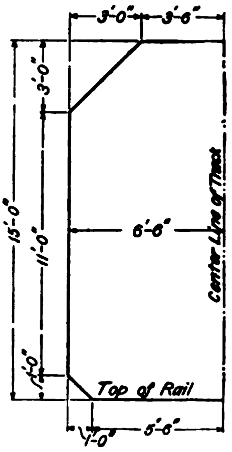

FIG. 4

95. Spacing of Tracks.—When there is more than one track, the tracks shall be assumed as 10 feet center to center.

LOADING

96. Loads.—Bridges carrying highways only shall be designed to resist properly the stresses caused by the following forces: *dead load*, *live* or *moving load*, and *wind pressure*. Bridges carrying railways alone or in connection with highways shall be designed to resist properly the stresses caused by the forces mentioned and, in addition, those caused by *impact* and *vibration*, *centrifugal force*, and *the longitudinal force due to suddenly stopping cars*.

97. Dead Load.—The dead load shall consist of the estimated weight of the entire structure. The actual weight of the floor and track, if any, shall be computed; timber shall be assumed to weigh 4½ pounds per board foot. For bridges carrying railways alone, the weight of rails, ties, etc. may be taken as 300 pounds per linear foot of track. In truss bridges, one-half the weight of the trusses shall be assumed as applied at the loaded chord, and one-half at the unloaded chord; the entire weight of floor shall be assumed as applied at the loaded chord.

98. Live Load.—The live load shall consist of the estimated maximum moving loads that the bridge is expected to carry. It will depend on the amount and kind of traffic to which the bridge is to be subjected, and shall in general be assumed as follows:

1. *For City Bridges Subject to Heavy Loads.*—For the hip verticals, subverticals, short diagonals, floor hangers, and floor members of all spans, either a uniform load of 100 pounds per square foot on all parts of the floor, or a steam road roller weighing 20 tons distributed as represented in Fig. 5. For the girders or trusses of all spans up to 100 feet, a uniform load of 100 pounds per square foot on the entire surface of the floor; of all spans over 200 feet, 80 pounds per square foot; and of intermediate spans, proportional intermediate values. (Between 100 and 200 feet, the uniform load decreases 1 pound per square foot for each 5 feet increase in span.)

2. *For Bridges in the Suburbs of Cities and in Well-Settled Town Districts.*—For the hip verticals, subverticals, short diagonals, floor hangers, and floor members of all spans, either a uniform load of 100 pounds per square foot on all parts of the floor, or a steam road roller weighing 15 tons distributed as represented in Fig. 6. For the girders or trusses of all spans up to 100 feet, a uniform load of 80 pounds per square foot on the entire surface of the

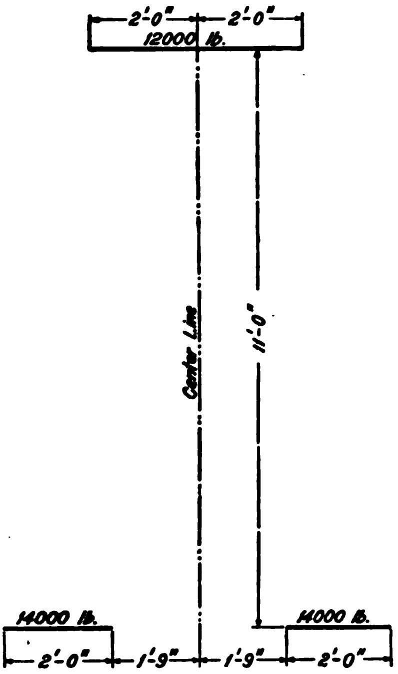

Fig. 5

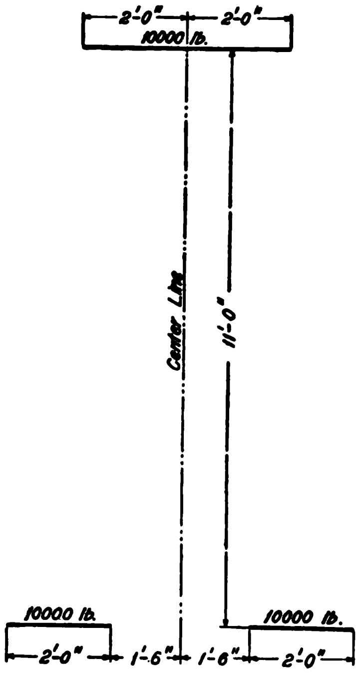

Fig. 6

floor; of all spans over 200 feet, 60 pounds per square foot; and of intermediate spans, proportional intermediate values. (Between 100 and 200 feet the uniform load decreases 1 pound per square foot for each 5 feet increase in span.)

3. *For Bridges in Country Districts and in Thinly Settled Communities.*—For the hip verticals, subverticals, short diagonals, floor hangers, and floor members and connections

of all spans, either a uniform load of 80 pounds per square foot on all parts of the floor, or a steam road roller weighing 15 tons distributed as represented in Fig. 6. For the girders or trusses of all spans up to 75 feet, a uniform load of 80 pounds per square foot on the entire surface of the floor; of all spans over 200 feet, 55 pounds per square foot; and of intermediate spans, proportional intermediate values. (Between 75 and 200 feet, the uniform load decreases 1 pound per square foot for each 5 feet increase in span.)

4. *For All Bridges Carrying Street Railway, or That Are Expected to Carry Street Railway in the Near Future.*—For the hip verticals, subverticals, short diagonals, floor hangers, and floor members of all spans, and for the girders of spans less than 75 feet, an electric car on each track weighing 40 tons, distributed as shown in Fig. 7; for the web members of spans 75 to 100 feet in length, 1,600 pounds per linear foot, and of all spans over 100 feet, a floorbeam load of $(1{,}600\,p)$ pounds for each track at $\left(\frac{100}{p}\right)$ floorbeams (p being the panel length, in feet); for the chord members of spans 75 feet in length, 1,600 pounds per linear foot; of spans 275 feet or more in length, 1,000 pounds per linear foot; and of intermediate spans, proportional intermediate values. (Between 75 and 275 feet, the uniform load decreases 3 pounds for each 1 foot.)

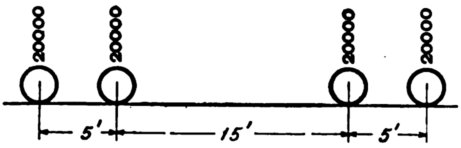

FIG. 7

5. *For Bridges Carrying Both Highway and Street Railway.*—The live load shall consist of the electric car or uniform load given in item 4 above, together with the uniform loads given in items 1, 2, or 3 covering the entire floor, except a width of 10 feet for each track.

99. Impact and Vibration.—To provide for impact and vibration in bridges carrying street railways, an amount I is to be added to the stress or bending moment caused by the car in each member, as given by the following formulas:

For counters, hip verticals, subverticals, short diagonals, floor hangers, floor members and connections, members subject to reversal of stress, and all other members for which L is less than 25 feet,

$$I = \tfrac{3}{10}\,S$$

For members for which L is greater than 25 feet and less than 200 feet,

$$I = \frac{300 - L}{1{,}000}\,S$$

For all members for which L is greater than 200 feet,

$$I = \tfrac{1}{10}\,S$$

Here, S = maximum live-load stress or bending moment in the member due to load on car track;
L = length of track, in feet, that must be loaded in order to obtain the value S.

100. Wind Pressure.—Wind pressure shall be assumed in girder bridges as 50 pounds per square foot on the exposed area of one girder, and in truss bridges as 50 pounds per square foot on twice the exposed area of one truss together with the exposed area of the floor. In designing the stringers of floor systems in bridges carrying railway, the wind pressure on the cars shall be assumed to be 250 pounds per linear foot, applied 6 feet above the rail, and the center of moments for the increase in load on the leeward stringer shall be taken at the top of the rail.

101. Centrifugal Force.—In bridges on curves, the centrifugal force F caused by electric cars shall be assumed to act 5 feet above the rail, and shall be found by the formula

$$F = \left(\frac{1.5 - .05\,D}{100}\right) D\,W$$

in which D = degree of curve;
W = live load.

102. Suddenly Stopping Cars.—The longitudinal force due to a suddenly stopping car shall be taken equal to 16,000 pounds applied at the top of the rail.

DESIGN OF MEMBERS

103. Working Stresses.—All parts shall be so designed that the sum of the maximum stresses shall not cause the intensities of stress to exceed the following values, in pounds per square inch:

Tension on net section, 16,000.

Compression on gross section,

$$\frac{16{,}000}{1 + \dfrac{l^2}{18{,}000\, r^2}}$$

in which l = unsupported length of member, in inches;
r = least radius of gyration, in inches.

In half-through truss bridges, the entire length of the upper chord shall be considered unsupported laterally, the stiffening effect of knee braces being ignored.

Shear on net sections of web plates,

$$\frac{12{,}000}{1 + \dfrac{d^2}{3{,}000\, t^2}}$$

in which t = thickness of the web, in inches;
d = clear distance, in inches, between stiffeners or flange angles, whichever is the smaller.

The intensity of shearing stress found by dividing the total vertical shear by the gross area of cross-section of the web shall in no case exceed 9,000.

Shear on shop rivets and pins, 11,000.

Shear on field rivets and turned bolts, 9,000.

Bending on pins, 22,000.

Bending on rolled sections and plate girders:

Tension, 16,000.

Compression, when unsupported length l of compression flange is not greater than twenty times the width w, 16,000.

Compression, when l is greater than 20 w,

$$20{,}000 - 200\,\frac{l}{w}$$

Bearing on shop rivets and pins, 22,000.

Bearing on field rivets, turned bolts, and ends of stiffeners, 18,000.

Bearing on masonry:

Sandstone and limestone, 300.

Cement concrete, 400.

Granite, 500.

The bearing on rollers shall not exceed 600 D pounds per linear inch of roller, in which D is the diameter of roller, in inches.

Bending on fir, yellow-pine, and white-oak beams, 1,200.

Bending on white-pine and spruce beams, 1,000.

104. Data.—The following general dimensions shall first be calculated or assumed:

Length of trusses, center to center of end pins or pedestals.

Length of girders, center to center of bearings.

Length of floorbeams, center to center of girders or trusses.

Length of stringers, center to center of floorbeams.

Depth of trusses and girders, center to center of gravity of chords or flanges.

105. Compression Members.—Pin and bolt holes shall be deducted from the gross section of compression members. The value of $\frac{l}{r}$ will preferably be from 40 to 60, and must not exceed 100 for main members, nor 120 for members of lateral systems. Splices in compression members shall have sufficient rivets to develop fully the stresses in the members.

106. Tension Members.—The net section of a riveted tension member shall be determined by deducting from the gross section the area of cross-section of the greatest number of pin, bolt, or rivet holes that can be cut by a plane at right angles to the member. In addition, for rivets $\frac{7}{8}$ inch in diameter and larger, there shall be deducted each hole whose center lies within $\frac{3}{4}$ inch of the cutting plane and a proportional part of each hole whose center lies within $2\frac{3}{4}$ inches; and, for rivets $\frac{3}{4}$ inch in diameter and smaller, there shall be deducted each hole whose center lies within $\frac{1}{2}$ inch and a proportional part of each hole whose center lies within 2 inches. Rivet holes shall be taken $\frac{1}{8}$ inch larger in diameter than the rivets.

107. Reversal of Stress.—Members subject to both tension and compression shall be designed to resist each stress plus eight-tenths of the other stress.

108. Combined Stresses.—Members subject to transverse stresses in addition to the direct stresses shall be designed for both kinds of stress.

109. Bearing Values of Rivets.—In calculating the bearing value of a rivet, the area subjected to stress shall be taken equal to the product of the thickness of the plate and the diameter of the rivet before driving, that is, the nominal diameter. The value of countersunk rivets shall not be counted.

110. Floor Stringers.—In bridges with steel stringers, each stringer shall be designed to support one-half the load on a front wheel of a steam roller and one-half the load on one rear roller, forming a system of two concentrated loads. In bridges with wooden joists, the loads on each roller of a steam road roller may be assumed to be distributed over a width 12 inches greater than the width of the roller, and the portion that goes to each stringer or joist calculated on this basis.

GENERAL DETAILS

111. Details and Connections.—Special attention shall be given to all details and connections, which shall always be of greater strength than the body of the member. All details shall be accessible for inspection, cleaning, and painting. Details that permit the collection of water shall be avoided if possible; if used, they shall be provided with drainage holes or filled with cement concrete.

112. Minimum Thickness.—No material less than $\frac{5}{16}$ inch thick shall be used except for latticing and fillers, and for webs of channels, which may be $\frac{1}{4}$ inch thick. If the bridge is over a steam railroad, no material less than $\frac{3}{8}$ inch thick shall be used below the floor, except for buckled plates, for which $\frac{5}{16}$-inch material may be used under sidewalks.

113. Single Angles.—Members or sides of members composed of single angles shall have both legs of each angle connected at the ends, or only 75 per cent. of the section shall be counted. No angle shall be smaller than $2\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. $\times$ $\frac{5}{16}$ in., nor be connected by less than three rivets, except for unimportant details.

114. Sizes of Rivets.—Rivets shall generally be either $\frac{7}{8}$ inch or $\frac{3}{4}$ inch in diameter. The diameter of the rivet shall not be greater than one-quarter the width of the bar or angle through which it passes, except for unimportant details, where $\frac{7}{8}$-inch rivets may be used in 3-inch angles, and $\frac{3}{4}$-inch rivets in $2\frac{1}{2}$-inch angles.

115. Spacing of Rivets.—The distance center to center of rivet holes shall be not less than three times the diameter of the rivets, nor more than 6 inches in any line. The centers of rivet holes shall not be closer to the edge of any piece than $1\frac{1}{2}$ times the diameter of the rivet. The spacing of rivets at the ends of compression members shall not exceed four times the diameter of the rivet for a distance equal to the width of the member. For the remainder of the length of compression members, the distance center to center of rivets shall be not greater than sixteen times the thickness of the thinnest outside plate.

116. Compression Members.—In compression members, the material shall be concentrated at the sides as much as possible. The unsupported width of plates shall not exceed thirty times their thickness for web-plates, nor forty times for cover-plates of chords and end posts. No closed sections will be allowed.

117. Tie-Plates and Lattice Bars.—The open sides of all built-up members shall be stiffened by means of tie-plates and lattice bars. The length of tie-plates shall be not less than their width. Double latticing shall preferably make an angle of about 45° with the axis of the member, and the bars shall be riveted where they cross each other. Single latticing shall preferably make an angle of about 60° with the axis of the member. The thickness of tie-plates shall be not less than one-fortieth the distance between their rivet lines; the thickness of bars in single latticing shall be not less than one-fortieth, and in double latticing one-sixtieth of the distance between the rivets in their ends. Lattice bars shall be not less than $2\frac{1}{4}$ inches wide for members up to 9 inches, not less than $2\frac{1}{2}$ inches for members from 9 to 15 inches, nor less than 3 inches for members more than 15 inches in width or depth.

118. Expansion.—Provision for expansion and contraction shall be made at the rate of 1 inch for every 100 feet of span.

119. Camber.—All trusses shall be cambered by giving the panels of the top chord an excess of length in the proportion of $\frac{1}{8}$ inch for each 10 feet. Plate girders shall not be cambered.

DETAILS OF FLOOR SYSTEMS

120. Wooden Floors.—In bridges constructed solely for street-railway purposes, ties shall be of yellow pine not less than 5 inches wide, 7 inches deep, and 10 feet long, and shall be framed to $6\frac{1}{2}$ inches over bearings for stringers and girders 6 feet 6 inches center to center. For girders over 6 feet 6 inches center to center, the depth of tie shall be increased $\frac{1}{2}$ inch for each 6-inch increase in the spacing of the girders. Ties shall be spaced not more than 12 inches center to center, and every fourth tie shall be fastened at both ends to the stringers or girders by $\frac{3}{4}$-inch bolts. Guard timbers of yellow pine not less than 6 in. × 6 in., framed to 4 inches over ties, shall be bolted by $\frac{5}{8}$-inch bolts to the ties that are bolted to the stringers, and so that the inner edge of the guard timber shall be 4 feet from the center of the track. Guard timbers shall extend over all piers and abutments.

Superelevation of rails on curves shall be provided for as may be required in each case.

121. In bridges with wooden floors constructed for combined street-railway and highway purposes, the rails shall preferably be supported on yellow-pine ties not less than 6 in. × 6 in. in size, nor less than 8 feet long, spaced not over 15 inches center to center and resting on the stringers. Longitudinal nailing pieces not over 2 feet apart shall be spiked to the top of the tie; they shall be not less than 3 inches wide, and of sufficient height to bring the top of the plank floor level with the top of the rails. A plank floor not less than 2 inches thick, preferably of spruce or oak, shall be nailed to the nailing pieces.

122. For bridges in country districts and in thinly settled communities, the highway portion of the floor shall preferably consist of one layer of white-oak plank not less than 3 inches in thickness laid at right angles to the trusses or girders, and with joints about $\frac{1}{4}$ inch open. Wooden stringers not less than 3 in. × 12 in. or steel beams with wooden nailing pieces shall be used, and the former shall be spaced not over 2 feet 6 inches center to center. In general, the width of wooden stringers shall be not less than one-fourth of the depth. The plank floor shall be securely spiked to the stringers.

123. For bridges in the suburbs of cities, in well-settled town districts, and in some cases for city bridges not subject to heavy loads, the highway portion of the floor shall preferably consist of two layers of plank; the lower layer shall be of white oak not less than 3 inches in thickness, and shall be laid diagonally with joints not over $\frac{1}{2}$ inch open; the upper layer shall be of white oak or spruce 2 inches thick, laid tight at right angles to the girders or trusses and securely spiked to the lower layer. Wooden stringers, as before, may be used, but steel stringers with wooden nailing pieces shall have the preference. When one layer of floor plank is used, the distance, in feet, between the centers of joists or nailing pieces shall not be greater than the thickness of the plank, in inches. When more than one layer is used, the clear distance, in feet, between joists or nailing pieces shall not be greater than the thickness of the lower layer, in inches.

124. Paved Floors.—For city bridges subject to heavy loads, the floor shall preferably consist of prepared wooden blocks, asphalt, brick, or granite blocks. Wooden blocks shall be given the preference; granite blocks shall be used only in the immediate vicinity of warehouses, docks, or freight houses, where the traffic is exceedingly heavy and continuous. Paved floors shall be supported on buckled plates securely riveted to the upper flanges of stringers and to the floorbeams. Buckled plates shall be laid with the buckle hanging down, and shall be covered with cement concrete having a thickness not less than 3 inches under the roadway nor less than 2 inches under the sidewalk. Between the concrete and the paving there shall be spread a cushion coat of clean, sharp sand, perfectly free from moisture, to an even thickness of 1 inch. Open joints between blocks shall be filled with cement grout or coal-tar pitch.

125. Wheel-Guards.—In bridges with wooden floors, a wheel-guard of timber not less than 6 inches wide and 4 inches thick shall be placed longitudinally on the floor. The upper edge of the wheel-guard shall be 6 inches from the surface of the floor. The edge of the guard toward the roadway shall be 6 inches from the clearance line of the trusses or girders. In bridges with paved floors, a metal or stone wheel-guard shall be provided; it shall be 6 inches high and at least 6 inches outside of the clearance line of the trusses or girders.

126. Floor Connections.—Stringers shall be at right angles to the floorbeams, and shall be riveted to the floorbeam webs. If possible, the stringers shall also rest on shelf angles riveted to the webs of the floorbeams. Floorbeams shall preferably be riveted to the webs of the girders in half-through girder bridges, and to the vertical posts or web connection plates in through truss bridges; in deck truss bridges, the floorbeams shall either be riveted to the trusses, as in through bridges, or rest on the top chords. Where sidewalks are supported outside the girders or trusses, the loorbeams shall be extended under the sidewalks, or sidewalk brackets shall be riveted to the outsides of the posts, and their top flanges connected to those of the floorbeams.

The connection angles of stringers to floorbeams and of floorbeams to girders or trusses shall be not less than 3 in. × 3 in. × $\frac{7}{16}$ in. The fillers under connection angles shall be twice as wide as the adjacent leg of the angle, and shall have two-thirds as many rivets through the projecting portion as through the portion under the angle.

127. Hand Railing.—A suitable hand railing or fence shall be placed at each side of the bridge. The railing shall be not less than 3 feet 6 inches above the top of the floor, and have not more than 6 inches clearance beneath it.

DETAILS OF PLATE GIRDERS

128. Stiffeners.—Webs shall have stiffeners over bearing points, at points of local concentrated loadings, and at intervals not greater than the depth of the girder nor more than 6 feet. Near the ends, the spacing of stiffeners shall be one-third to one-half the depth. Stiffeners shall be composed of two angles, one on each side of the web, and shall fit tight between the horizontal legs of the flange angles. For girders having flange angles with outstanding or horizontal legs 5 inches in width, stiffeners shall be not less than $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{5}{16}$ in.; with outstanding legs 6 inches in width, not less than 4 in. × $3\frac{1}{2}$ in. × $\frac{5}{16}$ in.; with outstanding legs 8 inches in width, not less than 5 in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in. Rivets in stiffeners shall be spaced not over $4\frac{1}{2}$ inches apart for a distance of 18 inches at each end, and not over 6 inches apart for the remaining portion. When the clear vertical distance between flange angles is less than fifty times the thickness of the web, stiffener angles may be omitted, except over bearings. Stiffeners over bearings shall be designed to resist the reactions.

129. Web Splices.—Web-plates shall be spliced by one or more plates on each side, the plates on each side to have a section equal to three-fourths that of the web, and a pair of stiffeners placed outside the plates. There shall be not less than two rows of rivets 3½ inches apart on each side of the splice. Rivets in splice plates shall be spaced not over 3½ inches apart for a distance of 14 inches at top and bottom, and not over 5 inches between. Web splices shall be designed for the same resisting moment as the web.

130. Flange Rivets.—The pitch of rivets connecting the flange angles to the web at any point shall be calculated by means of the formula $p = \frac{K h_r}{V}$ (see Art. **57**). When the ties of street-railway bridges rest on the top flanges of deck girders, the pitch of the rivets in the top flange shall be 90 per cent. of the calculated pitch. The rivets connecting flange plates to flange angles shall have the same spacing as, and stagger with, those connecting the flange angles to the web. The spacing of rivets in plate-girder flanges shall in no case exceed 6 inches.

131. Flanges.—Flanges shall be designed for the net section of the bottom flange, and the gross section of the top flange. One-eighth of the web shall be considered as part of the flange section. Girders with deep webs may have flanges composed of vertical as well as horizontal flange plates.

132. Flange Angles.—Flange angles shall preferably be of large sections; in general, not less than one-third to one-half the flange section shall be composed of angles.

133. Flange Plates.—Flange plates shall preferably have a thickness not greater than the angles nor more than ¾ inch. When two or more plates are used, they shall have the same thickness, or shall diminish in thickness outwards from the angles, except the first plate of the top flange, which shall extend the full length of the flange and may be thinner than the other plates. Other plates shall extend 12 inches at each end beyond their theoretical ends. Flange plates shall extend beyond the outer lines of rivets not more than 4 inches nor more than eight times the thickness of the thinnest plate.

134. Flange Splices.—Flange members of girders less than 70 feet long shall not be spliced. For girders longer than 70 feet, each flange angle shall be spliced by two splice angles, each having a cross-section 75 per cent. of that of the flange angle, and with sufficient rivets on each side of the splice to develop fully the stress in the splice angle. Flange plates shall preferably be spliced without additional splice plates, by continuing the outer plates beyond their theoretical ends a sufficient distance to splice the lower plates. When splice plates are not in direct contact with the plates they splice, the calculated number of rivets shall be increased 20 per cent. for each intervening plate. Only one member shall be spliced at any section.

DETAILS OF RIVETED TRUSSES

135. Chord Members.—The chords and end posts shall be composed of channels, or of vertical plates and flange angles, connected by cover-plates at the top, and by tie-plates and lattice bars at the bottom. Gusset plates for the connection of web members to chords shall be riveted to the inside of the chords, and shall be designed to resist the forces to which they are subjected.

136. Web Members.—Web members shall intersect each other and the chords on lines passing through their centers of gravity, and shall be thoroughly riveted to each other and to the connection plates at every intersection. Web members shall be composed of symmetrical sections, connected by web-plates or tie-plates and lattice bars. The clear distance between gussets shall be not more than $\frac{1}{8}$ inch greater than the width of the web members that connect to them.

137. Connections and Splices.—Splices of chords shall be as close as practicable to panel points. All splices of chords and connections of web members shall have enough rivets to develop fully the strength of the members. If a splice occurs at a joint, that part of the gusset in contact with the chord shall be counted as a splice plate.

138. Tension Members.—Tension members shall be of the same general form as compression members. The use of flat bars alone for riveted tension members will not be allowed.

DETAILS OF PIN-CONNECTED TRUSSES

139. Chord Members.—The top chord and end posts shall be composed of channels, or of vertical plates with flange angles, connected by cover-plates at the top, and by tie-plates and lattice bars at the bottom, or by tie-plates and lattice bars at both top and bottom. Splices of top chords shall be as close as practicable to panel points, and shall have enough rivets to develop fully the stresses in the members. The bottom chord shall be composed of eyebars; the inside bars in the two end panels shall be connected to each other by diaphragms or by lattice bars. The eyebars shall be packed on the pins as narrow as possible; those in any panel shall not be in contact, and shall not diverge from the center line by more than $\frac{1}{16}$ inch per foot.

140. Web Members.—Web members shall intersect each other and the chords on lines passing through their centers of gravity, and pins shall be located at the intersections of these lines. Compression web members shall be composed of symmetrical sections, connected by web-plates or by tie-plates and lattice bars. Tension web members, except hip verticals and subverticals, shall be composed of eyebars. Hip verticals and subverticals shall be of the same general form as compression members.

141. Counters.—Counters shall be adjustable eyebars with screw ends and open turnbuckles. The area at the root of an upset screw end shall in no case be less than 10 per cent. greater than the cross-sectional area of the body of the bar. No counter shall have a sectional area of less than 2 square inches.

142. Pins.—Pins shall be not less than $2\frac{1}{2}$ inches in diameter, and shall project $\frac{1}{4}$ inch at each end beyond the outside surfaces of the members.

143. Riveted Tension Members.—Riveted tension members shall have a net section back of the pinholes equal to that of the member, and through the pinholes 25 per cent. greater.

144. Pin Plates.—Where necessary for section or bearing, members shall be reinforced at pinholes by pin-plates. Each plate shall contain sufficient rivets to transmit its proportion of the bearing pressure to the members; one plate on each side shall extend at least 6 inches beyond the end of the tie-plate. The cross-section of a compression member through the pinhole shall be equal to that of the member.

DETAILS OF STEEL TRESTLES

145. Tower and Main Spans.—Steel trestles shall consist of riveted spans on trestle bents, braced in pairs to form towers. Tower spans shall be not less than 30 feet long, and shall be riveted to the tops of the trestle bents. Main spans shall be riveted to the tops of the trestle bents at one end, and bolted to them at the other through expansion holes. Girders and trusses may be connected to the tops of the columns or to cross-girders that are connected to the tops of the columns.

146. Trestle Bents.—The batter of columns in trestle bents shall be not less than 1 horizontal to 8 vertical. In trestles for single-track railway only, bents shall be not less than 8 feet wide on top, and if the width is less than 10 feet, the batter of the columns shall be not less than 1 horizontal to 6 vertical. On curves, towers shall be placed so that the center lines of the bents are at right angles to the chord of the curve between the bents.

147. Towers.—Towers shall be divided into stories not more than 30 feet in height, by horizontal struts and diagonal bracing between the columns.

148. Negative (Downward) Reactions.—In estimating negative (downward) reactions at the feet of the columns, wind pressure shall be assumed to have an intensity of 50 pounds per square foot, acting on an area equal to twice the exposed area of the unloaded trestle.

DETAILS OF BEARINGS

149. Bedplates.—The ends of all spans, and the bottoms of columns of trestle bents, shall rest on bedplates or pedestals, and shall be held in place by anchor bolts. Bedplates for girders and stringers shall be not less than $\frac{3}{4}$ inch in thickness; and for trusses and columns, not less than 1 inch. Holes for anchor bolts may be $\frac{1}{4}$ inch larger in diameter than the bolts.

150. Anchor Bolts.—Anchor bolts shall be not less than 1 inch in diameter; they shall be set in holes drilled in the masonry, and the holes shall be filled with cement grout. Anchor bolts for columns having negative reactions shall be designed to resist the reaction, and shall be built in a mass of masonry the weight of which is not less than twice the estimated reaction. Anchor bolts in expansion ends shall be so placed that the ends can move freely in the direction of expansion and in no other direction.

151. Pedestals.—Spans over 75 feet in length shall have pin bearings and pedestals at both ends. Pedestals shall preferably be built up of base plates and web-plates, not less than $\frac{5}{8}$ inch in thickness. The webs shall be secured to the base plates by angles not less than 5 in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{1}{2}$ in., with the 5-inch leg vertical, and the webs shall be connected to each other. Pedestals shall be of sufficient height to distribute the load over the bearings.

152. Ends of Columns.—Cap and base plates shall be connected to the tops and bottoms, respectively, of all trestle columns by means of horizontal angles not less than 5 in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{1}{2}$ in., with the 5-inch leg vertical or parallel to the batter of the column.

153. Rollers.—Spans over 75 feet in length shall have rollers at one end. Rollers shall be not less than 3 inches in diameter, and shall be turned down to a groove $\frac{1}{8}$ inch deep to fit guiding strips of this thickness on the bearing plates above and below the rollers. Special attention shall be given to roller bearings, so that they will not hold water and so that they can be readily cleaned.

DETAILS OF BRACING

154. All spans shall be independently braced; no bracing shall be used in common for any two adjacent spans.

155. Style of Members.—Members of bracing shall be composed of simple rolled sections or built-up members. No member shall be less than $2\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. $\times$ $\frac{5}{16}$ in., and no connection shall have less than three rivets.

156. Lateral Bracing.—Top and bottom lateral bracing shall be provided in all deck and through bridges, except deck plate girders less than 5 feet deep, where the bottom bracing may be omitted. Bottom lateral bracing shall be provided in half-through bridges. Lateral bracing shall be riveted to the stringers of the floor system wherever they come in contact with them. Deck girders shall have the top lateral bracing so arranged that the length of flange between lateral connections will not exceed twenty times the width of flange.

157. Transverse Bracing.—Deck girder bridges shall have transverse frames of the same depth as the girders riveted to the stiffeners near the ends and at other points at distances apart not greater than 20 feet. Deck truss bridges shall have at every panel point sway-bracing of the same depth as the trusses.

158. Knee Bracing.—Half-through bridges shall have brackets or knee braces riveted to the floorbeams and to the webs of the girders. The brackets shall fit tight under the flange angles of the girders, and extend out to within 3 inches of the edge of the wheel-guard.

159. Portal Bracing.—Through bridges shall have portals and portal brackets, and intermediate brackets at each transverse strut of the upper lateral bracing. Portals shall be as deep as the specified headroom will allow. Where the headroom above the floor is 20 feet or more, sway frames shall be provided at every panel point of the top chord. They shall be as deep as the required headroom will allow.

SPECIFICATIONS FOR THE CONSTRUCTION OF STEEL BRIDGES

MATERIALS

CHEMICAL AND PHYSICAL PROPERTIES

160. Kind of Material.—The materials used in the construction of bridges shall generally be rolled steel, steel castings, and wrought iron. In case other materials are required, additional specifications will be furnished relating to them.

161. Grades of Steel.—Steel shall be made only by the open-hearth process. Rolled steel shall be of two grades; namely, *rivet steel* and *structural steel.* Rivet steel shall be used for rivets; structural steel shall be used for all other purposes, unless steel castings or wrought iron are especially called for.

162. Rivet Steel.—Rivet steel shall contain not more than .04 per cent. of phosphorus, nor more than .04 per cent. of sulphur. It shall have an ultimate tensile strength of not less than 46,000, nor more than 54,000, pounds per square inch, an elastic limit of not less than 60 per cent. of the ultimate strength, an elongation of not less than 28 per cent., including the break, and a reduction of area of not less than 55 per cent. Rivet rods shall bend double, cold, one side flat on the other, without cracking of the outer fibers.

163. Structural Steel.—Structural steel shall contain not more than .08 per cent. of phosphorus for acid steel, nor more than .04 for basic steel, and not more than .04 per cent. of sulphur for either kind of steel. It shall have an ultimate tensile strength of not less than 56,000, nor more than 64,000, pounds per square inch, an elastic limit of not less than 60 per cent. of the ultimate strength, an elongation of not less than 28 per cent., and a reduction of area at fracture of not less than 50 per cent. The fracture shall appear fine-grained, silky, and bluish gray, and shall be entirely free from hard and granular spots.

Test pieces shall bend double, cold, until the surfaces touch each other, without cracking of the outer fibers. They shall stand punching and cold reaming to 1¼ times the diameter of the punched hole without cracking the edges of the hole. Angles of all thicknesses shall, while cold, open flat, and if under ⅝ inch thick shall bend close shut without showing signs of fracture.

164. Steel Castings.—Steel castings shall contain not more than .08 per cent. of phosphorus for acid steel, nor more than .05 per cent. for basic steel, nor more than .05 per cent. of sulphur for either kind of steel. They shall have an ultimate tensile strength of not less than 60,000 pounds per square inch, an elastic limit of not less than 60 per cent. of the ultimate strength, an elongation of not less than 18 per cent., and a reduction of area at fracture of not less than 25 per cent. Steel castings shall be fine-grained, homogeneous, and free from blowholes and other defects. A test piece 1 in. × ½ in. shall bend cold through an angle of 90° around a rod whose diameter is 1½ inches, without showing signs of fracture.

165. Wrought Iron.—Wrought iron shall be, as nearly as practicable, the best grade of pure iron. It shall be double-rolled, fibrous, tough, uniform in character, and thoroughly welded in rolling. It shall be entirely free from surface defects. It shall have an ultimate tensile strength of not less than 50,000 pounds per square inch, an elastic limit of not less than 60 per cent. of the ultimate strength, an elongation, including the break, of not less than 18 per cent., and a reduction of area at fracture of not less than

25 per cent. Test pieces shall bend, when cold, through an angle of 180° around a rod whose diameter is twice the thickness of the test piece, without showing signs of fracture.

MILL TESTS

166. Reports of Tests and Melt Numbers.—The Contractor shall furnish the Engineer, free of charge, a report showing the physical and chemical properties of every melt that goes into the material of the bridge. The chemical analysis shall show the amount of carbon, phosphorus, sulphur, and manganese contained in each melt. The melt numbers shall be clearly stamped on all finished material, and omission to do so, or changes, or confusion of the melt numbers may be cause for rejection.

167. Test Pieces.—Two test pieces shall be cut from the finished material of every melt. Test pieces of rivet steel shall be round rods having the same diameter as the rivets. Test pieces of structural steel for pins and rollers shall be round rods turned to the form and size shown in Fig. 8. The elongation shall be measured in a length of 2 inches, and include the break.

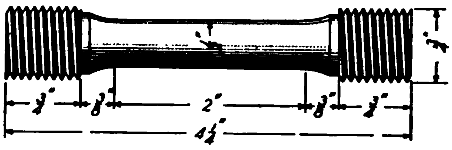

FIG. 8

Test pieces of structural steel for other purposes, and of wrought iron, shall be flat bars of the same thickness as the material from which the test pieces are cut, and shall have the form and size shown in Fig. 9. The elongation shall be measured in a length of 8 inches, including the break.

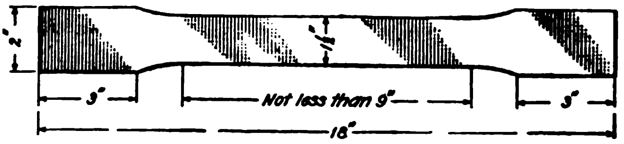

FIG. 9

Test pieces of steel castings shall be cast with one or more castings, and shall be cut from them when they are cold; they shall be turned to the form and size shown in Fig. 8, and the elongation shall be measured in a length of 2 inches, including the break.

168. Annealed Material.—Two test pieces shall be cut from material that is to be annealed, one before and the other after annealing.

169. Varying Sections.—Test pieces shall be cut from the thickest and from the thinnest materials when sections differing by $\frac{3}{8}$ inch or more are rolled from the same melt, and from each section when widely different sections, such as angles and I beams, are rolled from the same melt.

170. Labor and Tools.—The Contractor shall furnish the Engineer, free of charge, proper test pieces, machines, tools, and labor necessary to make the required tests.

171. Rejection of Materials.—If material does not possess the specified properties, the entire melt from which the material was taken shall be rejected, unless by additional tests it can be proved that the defects are confined to only a part of the melt. All material that, subsequently to its acceptance at the mills, shows that it is not of the desired quality shall be rejected and shall be replaced with satisfactory material. Rusty material shall be rejected, unless it is thoroughly freed from rust before being used.

FULL-SIZE TESTS

172. Full-sized members shall be tested to destruction if so directed by the Engineer. The material in such members shall be paid for by the Engineer at the contract price per pound, if it complies with the requirements; otherwise, it will be rejected at the Contractor's expense, and all similar members may be rejected unless it can be shown, by additional tests, that the failure is due to defects that are confined to one or a very few members.

173. Eyebar Tests.—In general, one full-size test shall be made from every twenty-five eyebars. After being properly annealed, eyebars shall have an ultimate tensile strength of not less than 55,000 pounds per square inch, an elastic limit of not less than 50 per cent. of the ultimate strength, and an elongation of not less than 15 per cent. in 10 feet, including the break. If a bar breaks in the head, but develops the required ultimate strength and elongation, it shall not on that account be rejected, unless more than one-third of the number of bars tested break in the head.

WORKMANSHIP

GENERAL REQUIREMENTS

174. **Finished Material.**—Workmanship and finish shall be first-class and in accordance with the best practice. Material shall get a thorough rolling at the proper temperature; large sections shall be rolled from large-sized billets; pins shall be forged in the most approved manner. Finished material shall be entirely free from surface defects, shall have good finish, and be well straightened in the mill before shipment.

175. **Variation in Dimensions.**—Material shall be rolled as near as practicable to the weight and thickness specified; no variation will be allowed greater than 2½ per cent. above nor 1½ per cent. below the computed weights, except in wide-sheared plates, for which slightly greater variation will be allowed, according to the practice in the best rolling mills.

176. **Annealing.**—All parts that have been heated during manufacture shall be carefully annealed and thoroughly cooled before they are prepared for connections.

177. **Welding.**—Welds in steel will not be allowed under any circumstances. Welds in wrought iron will be permitted when specified.

178. **Reentrant Angles.**—No sharp reentrant angles will be allowed in any piece of metal; the corners shall always be drilled out before the sides are cut.

179. Rivet Holes.—Rivet holes in members of longitudinal and lateral bracing, stiffeners, and unimportant details may be punched not more than $\frac{1}{16}$ inch larger than the nominal diameter of the rivets. Rivet holes in flanges, the edges of web-plates where flanges are attached, floor connections, and all riveted members of trusses and their connections shall be punched $\frac{3}{16}$ inch smaller, and reamed to not more than $\frac{1}{16}$ inch larger than the nominal diameter of the rivets, if the material is $\frac{11}{16}$ inch thick or less. Reaming shall preferably be done after the parts are assembled. Rivet holes in material $\frac{3}{4}$ inch thick and over, and in flanges of I beams and channels, shall be drilled from the solid metal.

180. Punching.—Rivet holes shall be spaced and punched so accurately that, when parts are brought together, the corresponding holes will match. Slight inaccuracies may be corrected by reamers. Drifting to make holes large enough for the rivets will not be allowed in the shop or in the field.

181. Reaming and Drilling.—When parts are assembled for reaming, at least one-third of the holes shall be filled with bolts. If necessary to take them apart for shipment, they shall be match-marked, and a diagram of the marks shall be furnished the erector to insure the same position of each part in the finished bridge as in the shop. Parts assembled for drilling shall be taken apart, and any shavings between them removed before riveting. Burrs on reamed and drilled work shall be removed.

182. Field Connections.—All field connections, except for members of bracing, shall be fitted in the shop. The rivet holes shall be reamed to fit while the members are bolted together in their correct positions, or by means of metal templets not less than $1\frac{1}{4}$ inches thick, carefully clamped to the members in the correct position.

183. Rivets.—The sizes of rivets shown on the plans shall be taken to mean the diameters of the cold rivets before driving. When heated and ready for driving, the surfaces of all rivets shall be perfectly clean; when driven, they shall completely fill the holes and be perfectly tight. Loose and badly driven rivets shall be cut out and replaced with tight well-driven rivets. Rivet heads shall be round and of uniform size for the same-sized rivets all through the work; they shall be full and neatly made, concentric with the shank, and in full contact with the surface.

184. Riveting.—Wherever possible, rivets shall be driven by machine or power riveters. Power riveters shall be capable of maintaining the applied pressure after driving. Field riveting shall preferably be done by pneumatic riveting hammers. Before any rivets are driven, at least one-third the holes shall be filled with bolts of the same size as the rivets, and they shall be carefully tightened.

185. Bolts.—Bolts shall not be used in place of rivets, except by special permission. When bolts are used, the holes shall be exactly at right angles to the surfaces of the connected parts, and the bolts shall be turned to a driving fit.

186. Riveted Members.—After being riveted, members shall be straight and correct in dimensions. Great care shall be taken that the bearing surfaces of girders and the faces of flange angles of girders and riveted truss members are perfectly straight.

187. Web-Plates.—Web-plates shall be straight and not project beyond the faces of the flange angles; they shall be not more than ⅛ inch below the faces of the angles at any point. Splices in web-plates shall be not more than ¼ inch apart.

188. Flange Members.—Where flange members of plate girders are spliced, in either the top or the bottom flange, the ends shall be planed exactly square; and, after riveting, the spliced ends shall be in perfect contact throughout the entire section of the spliced member.

189. Stiffeners.—Stiffeners shall fit tight between the horizontal legs of flange angles; the ends of fillers under stiffeners and of web splice plates shall be not more than ⅛ inch from the edges of the vertical legs of the flange angles.

190. Ends of Floor Members.—The ends of floor-beams and stringers shall be planed perfectly smooth and straight; after planing, the members shall have the length shown on the plans. Not more than $\frac{1}{16}$ inch shall be planed off the faces of the connection angles. Ends of solid floor sections shall be perfectly straight and smooth; if necessary, they shall be planed to secure this result.

191. Ends of Riveted Members.—Where riveted members, either tension or compression, are spliced, the ends shall be planed smooth exactly at right angles to the axis of the member; and, after riveting, the spliced ends shall be in perfect contact throughout the entire section of the spliced member. The ends of columns of viaducts shall be planed smooth before the cap and base plates are riveted on, and so that the entire section of the column, as well as the faces of the horizontal angles riveted to the ends, shall have a full and even bearing on the cap and base plates.

192. Ends of Girders.—The ends of all girders shall be neatly finished; web-plates, flange angles, and flange plates shall be finished flush with each other.

193. Eyebars.—Eyebars shall be of uniform thickness throughout, perfectly straight, and free from welds. The heads shall be full, smooth, and sound, and accurately centered with the bars; they shall be formed by upsetting in the most approved manner. After the heads are formed, eyebars shall be carefully annealed and thoroughly cooled before further handling.

194. Pinholes.—Pinholes shall be bored exactly at right angles to the axis of the member, not more than $\frac{1}{50}$ inch larger in diameter than the pins up to 5 inches diameter, nor more than $\frac{1}{32}$ inch larger for diameters greater than 5 inches, and not more than $\frac{1}{64}$ inch greater or less than the calculated distance center to center as shown on the drawings. The centers of pinholes in riveted members shall generally lie on a line passing through the center of gravity of the member, unless shown elsewhere on the drawings.

The centers of pinholes in eyebars shall be on the center line. Bars that are to be placed side by side in a bridge shall be bored so accurately that, if stacked up one above the other, a pin of the required size can be passed simultaneously through all the holes at either end without much forcing.

195. Pins.—Pins shall be forged and carefully turned cylindrical, smooth, and true to size, and long enough to give all members a full bearing. They shall be driven with pilot nuts and caps; at least one driving cap and pilot nut for each size of pin shall be furnished by the Contractor. Threads on ends of pins shall project ¼ inch beyond the surfaces of the nuts when they are screwed on.

196. Pin Nuts.—Pin nuts shall be made so as to enclose the projecting ends of pins and come to a full bearing against the members.

197. Rollers.—Rollers shall be forged and carefully turned cylindrical, smooth, and true to size.

198. Bearings.—Sole plates and bedplates shall be planed smooth and straight. The sliding surfaces at expansion ends shall be planed in the direction of expansion. The bottoms of webs and connection angles of pedestals shall be planed before the base plates are riveted on.

199. Steel Castings.—Steel castings shall be planed where noted on drawings and wherever else it is necessary to insure good workmanship and even bearing. Cored holes shall be not more than ⅛ inch greater or less than the required sizes, nor more than ⅛ inch from the position shown on the drawings. Steel castings shall be true to the required dimensions after annealing.

200. Shipment.—All pins, rivets, and other small parts shall be boxed, and the screw threads wrapped with twine, before shipment. An excess of field rivets equal to 20 per cent. of the required number for each size and length shall be shipped for each bridge. All members shall be handled and loaded on cars in such a way as to avoid injury; any piece showing the effects of rough handling may be rejected. The weight and erection mark shall be plainly marked on each part, and the weight and contents on each box.

PAINTING

201. General.—As soon as material is finished and accepted, it shall be thoroughly cleared of rust, dirt, scale, and other surface deposits, and carefully painted in accordance with the following specifications. No painting shall be done until material is accepted.

202. Surfaces in Contact.—Surfaces that will be in contact with others shall be given one coat of red lead and linseed oil before assembling.

203. Inaccessible Parts.—All parts not accessible for painting after erection shall be given one heavy coat of approved paint at the shop as soon as finished and accepted, and one coat at the bridge site before erection.

204. Machined Surfaces.—All machined surfaces, such as screw threads, pins, and bearing surfaces shall be coated with a mixture of white lead and tallow as soon as finished and before leaving the shop.

205. Finished Members.—Finished members shall be given one heavy coat of approved paint before leaving the shop. In general, paint shall be allowed to dry 48 hours before loading material for shipment.

206. Painting After Erection.—After erection, the bridge shall be given two heavy coats of approved paint. At least 48 hours must elapse between the applications of the two successive coats to any part of the bridge.

207. Weather Conditions.—Painting shall be done only when the surface of the metal is perfectly dry. Field painting shall not be done in wet or freezing weather. Shop painting may be done in such weather, if, after painting, the material is allowed to remain at least 48 hours in a covered building whose inside temperature is not below freezing.

208. Quality.—The quality of paint and class of labor for painting shall be the best obtainable; special attention shall be given to this part of the work.

ERECTION

209. Commencement of Work.—The Contractor shall notify the Engineer when he is ready to commence work, and the erection shall not begin until authority has been received in writing from the Engineer.

210. Care of Material.—Before and during erection, all material shall be kept clean and so stored and handled as to avoid injury.

211. Old Structures.—If the new bridge is to take the place of an old bridge on the same site, the Contractor shall take down the old bridge; if required, he shall take it down without loss or injury to any part, and shall mark all parts for reerection. A diagram showing these marks shall be furnished to the Engineer.

212. Method of Erection.—If it is necessary to place any restrictions on the method of erection, the Engineer shall state them in the letter of invitation to bid; he shall also state the desired disposal of the old bridge.

213. Lines and Grades.—The Contractor will be expected to preserve with care all stakes set by the Engineer.

214. Field Riveting in Splices.—Field rivets in splices of compression members shall not be driven until the members are subjected to dead-load stress. The splices shall be well bolted prior to this, to hold the members firmly in line.

215. Bridge Seats.—Bridge seats shall be dressed by the Contractor. If they are out of level, he shall place the bedplates or pedestals level and at the correct elevation; if necessary, he shall fill in under them with cement grout well rammed into all open spaces under the bedplates or pedestals. The grout shall be allowed to set at least 24 hours before any load is placed on the bedplate or pedestal.

216. Laws and Ordinances.—The Contractor shall comply with all laws and ordinances applying to and governing the work of erection, and shall obtain all necessary permits and comply with their requirements. He shall take precautions to guard against accidents and injury to persons and property, and shall be responsible for all losses due to floods, storms, and other casualties. He shall so conduct his work as not to interfere with the work of other Contractors, nor with the traffic on railroads, highways, or waterways, unless he procures written permission to do otherwise.

217. Extra Work.—If the Engineer erects the bridge and extra work is found to be necessary owing to defective shop work or careless handling, the Contractor shall bear the cost of it; this cost shall be deducted from the amount due him.

218. Employment of Men.—The Contractor shall follow all reasonable directions of the Engineer in regard to the discipline of his men during the work of erection. At the completion of the work he shall, if desired by the Engineer, furnish proper bond to protect the Engineer from all liabilities resulting from the failure of the Contractor to pay for the materials or labor.

219. Final Test.—As soon as the bridge is completed and before its final acceptance, the Engineer may test it by loading it with the specified loads. Any defect that becomes apparent shall be corrected by the Contractor.

220. Name Plate.—A name plate of neat design and finish, giving the name of the Contractor and the date of erection, shall be firmly attached to each bridge in a prominent position.

INSPECTION

221. Inspectors.—All material shall be tested by, and all workmanship shall be under the supervision of, inspectors appointed by the Engineer. The Engineer and his inspectors shall have free access at all times to all parts of the mills and shops in which any part of the bridge is being manufactured.

a copy is sent to the bidder with the letter of invitation, and he is requested to submit a proposed plan with his bid. In the majority of cases, however, the bidder is furnished with a plan that gives all the information called for on the blank form, and in addition the stresses in the members.

227. When it is necessary to have a bridge ready for traffic at a certain date, this date is stated in the letter of invitation. In some cases, bidders are offered a bonus for each day the bridge is finished before, and required to pay a penalty for each day the bridge is finished after, the stated time.

228. Location.—The first step in the design of a bridge is the selection of a site, and the arrangement of spans and location of piers and abutments. No fixed rule can be given for the location of the bridge and abutments or piers; these are matters that depend almost wholly on local conditions, and must be decided by the judgment of the engineer. In general, however, it may be said that the site should be chosen with regard to economy and so that the bridge will be in a good position to accommodate the traffic. As a rule, single spans are most economical for short bridges, and steel trestles for long bridges. If the bridge is over a river, and a steel trestle cannot be used, owing to the danger from floods or because the piers would block navigation, the bridge may be composed of two or more spans resting on masonry piers; or, if at a great elevation above the river, so that high piers are expensive and objectionable, a single long span may be used. It is frequently necessary to estimate the approximate cost of several plans and designs before one is finally selected.

229. Kind of Bridge.—When the location of the piers has been decided, the number and length of spans may be determined; then the general style of bridge, whether deck or through, the kind of trusses or girders, together with panel lengths and depths, and the width of bridge, are selected. In general, deck bridges should be used when possible, as they are somewhat more economical and, on

GENERAL DATA

For bridge over______________________________

at____

Length and general dimensions____________________

____

Skew or angle of abutments with center line of bridge______

____

Width of bridge and location of trusses______________

____

Floor system_________________________________

Number and location of tracks____________________

Loading____________________________________

Description of abutments________________________

Distance from floor to clearance line________________

" " " " high water____________________

" " " " low water_____________________

" " " " river bottom___________________

Character of river bottom_______________________

Usual season for floods________________________

Name of nearest railroad station___________________

Distance to nearest railroad station_________________

Time limit__________________________________

Name of Engineer_____________________________

Address of Engineer__________________________

Remarks___________________________________

____

____

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229. **Kind of Bridge.**—When the location of the piers has been decided, the number and length of spans may be determined; then the general style of bridge, whether deck or through, the kind of trusses or girders, together with panel lengths and depths, and the width of bridge, are selected. In general, deck bridges should be used when possible, as they are somewhat more economical and, on account of the transverse bracing between the trusses, somewhat stiffer than through bridges; their use is limited, however, to locations where there is plenty of room below the floor. The kind of structure—plate girders, riveted or pin-connected trusses, etc.—depends on the span; the style of truss—Pratt, Warren, Baltimore, etc.—is left to the judgment of the engineer. The depth and panel length are, as a rule, controlled by the specifications; but a few words may be added. It has been found in practice that for through plate-girder bridges, panel lengths of 10 to 15 feet are best; for riveted truss bridges, from 15 to 20 feet; and for pin-connected truss bridges, from 20 to 30 feet. Panels may be made longer than 30 feet in spans greater than 250 feet in length, but for shorter spans it is better not to exceed this limit.

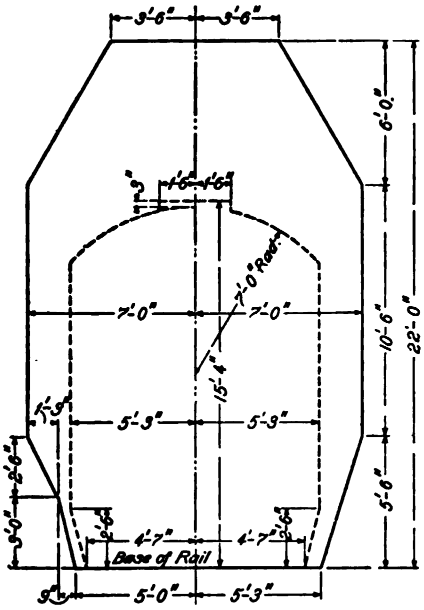


FIG. 10

230. Width and Clear Height.—The width and clear height must be sufficient to accommodate the traffic. For railroad bridges, they depend on the outside dimensions of the car or engine having the largest cross-section; or, since one car may have the greatest width and another the greatest height, the outside dimensions of a car that would have that width and height. This is usually referred to as the **maximum equipment.** The dotted lines in Fig. 10 represent the approximate outline of the maximum equipment in use on steam railroads in the United States, the full lines forming the diagrams given in Figs. 1 and 2. The latter are commonly spoken of as **clearance diagrams.** Their lines are located somewhat outside the lines of the maximum equipment to give required clearance, so as to keep unlooked-for projections, such as heads and arms out of car windows, from striking any part of the bridge. The additional height at the top is to prevent any part of the overhead bracing from striking the heads of brakemen on top of the cars. Owing to difficulties of design, the top flanges of through plate girders are allowed to come closer to the lower part of the outline of maximum equipment than any part of truss bridges. The same clearance diagram should be used for bridges on railroads that are undergoing electrification, that is, on which the motive power is being changed from steam to electricity, as well as on roads that are built especially for heavy cars operated by electricity on private right of way for freight and passenger service. The indications at present are that the equipment on such roads will be as large as the largest now in use on steam railroads.

As the equipment on street railroads is somewhat smaller than on steam or heavy electric roads, less width and height need be provided. Fig. 4 represents the outline of one-half the clearance diagram used on street railways. Less clearance is needed above the cars than on steam roads, as it is not necessary to allow for men standing on top of the cars, but simply to provide ample room for the trolley.

On both steam and electric roads, more clearance is allowed at the sides on curves than on straight track, as parts of the cars overhang the track and the tops lean over.

231. The width of a highway bridge depends altogether on the amount and kind of traffic to be provided for. For country bridges, the clear width should never be less than about 16 feet; for suburban and city bridges, the width may be anything from about 20 feet to the full width of the street leading to the bridge. Each case must be decided by the local requirements.

232. Floor System.—The arrangement of stringers and floorbeams depends, to a great extent, on the allowable depth of floor from the surface to the underneath clearance line. As a rule, it is well to have as much depth as possible. Two-truss bridges are preferred, as a center truss requires the spreading of the tracks or roadways. When two-truss bridges are used, the depth of the floorbeams and that of the floor are much greater than for three-truss bridges. In the elimination of grade crossings, and frequently in other cases, it is exceedingly expensive to separate the grades enough to permit the use of deep floorbeams; and it is then advisable at times to spread the track on railway bridges, or separate the roadways on highway bridges, so as to allow for the insertion of a center truss. In such cases, the floorbeams need not be so deep, as they will be but one-half as long as if two trusses were used.

Some engineers think it is best to place the stringers of railroad bridges directly under the rails, but it seems better practice to space them 6 feet 6 inches center to center, each one about 9 inches from the center of the rail. In this way, some of the shock and vibration of the train is absorbed by the elasticity of the ties, which have a chance to deflect.

Two stringers are frequently placed close side by side to carry the load on each rail, but this is not good practice, as it is almost impossible to distribute the load equally between them on account of poor fitting of ties, etc. If this inequality occurs at the center of a panel, the stringer getting the greater part of the load will deflect, thereby transmitting some of the load to the other stringer, and not much harm will result. If the inequality occurs at the end of a panel, the rivets connecting one of the stringers to the floorbeam web are liable to get the load that should go through two sets of rivets; they will be overstrained, and may, in time, loosen. When stringers are riveted to floorbeam webs, it is better to use one stringer for each rail.

233. Short-span I-beam bridges for railroads are composed of two or three beams for each rail. The beams are placed close together, and are made to act as one beam by being firmly bolted together and held in place by separators.

There is then no objection to using more than one beam under a rail. If the stringers in floor systems were laid close and bolted together in the same way, it would be impossible to get satisfactory connections to the floorbeams.

234. Deck truss railroad bridges are occasionally built with a floor system, the ties resting on the top chords of the

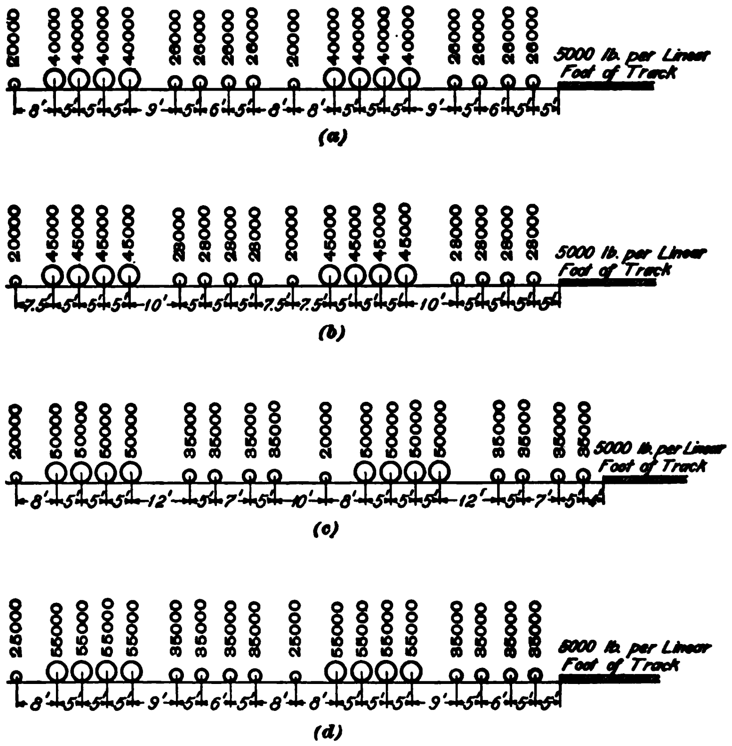


Fig. 11

trusses. The sections of top chord then act as beams as well as compression members, and are subjected to simultaneous compressive and bending stresses. This practice is condemned by the best engineers. It is best in all cases to provide a floor system in the top chord, the ends of the floorbeams being connected to the insides of the trusses or resting on top of the trusses at the panel points.

235. Live Loads.—One of the most important steps in the design of a bridge is to ascertain the live or moving load the bridge is to carry. The live load on a railroad bridge consists of locomotives and cars. As explained in *Stresses in Bridge Trusses*, Part 4, it is customary to use typical loadings that represent the heaviest loads it is expected the bridge will ever have to carry. Fig. 11 shows four typical loadings in use on leading railroads in the United States; Cooper's loadings also have been adopted by many of the leading railroad companies. At the present time, Cooper's E50 well represents the heaviest loads on most American railroads. In a few special cases, where the loads in use are somewhat heavier than this, each load of Cooper's E50 may be multiplied by 1.1 or 1.2, as desired, giving what may be called E55 and E60, respectively, approximately equivalent to the actual loads, and the resultant systems substituted for E50 in the specifications. For bridges on branch lines and on lines on which the locomotives and cars are light, E40 will give sufficiently heavy loads. In case extra heavy locomotives and cars are used for any purpose, the bridges on lines over which they operate should be designed for the actual loads, increased 10 per cent. in some cases to allow for future increase.

236. Up to the present time, the only type of concentrated load that has been considered for railroad bridges has been the steam locomotive and train of cars. There are now in use electric locomotives very nearly equal in weight to the steam locomotives. Fig. 12 shows the distances between axles and the weights on the axles of one of the heaviest electric locomotives that has been built. Bridges over which such heavy locomotives are to pass, or over which it is likely they will pass in the future, should be designed for Cooper's E50, in the same way as other railroad bridges, or for the actual loads, as explained above.

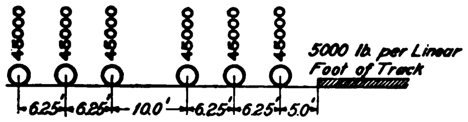


Fig. 12

237. On electric roads designed for the multiple-unit system, which uses no locomotives, each car carrying its own motors, it is necessary to ascertain if there is any possibility of the road being used in the future by steam or electric locomotives; if so, due allowance should be made so that the bridges will be strong enough for them. If it is likely that nothing but the cars will ever run on the road, the bridges should be designed for the actual weights of the cars when fully loaded, increased 10 or 20 per cent. to provide for a possible increase in the weight of future cars. Fig. 13 rep-

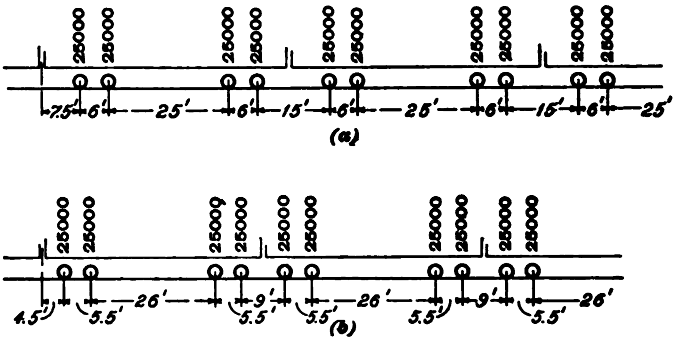


FIG. 13

resents the weights and axle spacing of the cars run by the multiple-unit system on two different roads. The specifications given for railroad bridges should be used for such system of loading, except that the new loading should be substituted for that given in Art. **24.**

238. The loads to which highway bridges are subjected differ considerably from those to which railroad bridges are subjected. They usually consist of heavy snowfalls, crowds of people, wagons, road rollers, and street cars. It has been observed that city bridges are more likely to be subjected to the weight of large crowds of people and of wagons than country bridges; a crowd of people may cover the entire floor of a short span, but is not likely to cover the whole floor of a long span. For this reason, the uniform load per square foot that is used is heavier for city than for country bridges, and heavier for short than for long spans.

Heavier road rollers are used on city streets than on country roads. The case of street cars, however, is somewhat different. The heaviest cars run in city streets are, as a rule, those that are used for interurban traffic; that is, to connect cities. These cars cross country bridges as well as city bridges, and both kinds of bridges must be designed to support them. The loads given in Art. **98** represent, as near as possible, those to which the different bridges may be subjected, and are approximately the same as those now in use by the best engineers. It is not probable that they will increase for a great many years.

239. Impact and Vibration.—As explained in *Strength of Materials*, Part 1, a load that is suddenly applied produces twice as great a stress as one that is gradually applied, and a load that is applied in a very short interval of time causes a stress greater than that caused by a load that is gradually applied, but less than that caused by a suddenly applied load. The moving loads that cross bridges are applied in a short interval of time, more so in the case of railroad than of highway bridges, and it is customary to make some allowance for the resulting increased stresses in the members and for the shock and vibration of the bridge under the passing loads. The increase in the stresses is called the **effect of impact and vibration.** Some experiments have been made and several formulas proposed for determining the magnitude of this increase. Owing, however, to the difficulty and expense of making experiments, it has been impossible to obtain reliable results. Practice varies considerably in respect to the formula used; some engineers use two values for the allowable intensity of stress – one for dead-load stress, and the other, usually one-half the former, for live-load stress. This method has the disadvantage that it causes difficulty in designing, especially in compression members; it is necessary to find separately the areas of cross-section required to resist the dead-load and the live-load stresses and to add them in each case to obtain the required dimensions. This method is, besides, illogical, as it tacitly assumes that the proportional effect of impact and vibration is the same in all members, while as a matter of fact it is greater in such members as floor members and hip verticals, which receive their maximum live-load stresses in a short interval of time (in some cases, $\frac{1}{4}$ second), than in such members as chords of long spans, which do not receive their maximum live-load stresses in so short a time. The best engineers allow the same intensity of stress for both dead-load and live-load stresses, but add a certain amount to the live-load stress as calculated from the loading. In some cases, the amount added is a percentage of the live load equal to the ratio of the live- to the sum of the live- and dead-load stresses; but in the great majority of cases allowance is made as specified in Arts. **25** and **99.** The time required for any live-load stress to rise from zero to its maximum depends on the time it takes the moving load to cover the part of the bridge that must be loaded in order to cause the maximum stress. The formulas just referred to take this into account; they are the results of experience, and are the most satisfactory so far devised. More allowance is made in railroad bridges than in highway bridges that carry electric railways, as in the former class of bridges the loads move much faster and also cause more shock and vibration.

EXAMPLE 1.—The live-load stress in the center panel of the upper chord of a railroad bridge truss 150 feet long is 280,000 pounds. What amount must be added for impact and vibration?

SOLUTION.—In this case, $S = 280{,}000$; as the member under consideration is a chord member, the entire span must be loaded to produce the stress S. Then (see Art. **25**), $L = 150$, and, therefore,

$$I = \frac{300}{150 + 300} \times 280{,}000 = 186{,}700 \text{ lb. Ans.}$$

EXAMPLE 2.—In a highway bridge truss 125 feet long, the live-load stress in the end post, due to the load on the car track, is 75,000 pounds. What amount must be added for impact and vibration?

SOLUTION.—In this case, $S = 75{,}000$; as the member under consideration is an end post, the entire length of span must be loaded to produce the stress S. Then (see Art. **99**), $L = 125$, and, therefore,

$$I = \frac{300 - 125}{1{,}000} \times 75{,}000 = 13{,}100 \text{ lb. Ans.}$$

240. Reversal of Stress.—It is a well-known fact, established by experiment, that a piece of metal that is subjected to a large number of repetitions of varying stresses will finally break, even though the greatest stress to which it has been subjected is much less than the ultimate strength of the metal. This is true whether the stresses are of the same kind or are of opposite kinds. Bridge members are subjected to a great number of repetitions of varying stresses, and various methods are in use to make allowance for the effect. Some of these methods make use of different allowable intensities of stresses in different members, the actual value in any case depending on the ratio of the minimum to the maximum stress. It is the best practice at the present time, however, to ignore the effect in those members in which the maximum and minimum stresses are of the same kind, as the range of stress is comparatively small, and to allow for it in those members in which the maximum and minimum stresses are of opposite kinds, as the range of stress is then comparatively large.

The customary way to make allowance in those members in which the stress reverses is to add to each stress eight-tenths of the other and then design the member for both of the increased stresses. For example, if the maximum stress in a member is 25,000 pounds compression, and the minimum stress is 10,000 pounds tension, the member must be designed for

$$25{,}000 + \tfrac{8}{10} \times 10{,}000 = 33{,}000 \text{ pounds compression}$$

and for

$$10{,}000 + \tfrac{8}{10} \times 25{,}000 = 30{,}000 \text{ pounds tension}$$

Example.—The maximum stress in a member is 50,000 pounds tension, and the minimum stress 20,000 pounds compression. (*a*) If the allowable intensity of tensile stress is 16,000 pounds per square inch, what is the required net section of the member? (*b*) If the value of $\frac{l}{r}$ is such that the allowable intensity of compressive stress is 13,500 pounds per square inch, what is the required gross section?

Solution.—(*a*) The tension for which the member is to be designed is $50{,}000 + \tfrac{8}{10} \times 20{,}000 = 66{,}000$ lb.

Then, the required area of net section is

$$66{,}000 \div 16\,000 = 4.125 \text{ sq. in.} \quad \text{Ans.}$$

(*b*) The compression for which the member is to be designed is

$$20{,}000 + \tfrac{8}{10} \times 50{,}000 = 60{,}000 \text{ lb.}$$

Then, the required gross area is

$$60{,}000 \div 13{,}500 = 4.444 \text{ sq. in.} \quad \text{Ans.}$$

241. Dead Load.—In finding the dead load, it is customary first to decide on the type of floor and calculate its weight. Then the approximate weight of the steelwork can be calculated by some formula. The weight of a bridge per linear foot depends on the moving load for which the bridge is designed and on the intensities of stress adopted. Formulas for calculating weights of bridges are purely empirical; they are based on values that have been observed in actual structures. On account of the fact that railroad bridges are, as a rule, designed for one loading, the formulas for their weights are fairly reliable. In the case of highway bridges, in which the loadings and widths vary within a wide range, it is almost impossible to give formulas that represent all the conditions. In any event, it is necessary, after having completed the design of a bridge, to compute its weight accurately; if the computed weight differs much from the assumed weight, the dead-load stresses should be recomputed and the cross-sections of the members altered to correspond with the corrected stresses. This is sometimes spoken of as **revising the design.**

242. Weights of Railroad Bridges.—The following formulas give the weights w, in pounds per linear foot, of the steel work in railroad bridges designed according to the specifications from Arts. **14** to **90** for Cooper's E50, l being the span, in feet:

Rolled I beams, $w = 25\,l$ for each track.

Deck plate-girder bridges, $w = 500 + 8\,l$ for one track.

Half-through plate-girder bridges, $w = 800 + 12\,l$ for one track.

Riveted truss bridges,

$$w = 1{,}500\left[1 + \left(\frac{l - 90}{100}\right)^2\right] \text{ for one track}$$

Riveted truss bridges,

$$w = 2{,}600\left[1 + \left(\frac{l - 90}{100}\right)^2\right] \text{ for two tracks}$$

Pin-connected truss bridges may be assumed 5 per cent. lighter than riveted truss bridges for spans over 200 feet in length. For spans shorter than 200 feet, the same formulas may be used as for riveted truss bridges.

The above weights are for bridges having standard-tie floors, as described in Art. **48.** Solid-floor bridges will, in general, be about 25 per cent. heavier.

243. Weights of Highway Bridges.—The following formulas give the weights w, in pounds per linear foot, of one girder or truss in highway bridges designed according to the specifications given in Arts. **91** to **159**; l being the span, in feet, and W the maximum load per linear foot supported by the girder or truss, including the live load together with the impact, and the weight of floorbeams, stringers, railings, and floor.

Plate-girder bridges, $w = \frac{W}{1{,}000}(24 + .8\,l)$.

Riveted truss bridges, $w = \frac{W}{12}\left[1 + 2\left(\frac{l - 90}{100}\right)^2\right]$.

Pin-connected truss bridges may be assumed 5 per cent. lighter than riveted truss bridges for spans over 200 feet in length. For spans shorter than 200 feet, the same formulas may be used as for riveted truss bridges.

DESIGN OF PLATE GIRDERS

(PART 1*)

GENERAL PRINCIPLES

BEAMS

1. Introduction.—The principles governing the distribution of stress in the cross-section of a beam, and the formulas by which the stresses are obtained from the moments and shears, have been explained in connection with the theory of beams in *Strength of Materials*. The formula for the maximum tensile and compressive stresses at any section of a beam is

$$s = \frac{Mc}{I}$$

in which s = maximum intensity of stress;

c = distance of most remote part of section from neutral axis;

I = moment of inertia of entire cross-section about neutral axis;

M = bending moment at section considered.

If the expression $\frac{I}{c}$, which is the section modulus, is represented by Q, the formula for the maximum intensity of stress becomes

$$s = \frac{M}{Q}$$

*All the tables referred to in this Section are given in *Bridge Tables* and explained in *Bridge Members and Details*, Parts 1 and 2. In referring to the Section entitled *Bridge Specifications*, the title will for convenience be abbreviated to *B. S.*

From this formula follows

$$Q = \frac{M}{s}$$

Care must be taken that the proper units are used in these formulas. It is customary to express s in pounds per square inch, c in inches, and M in inch-pounds; the moment of inertia I and section modulus Q found from the dimensions of the section expressed in inches.

2. I Beams.—The maximum intensity of stress in an I beam is found by the formula $s = \frac{M}{Q}$. The section moduli of I beams of different weights and depths are given in Table XIV. The practical problem, however, usually consists in finding what size of I beam will safely carry a given load.

To solve this problem, the maximum bending moment M is computed, and then, by means of the formula $Q = \frac{M}{s}$, the required value of the section modulus is found. An I beam having a section modulus equal to or slightly greater than that required is then chosen from Table XIV. The same method is followed in designing channels to be used as beams.

EXAMPLE 1.—The bending moment at a given section of a 12-inch 40-pound I beam is 672,000 inch-pounds. What is the maximum intensity of stress at that section?

SOLUTION.—Consulting Table XIV, the section modulus of a 12-in. 40-lb. I beam is found to be 44.8. Then, since M is 672,000 in.-lb., the maximum intensity of stress s is

$$672{,}000 \div 44.8 = 15{,}000 \text{ lb. per sq. in. Ans.}$$

EXAMPLE 2.—The maximum bending moment on a beam is 1,840,000 inch-pounds. If it is required that the maximum intensity of stress shall not exceed 16,000 pounds per square inch, what size and weight of I beam must be used?

SOLUTION.—Since M is 1,840,000 in.-lb., and s is 16,000 lb. per sq. in., the required value of section modulus Q is 1,840,000 ÷ 16,000 = 115. Consulting Table XIV, and following from the smaller beams toward the larger, the first I beam that has a section modulus as large as required is found to be a 15-in. 95-lb. beam, which has a section modulus of 116.4. Looking lower down, however, it is found that a 20-in. 65-lb. beam has a section modulus of 117, which is also sufficient. The latter beam should be used, as it is 30 lb. per ft. lighter, and therefore more economical than the 95-lb. beam.

NOTE.—The advantage of using a deep beam is here apparent, as the required value of section modulus can be had with a lighter beam than if a shallower beam were used.

EXAMPLES FOR PRACTICE

1. The bending moment at a given section of a 24-inch 100-pound I beam is 3,472,000 inch-pounds. What is the maximum intensity of stress at that section? Ans. 17,500 lb. per sq. in.

2. The bending moment at a given section of an 18-inch 65-pound I beam is 1,370,600 inch-pounds. What is the maximum intensity of stress at that section? Ans. 14,000 lb. per sq. in.

3. The maximum bending moment on an I beam is 133,300 foot-pounds. What size I beam must be used in order that the maximum intensity of stress shall not exceed 16,000 pounds per square inch? Ans. A 20-in. 65-lb. I beam

4. The maximum bending moment on an I beam is 400,000 inch-pounds. What size I beam must be used in order that the maximum intensity of stress shall not exceed 16,000 pounds per square inch? Ans. A 10-in. 30-lb. I beam

PLATE GIRDERS

SECTION MODULUS

3. The maximum stresses due to bending moment in a plate girder may be found by means of the formula $s = \frac{M}{Q}$. As the flanges are not the same size throughout the whole length of the girder, however, the value of Q changes wherever the section of flange changes, and in order to use the formula it is necessary to compute the value of Q at every section where the flange changes. This requires much time and is rarely done in practice; a modification of the foregoing formula is used that gives sufficiently close results and is much more convenient. This formula will now be explained.

4. Let Fig. 1 be a vertical section of a plate girder having dimensions as follows: thickness of web, t; width (depth) of web, h; vertical distance between centers of gravity of flanges, h_g; total height or depth of section, h_1; gross area of cross-section of top flange and net area of cross-section of bottom flange, which will be assumed equal, A. It will be assumed that the cross-section of the girder is symmetrical; then, the neutral axis will be at the center, at a distance $\frac{h_g}{2}$ from the center of gravity of each flange. As $Q = \frac{I}{c}$, it is necessary to compute the value of I; c is equal to $\frac{h_1}{2}$.

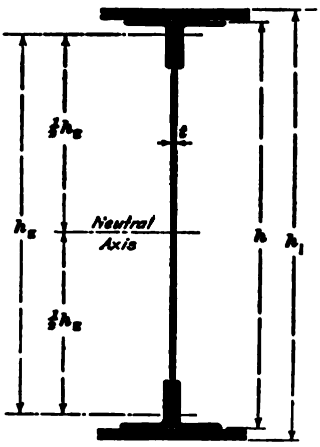


Fig. 1

The moment of inertia of the entire cross-section about the neutral axis is the sum of the moment of inertia of the web about the neutral axis, the moment of inertia of each flange about an axis parallel to the neutral axis and passing through the center of gravity of the flange, and the product of the area of each flange and the square of the distance from its center of gravity to the neutral axis.

The moment of inertia of the web about the neutral axis is $\frac{t h^3}{12}$. As part of the web is cut out by rivet holes, it is customary to allow for the decrease in strength by using for the moment of inertia three-fourths of the theoretical value; that is, in this case, $\frac{3}{4} \times \frac{t h^3}{12} = \frac{t h^3}{16}$. The moment of inertia of each flange about an axis through its center of gravity is very small compared with the other terms, and in practice it is customary to neglect it. The product of the area of each flange and the square of the distance of the center of gravity of the flange from the neutral axis is $A \times \frac{h_g^2}{4}$. For both flanges, twice this value, or $\frac{A h_g^2}{2}$, should be taken. Therefore, for the moment of inertia of the entire cross-section of the girder about the neutral axis, we have, approximately,

$$I = \frac{A h_g^2}{2} + \frac{t h^3}{16}$$

whence, since $Q = \frac{I}{c}$, and $c = \frac{h_1}{2}$,

$$Q = \frac{\frac{A h_g^2}{2} + \frac{t h^3}{16}}{\frac{h_1}{2}} = \frac{A h_g^2}{h_1} + \frac{t h^3}{8 h_1}$$

This formula is not convenient in this form. By assuming that in the first term h_1 can be replaced by h_g, and that in the second term h_1 can be replaced by h, and h^3 by $h^2 \times h_g$, the following more convenient, and sufficiently approximate, formula is found:

$$Q = \frac{A h_g^2}{h_g} + \frac{t h^2 h_g}{8 h} = A h_g + \frac{t h h_g}{8} = h_g\left(A + \frac{t h}{8}\right)$$

As a matter of fact, h, h_1, and h_g are as a rule very nearly equal, and there is not very much error in assuming that any of these quantities can be replaced by either of the other two.

EXAMPLE.—The web of a plate girder is 48 in. × ½ in. in cross-section. Each flange has an area of section equal to 20 square inches, and the vertical distance between their centers of gravity is 47 inches. What is the section modulus of the cross-section?

SOLUTION.—In this case, $h_g = 47$ in., $A = 20$ sq. in., $t = \frac{1}{2}$ in., and $h = 48$ in. Substituting these values in the formula,

$$Q = 47 \times \left(20 + \frac{48 \times .5}{8}\right) = 1{,}081. \text{ Ans.}$$

DESIGN OF FLANGES

5. Determination of Flange Area.—Since $s = \frac{M}{Q}$, we have also, $M = s Q$. Substituting for Q its value given in Art. **4**, this equation becomes

$$M = s h_g\left(A + \frac{t h}{8}\right);$$

whence
$$\frac{M}{s h_g} = A + \frac{th}{8}$$

and
$$A = \frac{M}{s h_g} - \frac{th}{8}$$

The expression $\frac{M}{s h_g}$ is sometimes spoken of as the flange area, and the term $\frac{th}{8}$ is spoken of as the portion of web that goes to make up the flange area, or that assists in resisting the bending moment.

In designing, the distance between the centers of gravity of the flanges is usually first assumed equal to the width of web, and the flange is designed on that basis; then, the correct distance is calculated, and, if necessary, the areas of the flanges are changed to correspond with it.

6. Effect of Web.—It is sometimes specified that the flanges shall be considered as resisting the entire bending moment, without considering the assistance of the web. In this case, as the last expression in the formula for A in Art. 5 represents the effect of the web, it is simply necessary to omit it. The resulting formula is

$$A = \frac{M}{s h_g}$$

By using this formula, the area of each flange is made larger than necessary by an amount equal to one-eighth the cross-section of the web. This assumption evidently gives incorrect results; but, as it provides more flange area than is required, it is on the safe side. The only objection to it is that it is not economical. In the following articles, it will be assumed that the web assists in resisting bending moment.

EXAMPLE.—A plate girder having a $60'' \times \frac{3}{8}''$ web is subjected to a maximum bending moment of 2,000,000 foot-pounds. If the allowable intensity of bending stress is 16,000 pounds per square inch, what is the trial value that would be used for the area of each flange: (*a*) if the web is assumed to assist in resisting the bending moment? (*b*) if the flanges are assumed to resist the entire bending moment?

SOLUTION.—(*a*) As the bending moment is given in foot-pounds, it must be multiplied by 12, in order to reduce it to inch-pounds. This gives $M = 12 \times 2{,}000{,}000 = 24{,}000{,}000$ in.-lb. The value of s

is 16,000 lb. per sq. in. As the trial values of the areas are required, the trial value of h_g, that is, the width of the web, or 60 in., will be used. In the first case, the required area of the flange is given by the formula in Art. 5,

$$A = \frac{M}{s\,h_g} - \frac{t\,h}{8}$$

Here, $t = \frac{3}{8}$ in., and $h = h_g = 60$ in. Then, substituting in the formula,

$$A = \frac{24{,}000{,}000}{16{,}000 \times 60} - \frac{.375 \times 60}{8} = 25 - 2.81 = 22.19 \text{ sq. in.} \quad \text{Ans.}$$

(*b*) In the second case, the required area of flange is given by the formula $A = \frac{M}{s\,h_g}$. Substituting the given values,

$$A = \frac{24{,}000{,}000}{16{,}000 \times 60} = 25 \text{ sq. in.} \quad \text{Ans.}$$

7. Length of Flange Members.—As the bending moment near the end of a girder is less than at the center, the required area of flange section is less near the end than at the center. For this reason, each flange is composed of several parts, usually two angles and one or more plates. The angles, and in some cases one plate, are continued the entire length of the girder; the other plates are shorter, and are cut off where they are no longer required. It is very difficult to find, by the analytic method, the sections at which the different flange plates are no longer required; a combination of the analytic and the graphic method is more convenient. The required areas of flange at several sections along the girder are computed by means of the formula $A = \frac{M}{s\,h_g}$, and a **curve of flange areas** is drawn. Fig. 2 shows the curves for the top and bottom flanges of a plate girder, and the graphic method of determining the sections at which the plates are no longer required. As plate girders are usually symmetrical about the center, only one-half of the span is shown. The diagram is explained in the following article.

8. Curve of Flange Areas.—On any line $X'X$, Fig. 2, the distance $R\,O$ is laid off to scale equal to one-half the span, and the sections A, B, C, etc., at which the flange areas have been computed, are marked in their proper positions. At A, B, C, etc., lines are drawn at right angles to $X'X$, and on them are laid off to scale the distances AA', BB', CC', etc., above and below $X'X$ to represent the values of A as found by the formula $A = \frac{M}{s h_g}$. If the load on the girder is concentrated at the points A, B, C, etc., as in a half-through plate-girder bridge, straight lines RA', $A'B'$, $B'C'$, etc. are drawn connecting the points just found; if the load on the girder is

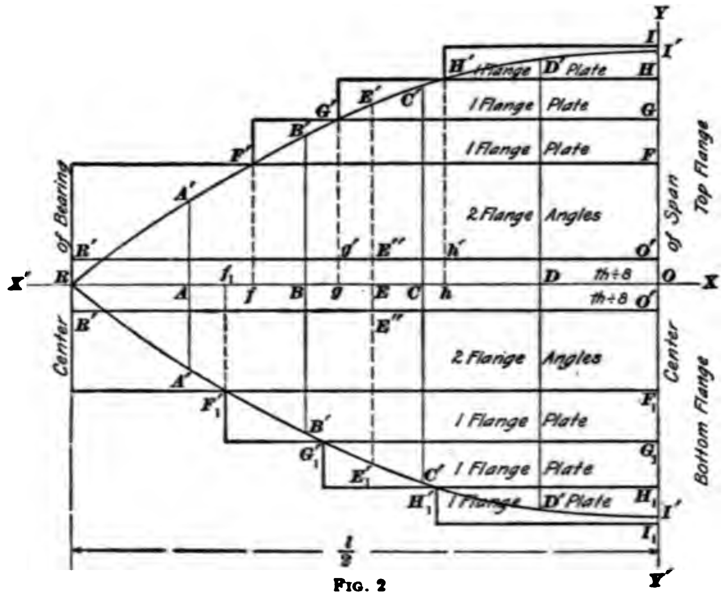


FIG. 2

distributed, as in a deck plate-girder bridge, smooth curves are passed through the points R, A', B', C', D', and I', above and below the line $X'X$, respectively. In the present case, it has been assumed that the load is uniformly distributed; then, the curves RI' are the curves of flange areas, and the ordinate at any section, such as EE' at E, represents the required flange area at that section. The next step in the construction of the diagram consists in laying off to scale on the line YOY' at right angles to $X'X$ at O the areas of the different sections that make up the flanges. When it is specified that the effect of the web in assisting to resist the bending moment is to be neglected, the areas of the angles and plates are laid off directly from O; when it is specified that the effect of the web is to be considered, as in the present case, the points O' are located on $Y O Y'$ at such a distance from O that $O O'$ represents to scale $\frac{t h}{8}$, and the lines $O' R'$ are drawn parallel to $O R$. Then, the ordinates between the lines $O' R'$ and the curves, at any section, such as $E'' E'$ at E, represent the required area that must be provided in the angles and plates at that section. In the present case, it will be assumed that two angles and three plates are required at the center in each flange; $O' F$ represents the gross area of the two angles, and $F G$, $G H$, and $H I$, the gross areas of the three plates in the top flange; $O' F_1$, $F_1 G_1$, $G_1 H_1$, and $H_1 I_1$ represent the net areas of the angles and plates in the bottom flange. Lines are drawn through the points F, G, H, etc. to their intersections F', G', H', etc., respectively, with the curve; then, f, g, etc., on perpendiculars from F', G', etc. to $X' X$, are the sections at which the different flange plates are no longer required.

For example, it is seen that at the section h the required area of the top flange is represented by $h' H'$, which is equal to $O' H$, and this represents the sum of the areas of the two angles and two plates. Hence, as the outside plate is not required, it can be discontinued at H', and $H H'$ will represent one-half its length. In a similar manner, at the section g the required area of the top flange is represented by $g' G'$, which is equal to $O' G$, and this represents the sum of the areas of the two angles and one plate. Hence, as the two outside plates are not required, the second plate can be discontinued at G'. By the same method, the section at which any plate is no longer required can be found when there are any number of plates.

EXAMPLES FOR PRACTICE

1. The web of a plate girder is 84 in. $\times \frac{9}{16}$ in. in cross-section. Each flange has an area of section equal to 36 square inches, and the vertical distance between their centers of gravity is 83 inches. What is the section modulus of the cross-section? Ans. 3,478

2. A plate girder having a 56″ × $\frac{1}{2}$″ web is subjected to a maximum bending moment of 1,792,000 foot-pounds. If the allowable intensity of bending stress is 16,000 pounds per square inch, what is the trial value that would be used for the area of cross-section in the design of each flange: (*a*) if the web is assumed to assist in resisting the bending moment? (*b*) if the flanges are assumed to resist the entire bending moment?

Ans. $\begin{cases} (a) \ 20.5 \text{ sq. in.} \\ (b) \ 24 \text{ sq. in.} \end{cases}$

3. The required flange areas at several sections of a deck plate girder 64 feet long are as follows: at 8 feet from the end, 21 square inches; at 16 feet from the end, 36 square inches; at 24 feet from the end, 45 square inches; and at the center of the span, 48 square inches. The upper flange is made up as follows:

	Square Inches
$th \div 8$	= 4.12
Two angles, 6 in. × 6 in. × $\frac{3}{4}$ in. @ 8.44	= 16.88
Three plates, 16 in. × $\frac{7}{16}$ in. @ 7.00	= 21.00
One plate, 16 in. × $\frac{3}{8}$ in. @ 6.00	= 6.00
	48.00

What are the theoretical lengths of the four flange plates to the next larger whole foot? Ans. 23 ft.; 34 ft.; 42 ft.; 48 ft.

DESIGN OF WEB

9. Longitudinal Shearing Stress.—The distribution of stress in the cross-section of a beam can be represented as in Fig. 3, in which the lines with arrowheads represent the intensities of stress at different distances from the neutral axis. All the stresses acting on the section above the neutral axis are in one direction; all below, in the other direction. On this account, there is a tendency for the portion of the beam on one side of the neutral plane to slide horizontally on the portion on the other side. The same is true at any horizontal section, such as xx, the part above xx tending to slide on or shear away from the part below the plane xx. This tendency causes a shearing stress called the **longitudinal shearing stress** or **longitudinal shear,**

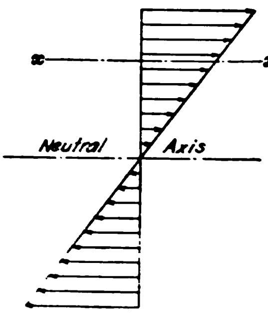


FIG. 3

which decreases as the distance from the neutral axis increases, is greatest at the neutral axis, and is zero at the outside of the section. It can be shown by advanced mathematics that the longitudinal shearing stress in a beam is given by the formula

$$s_1' = \frac{VG}{I} \qquad (1)$$

in which s_1' = longitudinal stress, in pounds *per linear inch* of beam (that is, the stress that would occur in a length of the beam equal to 1 inch, if the stress in all that portion had the same intensity as at the section considered);

V = maximum vertical shear, in pounds, on the section considered;

I = moment of inertia, about the neutral axis, of the entire section, derived from the dimensions of the cross-section in inches;

G = static moment, about neutral axis, of all that part of section outside of point considered (that is, the product of the area of that part of the section and the distance from its center of gravity to the neutral axis).

If the thickness of the beam is denoted by t, the intensity s_1 of longitudinal shear, per square inch, is equal to $s_1' \div t$; that is,

$$s_1 = \frac{s_1'}{t} = \frac{VG}{tI} \qquad (2)$$

In calculating longitudinal shear, it is immaterial whether the vertical shear is positive or negative, as only the numerical value is required.

10. Variation in Shearing Stress.—As the vertical shear V is greater near the end than at the center, the longitudinal shear is also greater at the end than at the center. Considering a vertical section of the beam, V and I are constant for that section; that is, they do not vary, no matter what part of the section is considered; G decreases as the point at which the longitudinal shear is desired is taken farther from the neutral axis, and is zero at the outside of the section. Therefore, according to formula **2**, Art. **9**, *the maximum intensity of longitudinal shear occurs at the neutral axis, near the end of the span.*

11. Web Shear.—It can be shown that the intensity of vertical shearing stress at any point in the cross-section of a beam is equal to the intensity of longitudinal shear at the same point. The maximum intensity of vertical shear at any section of a plate girder can therefore be found by applying the formula $s_1 = \frac{VG}{tI}$ to a point at the neutral axis of the section. In Art. **4**, it was found that the value of I is approximately $\frac{A h_g^2}{2} + \frac{t h^3}{16}$. The static moment G is the sum of the static moment of the area A of one flange and the static moment of the area $\frac{t h}{2}$. The distance of the center of gravity of A from the neutral axis is $\frac{h_g}{2}$; and, therefore, the static moment of A is $A \times \frac{h_g}{2}$. The distance of the center of gravity of the area of one-half the web from the neutral axis is $\frac{h}{4}$; and, therefore, the static moment of that area is

$$\frac{t h}{2} \times \frac{h}{4} = \frac{t h^2}{8}$$

Therefore,
$$G = \frac{A h_g}{2} + \frac{t h^2}{8}$$

Substituting in formula **2**, Art. **9**, the values just found for I and G, there results

$$s_1 = \frac{V\left(\frac{A h_g}{2} + \frac{t h^2}{8}\right)}{t\left(\frac{A h_g^2}{2} + \frac{t h^3}{16}\right)} \qquad (1)$$

As the terms $\frac{t h^2}{8}$ and $\frac{t h^3}{16}$ are small, compared to the other terms, they may be omitted without any error worth considering in practice. Formula **1** then becomes

$$s_1 = \frac{V \times \frac{A h_g}{2}}{t \times \frac{A h_g^2}{2}} = \frac{V}{t h_g}$$

or, approximately, since h_g is very nearly equal to h,

$$s_1 = \frac{V}{t h} \qquad (2)$$

The product $t h$ is equal to the area of cross-section of the web. Formula **2** may, therefore, be stated in the form of a rule as follows:

Rule.—*To find the maximum intensity of shearing stress in the web of a plate girder at any section, divide the maximum vertical shear, in pounds, by the gross area of cross-section of the web in square inches.*

12. Stiffeners.—In *Bridge Members and Details*, it was explained that a flat plate, such as the web of a plate girder, tends to buckle when subjected to a shearing stress, and for this reason it is stiffened by angles riveted to the sides of the plate. The holes for the rivets in the stiffeners decrease the cross-section of the plate, and it has been found in practice that the effective or net section through such a row of holes is very nearly 75 per cent. of the gross section. To allow for the decrease in section, it is customary to specify that the intensity of shearing stress found by the formula $s_1 = \frac{V}{t h}$ at any point shall not exceed 75 per cent. of the allowable intensity of shearing stress.

EXAMPLE 1.—The maximum vertical shear at a given section of a plate girder is 180,000 pounds, and the area of cross-section of the web is 30 square inches. What is the maximum intensity of shearing stress?

SOLUTION.—In this case, V is 180,000 lb. and $t h$ is 30 sq. in. Substituting in formula **2**, Art. **11**,

$$s_1 = 180{,}000 \div 30 = 6{,}000 \text{ lb. per sq. in.} \quad \text{Ans.}$$

EXAMPLE 2.—The maximum vertical shear at a given section of a girder is 240,000 pounds. If the width of web is 72 inches, and the clear unsupported distance at the given section is 26 inches, what thickness must be used in order that the intensity of shearing stress shall not exceed the values given in Table XXXVI?

Solution.—In an example of this kind, which is common, it is necessary to assume either the intensity of shearing stress or the thickness of web. The experienced designer can, as a rule, assume the thickness of web pretty close to the actual thickness, and later correct his assumption. In the present case, the intensity cannot exceed 9,000 lb. per sq. in. (see Table XXXVI); therefore, the cross-section of the web cannot be less than 240,000 ÷ 9,000 = 26.667 sq. in., and the thickness cannot be less than this divided by the height, or 26.667 ÷ 72 = .37 in., say $\frac{3}{8}$ in. The area of cross-section of a 72″ × $\frac{3}{8}$″ plate is 27 sq. in.; then, the actual intensity of shear, if a $\frac{3}{8}$-in. plate is used, will be 240,000 ÷ 27 = 8,888 lb. per sq. in. Consulting Table XXXVI, it is found that the allowable intensity of shear in a $\frac{3}{8}$-in. plate with an unsupported width of 26 in. is 4,600 lb. per sq. in. As this is so much less than the actual intensity, a thicker plate must be tried. The difference between the actual and the allowable intensity being so great, the next thickness, $\frac{7}{16}$ in., will not be considered, but a plate $\frac{1}{2}$-in. thick will be tried.

The area of cross-section of a 72″ × $\frac{1}{2}$″ plate is 36 sq. in.; then, the actual intensity of shear, if a $\frac{1}{2}$-in. plate is used, will be 240,000 ÷ 36 = 6,667 lb. per sq. in. Consulting Table XXXVI, it is found that the allowable intensity of shear is 6,400 lb. per sq. in. As this is less than the actual intensity, a thicker plate must be used. The next thickness, that is, $\frac{9}{16}$ in., will be tried. The area of cross-section of a 72″ × $\frac{9}{16}$″ plate is 40.5 sq. in.; then, the actual intensity of shear, if a $\frac{9}{16}$-in. plate is used, will be 240,000 ÷ 40.5 = 5,930 lb. per sq. in. Consulting Table XXXVI, it is found that the allowable intensity of shear is 7,000 lb. per sq. in. As this is greater than the actual intensity, the assumed thickness is sufficient. Therefore, a 72″ × $\frac{9}{16}$″ plate may be used.

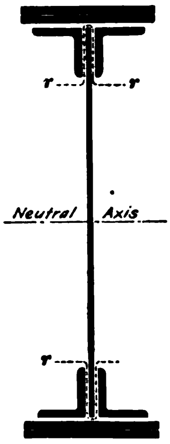


Fig. 4

13. Pitch of Flange Rivets.—The longitudinal shear in a beam of solid cross-section is resisted by the shearing strength of the material. That in a beam built up of various simple rolled sections is resisted by the rivets that hold the different parts together so as to make them act as one piece. The longitudinal shearing stress in a plate girder tends to cause the flange angles to slide on the web. In finding this stress, it is customary to consider a section, such as *r r*, Fig. 4, made between the flange angles and the web; this section cuts only the rivets that connect the vertical legs of the flange angles to the web. The longitudinal shearing stress on the section rr, per unit of length, is found by the formula in Art. 9,

$$s_1' = \frac{VG}{I}$$

In this case, G is simply the static moment of the flange about the neutral axis, or $\frac{A h_g}{2}$. Substituting this value for G, and using the value of I found in Art. 4, the formula becomes

$$s_1' = \frac{V\left(\frac{A h_g}{2}\right)}{\frac{A h_g^2}{2} + \frac{t h^3}{16}}$$

or, neglecting in the denominator the term $\frac{t h^3}{16}$, which is very small compared with $\frac{A h_g^2}{2}$,

$$s_1' = \frac{V\left(\frac{A h_g}{2}\right)}{\frac{A h_g^2}{2}} = \frac{V}{h_g}$$

If the pitch of the flange rivets is p, then, since the longitudinal shear per unit of length is s_1', the stress K that is transmitted by one rivet is $p s_1'$, or $p \times \frac{V}{h_g}$. In computing rivet pitch, K is the value of one rivet. We have, therefore,

$$K = \frac{pV}{h_g} \qquad (1)$$

and

$$p = \frac{K h_g}{V} \qquad (2)$$

As h_g does not materially differ from the distance h_r between the rivet lines in the vertical legs of the flange angles, it is the general practice, and one that will be followed in this Course, to replace h_g by h_r in the preceding formulas. Those formulas then become

$$K = \frac{pV}{h_r} \qquad (3)$$

$$p = \frac{K h_r}{V} \qquad (4)$$

The distance h_r is always less than h_g, and so formula **4** gives a smaller pitch than is actually required; that is, the error is on the side of safety.

EXAMPLE 1.—At a given section of a plate girder, the maximum vertical shear is 72,000 pounds; the distance between the rivet lines of the flanges is 27 inches; and the pitch of the flange rivets is 3 inches. How much stress is transmitted by each rivet?

SOLUTION.—Substituting the given values in formula **3**,

$$K = \frac{3 \times 72{,}000}{27} = 8{,}000 \text{ lb. Ans.}$$

EXAMPLE 2.—The maximum vertical shear at a given section of a plate girder is 150,000 pounds; the value of a flange rivet is 9,600 pounds; and the vertical distance between the rivet lines of the flanges is 37.5 inches. What is the required pitch of the rivets?

SOLUTION.—To use formula **4**, we have $V = 150{,}000$ lb., $h_r = 37.5$ in., and $K = 9{,}600$ lb. Substituting these values in the formula, we have

$$p = \frac{9{,}600 \times 37.5}{150{,}000} = 2.4 \text{ in. Ans.}$$

EXAMPLES FOR PRACTICE

1. The maximum vertical shear at a given section of a girder is 240,000 pounds. The width of web is 64 inches, and the thickness is $\frac{9}{16}$ inch. What is the maximum intensity of shearing stress?
Ans. 6,667 lb. per sq. in.

2. The maximum vertical shear at a given section of a girder is 100,000 pounds. If the width of web is 36 inches, and the clear unsupported distance is 24 inches, what thickness must be used in order that the intensity of shearing stress shall not exceed $\dfrac{12{,}000}{1 + \dfrac{d^2}{3{,}000\,t^2}}$?
Ans. $\frac{1}{2}$ in.

3. The maximum vertical shear at a given section of a plate girder is 102,000 pounds; the distance between the rivet lines of the flanges is 34 inches; and the pitch of the flange rivets is 2.5 inches. How much stress is transmitted by each rivet? Ans. 7,500 lb.

4. The maximum vertical shear at a given section of a plate girder is 178,200 pounds; the value of a flange rivet is 10,800 pounds; and the vertical distance between the rivet lines of the flanges is 49.5 inches. What is the required pitch of rivets? Ans. 3 in.

GENERAL DESIGN

14. Deck and Half-Through Girder Bridges.—The design of a plate girder requires the calculation of the maximum shears and moments at several sections along the girder.

In a half-through plate-girder bridge, the load is applied to the girders at the panel points, and it is simply necessary to calculate the maximum bending moment at each panel point, and the maximum shear in each panel. The bending moment may be assumed to vary uniformly between panel points; that is, the part of the moment curve between two panel points may be assumed to be straight. The shear in each panel may be assumed constant between panel points; then, the intensity of the shearing stress and the rivet pitch will be constant in each panel, but will change at each panel point.

In a deck plate-girder bridge, the load is applied at every point throughout the length of the girder, and it is necessary to compute the maximum moments and shears at several sections along the girder; for convenience, these sections are usually taken from 5 to 10 feet apart. From the values so found, the intensity of the shearing stress, the rivet pitch, and the required flange areas at the different sections are found by means of the formulas already given. The values at intermediate sections can then be found by means of a curve similar to the curve of flange areas.

15. Girders With Curved and Inclined Flanges. The same formulas are used for girders with curved or inclined flanges as for those with parallel flanges. As, however, the height of the girder is different at different sections, the width of the web h, and the vertical distance h_g between the centers of gravity of the flanges, must be calculated at each section at which the flange area, intensity of shearing stress, or rivet pitch is required. The results will be true for the horizontal flange, and for all practical purposes near enough to the true results for the inclined flange, unless the angle between the inclined flange and the horizontal is greater web-plates, and can be procured in greater lengths; angles also can be procured in great lengths, in many cases as long as 100 feet. Owing to practical difficulties in handling long slender pieces, however, it is usually specified that all flange members shall be spliced when they are longer than about 70 feet.

19. Web Splices.—The different portions of which the web is composed are made the full height of the girder and rectangular, with the ends at right angles to the flanges. The ends of two consecutive portions are placed together and covered by plates, called **splice plates**, on each side. These plates are riveted to the ends of the portions of the web in the manner represented in Fig. 7 (a), in which AB is the section at which the web is spliced, and the plate CD is one of the splice plates. The flange angles assist somewhat in splicing the web, but it is customary to ignore their effect, and to design the splice on the assumption that the web is spliced entirely by the splice plates, and to make the splice as strong as the web.

If the splice is designed to resist the same bending moment as the web, there will be no necessity to calculate the shear on the joint, as the joint will always be strong enough in shear. The bending moment M that the web can bear, sometimes called the **resisting moment**, is given by the formula

$$M = \frac{sI}{c} = \frac{s \times \frac{th^2}{16}}{\frac{h}{2}} = \frac{sth^2}{8},$$

using for the moment of inertia the approximate value $\frac{th^2}{16}$ (see Art. 4). If h_1 represents the height and t_1 the combined thickness of the two splice plates, one on each side of the web, $\frac{st_1h_1^2}{8}$ represents the combined resisting moment of the two splice plates. In order that they may have the same resisting moment as the web, we must have

$$\frac{st_1h_1^2}{8} = \frac{sth^2}{8};$$

whence

$$t_1 = t\frac{h^2}{h_1^2}$$

the vertical plates and secondary angles must be taken into consideration.

17. The vertical plates, as well as the main flange angles, are continued the entire length of the girder; the secondary flange angles are usually stopped where they are no longer required. To find the required length of flange members, the curve of flange areas is used as represented in Fig. 6.

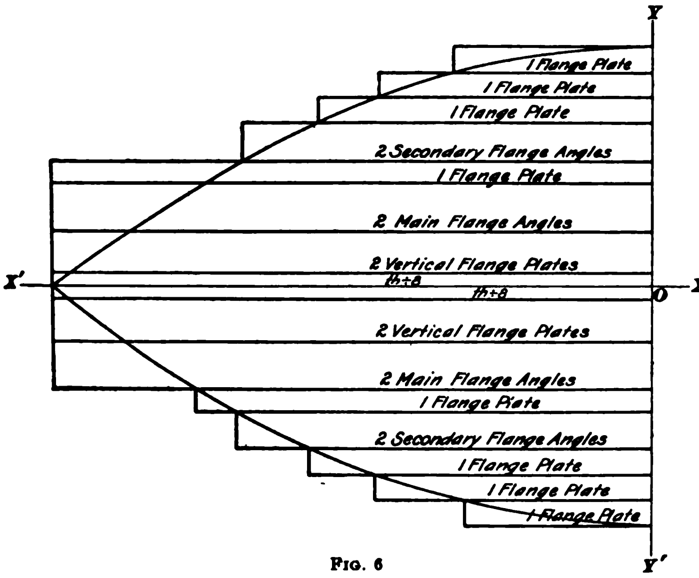


FIG. 6

The value $\frac{th}{8}$ is first laid off on YOY' above and below XOX'; then the area of the two vertical flange plates; then that of the two main flange angles; then the first flange plate; then the secondary flange angles; and then the remaining flange plates, in order.

SPLICES

18. Lengths of Members.—The web and the flange members require to be spliced when they are longer than the lengths given in Table V. Web-plates are generally made as long as possible. Flange plates are made narrower than axis. The resisting moment of a rivet is the product of the stress on the rivet and its distance from the neutral axis, as Kr_0, $K_1 r_1$, $K_2 r_2$, etc. Expressing the stress on each rivet in terms of K, the resisting moments of the rivets are

$$Kr_0 = \frac{Kr_0^2}{r_0},\ K_1 r_1 = K\frac{r_1}{r_0}r_1 = \frac{Kr_1^2}{r_0},\ K_2 r_2 = \frac{Kr_2^2}{r_0}, \text{ etc.}$$

Therefore, for the combined resisting moment M_r of all the rivets on one side of the splice, we have

$$M_r = \frac{Kr_0^2}{r_0} + \frac{Kr_1^2}{r_0} + \frac{Kr_2^2}{r_0} + \ldots = \frac{K}{r_0}(r_0^2 + r_1^2 + r_2^2 + \ldots)$$

or, denoting by Σr^2 the sum $r_0^2 + r_1^2 + r_2^2 + \ldots$ of the squares of the distances r_0, r_1, r_2, etc.,

$$M_r = \frac{K}{r_0}\Sigma r^2 \qquad \textbf{(1)}$$

It should be clearly understood that, in applying this formula, the distance of *every rivet* from the neutral axis should be squared, and the results added. When several rivets are at the same distance from the neutral axis, as is the case when rivets are arranged in horizontal rows, or when some rivets are as far above as others are below the neutral axis, the work is much simplified by squaring that distance and multiplying the result by the number of rivets to which the distance is common. Thus, in Fig. 7 (*a*), each distance, as r_1, is common to four rivets, two above and two below the neutral axis. In this case,

$$\Sigma r^2 = 4(r_0^2 + r_1^2 + r_2^2 + \ldots)$$

In order that the splice rivets may have sufficient strength, the value of M_r must be at least as great as the resisting moment of the web, and K must not exceed the value of the outside rivet. We must, therefore, have

$$\frac{K}{r_0} \times \Sigma r^2 = \frac{sth^2}{8} \qquad \textbf{(2)}$$

In applying this formula, it is customary to assume first a spacing of rivets and calculate their resisting moment. If the value is not sufficient, more rivets are added, either by spacing the rivets closer together, or by adding another row outside of the first two. For this purpose, the arrangement represented in Fig. 8 is frequently employed, the advantage being that most of the rivets are near the flanges, where the value of the resisting moment is greatest. The thickness of these plates may be found by means of the formula $t_1 = t \times \frac{h^2}{h_1^2}$ (Art. **19**), substituting for h_1 the sum of the heights of the three portions of the splice plate; the thickness on each side, however, is usually made about 75 per cent. of that of the web.

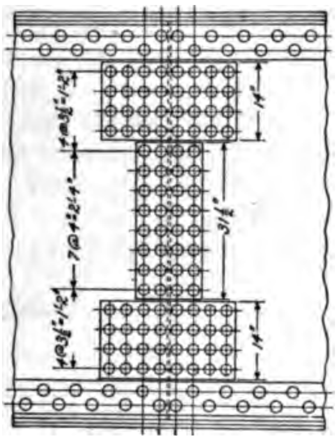


FIG. 8

EXAMPLE.—If the plate girder represented in Fig. 8 has a web 72 in. × ½ in., spliced with ⅞-inch rivets and splice plates as shown, what is: (*a*) the required thickness of splice plates on each side? (*b*) the resisting moment of the rivets on each side of the splice, assuming the value of a rivet to be 9,600 pounds?

SOLUTION.—(*a*) As the splice plates are made up of three parts on each side of the web, the value of h_1 is $14 + 31\frac{1}{2} + 14 = 59.5$ in. Then, since t is .5 and h is 72 in.,

$$t_1 = .5 \times \frac{72^2}{59.5^2} = .5 \times 1.46 = .73 \text{ in.}$$

for both sides, or $.73 \div 2 = .365$ in. (say ⅜ in.) for each side. Ans.

(*b*) The distances of the rivets from the neutral axis are 2, 6, 10, 14, 17½, 21, 24½, and 28 in., respectively. There are four rivets at each of the first four distances, and eight rivets at each of the last four. To apply formula **1**, we have $K = 9{,}600$ lb., $r = 28$ in., and

$$\Sigma r^2 = 4 \times (2^2 + 6^2 + 10^2 + 14^2) + 8 \times (17.5^2 + 21^2 + 24.5^2 + 28^2) = 18{,}396$$

Therefore,

$$M_r = \frac{9600}{28} \times 18{,}396 = 6{,}307{,}200 \text{ in.-lb. Ans.}$$

21. Flange-Angle Splices.—Flange angles are spliced by riveting angles to them, as represented in Fig. 9. In this figure, (*a*) is the elevation of a portion of the bottom flange, (*b*) is the cross-section at *B B*, and (*c*) is the plan of the flange and a cross-section at *C C*. The angles *d* are the flange angles, the angles *e* are the splice angles, and the line *f* is the section at which the flange angle is spliced. Practice varies as to the method of designing a flange-angle splice. Some engineers use one splice angle riveted to the flange angle that is spliced; this is open to the objection that the splice angle must be very long in order to get sufficient rivets to develop the stress, and therefore interferes with other details, such as stiffeners. Others use two angles, as in Fig. 9, and assume that the stress in the flange angle is equally divided between the two. On account of the fact that the angle on the side opposite the splice gets its stress through the web and the other flange angle, it is probable that the angle in contact with the flange angle that is spliced

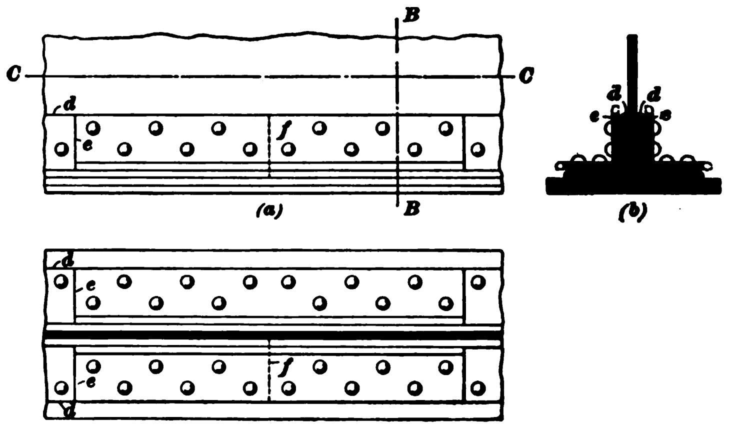


Fig. 9

gets somewhat more than one-half the stress. The assumption most frequently made is that the splice angle in contact with the flange angle that is spliced takes three-quarters of the stress; this angle is designed on this basis to have a cross-section three-quarters that of the flange angle, and made long enough to get sufficient rivets on each side of the splice to transmit three-quarters of the stress. The other splice angle is then made the same size and length. If F_A is the area of section (net area for tension, gross area for compression) of one flange angle, then the area F_A' of section of each splice angle is given by the formula

$$F_A' = .75\, F_A \qquad (1)$$

If s is the allowable intensity of bending stress, the total stress in the flange angle is $F_A s$, and in the splice angle, $.75 F_A s$. The number n of rivets in the splice angle on each side of the splice is given by the formula

$$n = \frac{.75 F_A s}{K} \qquad (2)$$

in which K is the value of one rivet. Splice angles are cut from angles having legs the same width as the flange angles, so that the edges of splice and flange angles will be even, and not as shown by dotted lines in Fig. 9 (*b*). The area of cross-section of such an angle can be found by deducting from the area given in Tables IX and X for the original angle the amount that is sheared off.

EXAMPLE.—The flange angles in the tension flange of a plate girder are 6 in. × 6 in. × ½ in., and the allowable intensity of stress is 16,000 pounds per square inch. The diameter of the rivets is ⅞ inch. (*a*) What size of splice angle must be used if the angle is spliced as represented in Fig. 9? (*b*) If the value of one rivet is 5,000 pounds, how many rivets are required on each side of the splice?

SOLUTION.—(*a*) The area of a 6″ × 6″ × ½″ angle is 5.75 sq. in. As we are considering the tension flange, the net section is required; as there are two rivet holes in each section, and the area of a hole for a ⅞-inch rivet is .5 sq. in., as given in Table XXVII, the net section is $5.75 - 2 \times .5 = 4.75$ sq. in. ($= F_A$). The required net area F_A' of one splice angle is, then, by formula **1**, $.75 \times 4.75 = 3.56$ sq. in. As the flange angle is ½ in. thick, ½ in. must be sheared off each leg of each splice angle. The thinnest 6″ × 6″ angle, which is ⅜ in. thick, will be tried first. The area of this angle, as given in Table IX, is 4.36 sq. in.; if ½ in. is cut from each leg, the area is reduced by $2 \times .5 \times .375 = .37$ sq. in., nearly. As there are two holes in each angle, the area will be still further reduced by $2 \times .375 = .75$ sq. in. Then, the net area of one angle 5½ in. × 5½ in. × ⅜ in. will be $4.36 - .37 - .75 = 3.24$ sq. in. As 3.56 sq. in. is required, this is not enough. The next size, 6 in. × 6 in. × $\frac{7}{16}$ in., the gross area of which is 5.06 sq. in., will be tried. If ½ in. is sheared from each leg, the area is reduced by $2 \times .5 \times .44 = .44$ sq. in. The two rivet holes still further reduce the section by $2 \times .44 = .88$ sq. in. Then, the net area of one angle 5½ in. × 5½ in. × $\frac{7}{16}$ in. is $5.06 - .88 - .44 = 3.74$ sq. in., which is sufficient. Ans.

(*b*) The total stress in one flange angle is $4.75 \times 16{,}000 = 76{,}000$ lb., and the portion to be transmitted by the rivets is three-fourths of this, or 57,000 lb. Then, the required number of rivets is 57,000 ÷ 5,000 = 11.4, or, say, 12 rivets on each side of the splice.

22. Flange-Plate Splices.—Flange plates are spliced either by additional plates, or by continuing the outer plates beyond their theoretical ends a sufficient distance to splice the plates below them. The plates nearest the flange angles are the longest, and are the ones that require to be spliced.

23. Additional Splice Plates.—In the first method of splicing, the joint in the plate that is to be spliced is usually

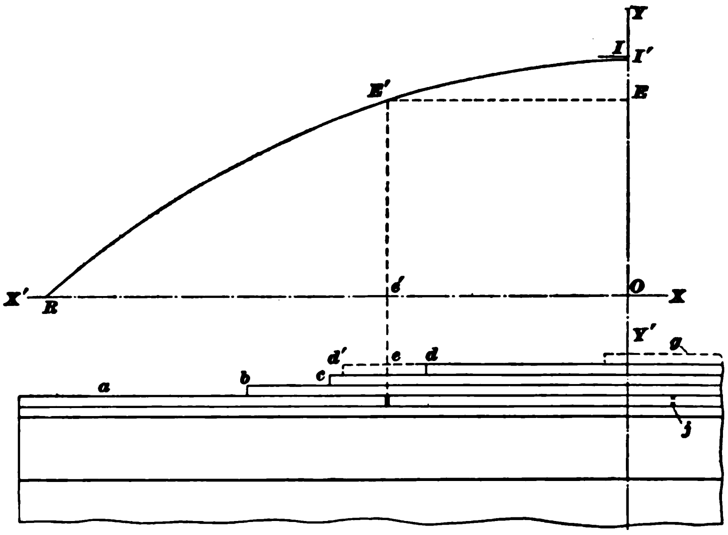


FIG. 10

located somewhere near the center of the girder, as represented at j, Fig. 10, and a splice plate g of the same thickness as the flange plate is riveted to the outside of the flange. The splice plate is made long enough to contain sufficient rivets on each side of the joint to properly transmit the stress from one part of the flange plate to the other. The number of rivets that is required to transmit the stress to or from the splice plate can be found by dividing the stress by the value of one rivet; the latter will usually be the value in single shear. When, as in Fig. 10, there are intermediate plates between the splice plate and the plate that is spliced, there is some uncertainty as to how the stress is carried around the joint; to provide for this, the number of rivets in the splice plate is increased as specified in Arts. **61** and **134** of *B. S.* In the present case, since there are three intermediate plates, the number of rivets will be increased $3 \times 20 = 60$ per cent. Any flange plate may be spliced in the same way.

24. Continuing Outer Plates.—In the second method of splicing, the location of the joint in the plate that is to be spliced is found by means of the curve of flange areas. In Fig. 10, the curve of flange areas and one-half of the flange are laid out to the same horizontal scale; the points *d*, *c*, and *b* are the theoretical ends of the three outside flange plates, and it is desired to splice the first flange plate. The ordinate OI' represents the flange area as found by the formula $A = \frac{M}{s\,h_s}$ (Art. **6**), and OI represents the actual area of flange. From I, IE is laid off to scale to represent the area of the plate that is to be spliced; the line EE' is drawn parallel to $X'OX$ to its intersection E' with the curve, and the line $E'e$ is drawn perpendicular to $X'OX$. Then, $e'E'$ represents the entire area of flange, exclusive of the plate *a*; that is, if all the plates are carried beyond *e*, the first plate *a* can be spliced at that section. For this purpose, the outer plate, instead of being stopped at *d*, is carried beyond *e*, as represented by the dotted lines. As there is no stress in the first flange plate *a* at the section *e*, and there is full stress in the outer plate *d*, it remains to find how many rivets must be contained in the plate *d* beyond *e*, in order to transmit its stress to *a*; this is found by dividing the stress in the outside plate by the value of one rivet in single shear. In the present case, as there are two plates between *d* and *a*, the required number of rivets will be $2 \times 20 = 40$ per cent. greater than the theoretical number. The plate *d* will be continued to a point d', the distance $e\,d'$ being made such that there will be sufficient room for the required number of rivets. In a similar manner, any flange plate may be spliced.

It will seldom be found necessary to splice any flange plate at more than one section, nor more than two plates in any flange. When two plates must be spliced, the first plate can be spliced at one end of the girder, as at *e*, Fig. 10, and the second plate at a corresponding point on the other side of the center.

EXAMPLE.—Let Fig. 10 represent the top flange of a plate girder in which the allowable intensity of stress is 16,000 pounds per square inch, and the sizes of the plates are as follows: *a*, 16 in. × $\frac{1}{2}$ in.; *b*, 16 in. × $\frac{1}{2}$ in.; *c*, 16 in. × $\frac{7}{16}$ in.; and *d*, 16 in. × $\frac{3}{8}$ in. (*a*) If it is desired to splice plate *a* at section *j*, what is the required size of the splice plate *g*? (*b*) If the value of one rivet in single shear is 6,600 pounds, how many rivets must be included in plate *g*? (*c*) If it is desired to splice plate *a* at section *e*, how many rivets must be included in plate *d* between *e* and *d'*, assuming the value of one rivet to be 6,600 pounds?

SOLUTION.—(*a*) As plate *g* is an additional splice plate, it must be the same size as the plate that is to be spliced, that is, 16 in. × $\frac{1}{2}$ in. Ans.

(*b*) The area of cross-section of plate *g* is 8 sq. in., and, since the intensity of stress is 16,000 lb. per sq. in., the stress in plate *g* is 8 × 16,000 = 128,000 lb. As the value of one rivet in single shear is 6,600 lb., the number of rivets required to transmit this stress to plate *g* is 128,000 ÷ 6,600 = 19.4. Since there are three intermediate plates between *g* and *a*, the number must be increased 3 × 20 = 60 per cent. Then, the total number of rivets required on each side of *j* is

$$19.4 + \frac{60}{100} \times 19.4 = 31$$

As the flange rivets in the plates are driven in pairs, it is necessary to have 32 rivets. Ans.

(*c*) The area of cross-section of plate *d* is 16 in. × $\frac{3}{8}$ in. = 6 sq. in.; and, since the intensity of stress is 16,000 lb. per sq. in., the stress in plate *d* is 6 × 16,000 = 96,000 lb. As the value of one rivet is 6,600 lb., the number of rivets required to transmit the stress to the plate *d* is 96,000 ÷ 6,600 = 14.5 rivets. Since there are two intermediate plates between *d* and *a*, the number must be increased 2 × 20 = 40 per cent. Then, the total number of rivets required in plate *d* between *d'* and *e* is

$$14.5 + \frac{40}{100} \times 14.5 = 20.3$$

As this is so close to 20, 20 rivets will be sufficient, although it is better to use 22. Ans.

25. Splices in Secondary Flange Angles and Vertical Flange Plates.—Secondary flange angles are usually spliced, as represented in Fig. 11, by means of one splice angle riveted to the inside of the flange angle and a splice plate having a width about the same as that of the flange angle, riveted to the back of the latter. The area of the splice angle is made about 75 per cent. and that of the splice plate about 25 per cent. that of the flange angle, the area of the two together being not less than the area of the flange angle.

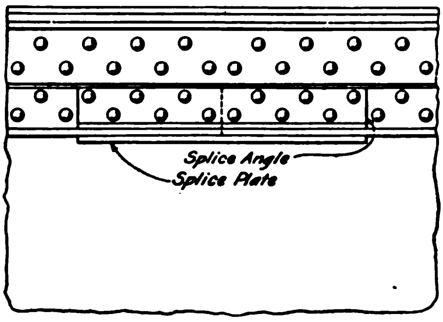


FIG. 11

Vertical splice plates are spliced, as represented in Fig. 12, by means of one vertical splice plate riveted to the vertical legs of the flange angles on the same side of the web as the vertical plate that is spliced. The area of the splice plate is made not less than that of the flange plate.

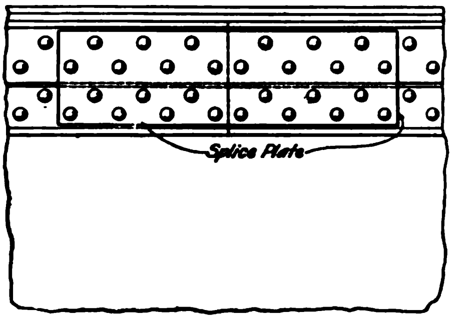


FIG. 12

EXAMPLES FOR PRACTICE

1. What is the resisting moment of a $36'' \times \frac{5}{8}''$ web, if the maximum intensity of the bending stress is 16,000 pounds per square inch?
Ans. 1,620,000 in.-lb.

2. If the web represented in Fig. 7 is 70 inches wide and $\frac{9}{16}$ inch thick, what is the required thickness of splice plate on each side, assuming the height to be 57.5 inches? Ans. $\frac{7}{16}$ in. thick on each side

3. If the rivets in Fig. 7 are spaced as shown at the left-hand side, and the value of one rivet is 10,800 pounds, what is the resisting moment of the rivets in the splice? Ans. 3,849,600 in.-lb.

4. The flange angles in the compression flange of a plate girder are 8 in. × 8 in. × $\frac{3}{4}$ in., and the allowable intensity of stress is 16,000 pounds per square inch. (*a*) What size of splice angle must be used? (*b*) If the value of one rivet is 6,600 pounds, how many rivets are required in the splice angle?

Ans. $\begin{cases} (a) \text{ 2 angles, 8 in.} \times \text{8 in.} \times \frac{5}{8} \text{ in., cut to } 7\frac{1}{4} \text{ in.} \times 7\frac{1}{4} \text{ in.} \times \frac{5}{8} \text{ in.} \\ (b) \text{ 22 rivets on each side of the joint} \end{cases}$

5. (*a*) If the vertical flange plate in Fig. 12 is 15 inches wide and $\frac{1}{2}$ inch thick, and the splice plate is 13 inches wide, what is the required thickness of the latter? (*b*) If the allowable intensity of stress is 16,000 pounds per square inch, and the value of one rivet is 6,600 pounds, how many rivets are required in the splice plate?

Ans. $\begin{cases} (a) \ \frac{5}{8} \text{ in. thick} \\ (b) \text{ 18 or 20 rivets on each side of the joint} \end{cases}$

BEARINGS

26. Size of Bedplates.—The end of a girder, where the girder rests on the masonry or other support, must be

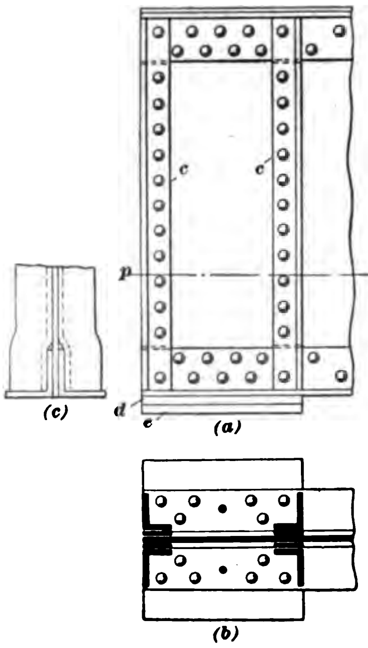


FIG. 13

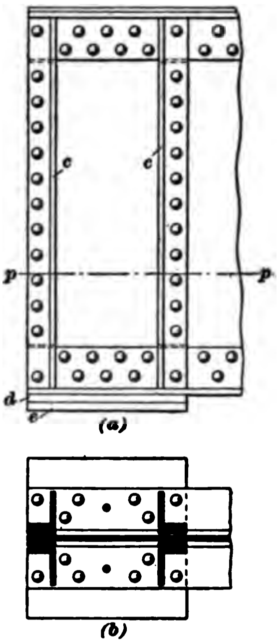


FIG. 14

strong enough to resist the reactions. The required area of bearing, if the girder rests on masonry, is found by dividing the maximum reaction by the allowable intensity of bearing on the masonry. For example, if the reaction is 200,000 pounds, and the allowable intensity of bearing is 500 pounds per square inch, the required area of bearing is 200,000 ÷ 500 = 400 square inches. Bedplates are usually made rectangular and very nearly square; in the present case, each will need to be about $\sqrt{400} = 20$ inches on each side.

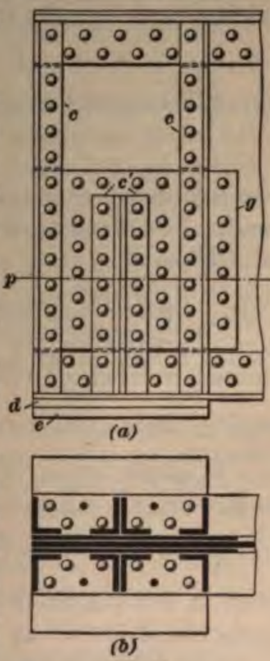


FIG. 15

27. End Stiffeners.—The ends of plate girders are stiffened by stiffeners at each end of the bedplates. The arrangements usually employed are represented in Figs. 13, 14, and 15, in which c, c are the stiffeners; d, the sole plates; and e, the bedplates. The arrangement represented in Fig. 13 is most used, although that shown in Fig. 14 is somewhat better on account of the fact that in it the bearing of the stiffeners is not so close to the edge of the bedplate; the pressure is therefore more evenly distributed over the area of the bedplate. In the arrangement represented in Fig. 15, the additional stiffeners c' are added. The plates g are called **reinforcing plates,** and are added to distribute the stress more evenly over the web.

28. Distribution of Reaction.—The reaction is assumed to be equally divided among the end stiffeners, and evenly distributed over the area of the outstanding legs of the stiffeners where they bear on the upper side of the lower flange angle. The length of the portion of a stiffener in contact with the lower flange angle is usually about $\frac{1}{2}$ inch less than the nominal width of the outstanding leg. If t' is the thickness of a stiffener angle; b, the nominal width of the outstanding leg; and s_b, the allowable intensity of bearing on the end of a stiffener, the stress that one stiffener can resist is $t'(b - \frac{1}{2})s_b$. If R is the reaction and n the number of end stiffeners, the amount of pressure that is transmitted by each stiffener is $\frac{R}{n}$; and, in order that the stiffener may be sufficiently strong, the following equation must be satisfied:

$$t'(b - \tfrac{1}{2})s_b = \frac{R}{n}$$

From this equation, we have

$$t' = \frac{R}{n\,s_b(b - \frac{1}{2})},$$

from which the required thickness of the stiffeners can be found. The nominal width of leg is controlled by Arts. **55** and **128** of *B. S.*

29. Rivets and Stiffeners.—The stress in each stiffener is transmitted to the web by means of rivets. The required number of rivets in a stiffener can be found by dividing the stress in one stiffener $\left(\frac{R}{n}\right)$ by the value of a rivet in single shear, or in bearing on the angle; or, since the same rivets connect two opposite angles, by dividing the stress in two stiffeners $\left(\frac{2R}{n}\right)$ by the value of a rivet in double shear, in bearing on two angles, or in bearing on the web, whichever is least. In practice, it is considered advisable to transmit the greater part of the stress in the stiffeners to the web below the neutral axis. When it is impossible to get sufficient rivets below the neutral axis in four stiffeners, as in Figs. 13 and 14, more stiffeners are used, as in Fig. 15.

30. Crimped Stiffeners.—Stiffeners are sometimes placed in contact with the web, and the ends **crimped**; that is, bent out around the vertical legs of the flange angles, as represented in Fig. 13 (*c*). This arrangement is open to the objection that it is difficult to bend the angles so they will bear evenly on the flange angle, especially on the outstanding leg.

31. Loose Fillers.—The best practice at the present time is to make the stiffener angles straight from top to bottom, and fill in the space between them and the web by means of bars the same width as the adjacent leg of the stiffener and the same thickness as the flange angles, as represented in Figs. 13 and 14. These bars are called **loose fillers**; they simply serve to fill the space between the angle and the web. The number of rivets required to connect the stiffener to the web must be increased 20 per cent. when this form of filler is used.

32. Reinforcing Plates or Tight Fillers.—The filler under the stiffeners in Fig. 15 is continued under all the angles, and riveted to the web by rivets located outside of the stiffeners. The filler is then called a **tight filler**, or **reinforcing plate**; it distributes the stress over the area of the web. Such a plate is not to be considered an intermediate plate, but rather a part of the web, as it is firmly riveted to it, and in calculating the bearing value of the rivet on the web the thickness of these plates must be included.

EXAMPLE 1.—The maximum reaction at the end of the plate girder represented in Fig. 13 is 135,000 pounds. (*a*) If the stiffeners are 5 in. × $3\frac{1}{2}$ in., and the allowable intensity of bearing is 18,000 pounds per square inch, what is the required thickness of the stiffener angles? (*b*) If the value of one rivet in single shear is 6,600 pounds, in bearing on the web, 10,800 pounds, and in bearing on each stiffener angle, 8,400 pounds, how many rivets are required to connect the stiffeners to the web?

SOLUTION.—(*a*) The required thickness of stiffeners can be found by the formula in Art. **28**,

$$t' = \frac{R}{n\, s_b\, (b - \frac{1}{2})}$$

In the present case, $R = 135{,}000$, $n = 4$, $s_b = 18{,}000$, and $b = 5$. Substituting these values in the formula gives

$$t' = \frac{135{,}000}{4 \times 18{,}000 \times 4.5} = .42 \text{ in., or } \tfrac{7}{16} \text{ in., nearly. Ans.}$$

(*b*) The rivets that connect the stiffeners to the web are in single shear on each side of the web, and in bearing on the $\frac{7}{16}$-inch stiffener angle. Considering first the stress in one angle, the value in single shear is evidently less than the value in bearing on the $\frac{7}{16}$-inch angle.

The stress in each stiffener is 135,000 ÷ 4 = 33,750 lb., and the required number of rivets is 33,750 ÷ 6,600 = 5.1.

Considering now the stress in two angles, the rivets are in double shear at 2 × 6,600 = 13,200 lb.; in bearing on two $\frac{7}{16}$-inch angles, at 2 × 8,400 = 16,800 lb.; and in bearing on the web, at 10,800 lb. The latter value being the smallest, and the stress in two stiffeners being 135,000 ÷ 2 = 67,500 lb., the required number of rivets is 67,500 ÷ 10,800 = 6.25. This number is larger than that first found, and must be used. As there are loose fillers (see Art. **31**), the actual number required is

$$6.25 + \tfrac{20}{100} \times 6.25 = 7.5, \text{ say, 8 rivets. Ans.}$$

EXAMPLE 2.—The maximum reaction at the end of the plate girder represented in Fig. 15 is 230,000 pounds; the thickness of web is $\frac{1}{2}$ inch, and the thickness of the flange angles is $\frac{5}{8}$ inch. Using the working stresses given in Art. **29** of *B. S.*, it is desired to find: (*a*) the required thickness of stiffeners, if the outstanding leg is 5 inches wide; (*b*) the number of rivets required to connect the stiffeners to the web, if $\frac{7}{8}$-inch rivets are used.

SOLUTION.—(*a*) In Art. **29** of *B. S.*, the allowable intensity of bearing on the ends of stiffeners is given as 18,000 lb. per sq. in. Then, since R = 230,000, n = 8, and b = 5, we have, applying the formula in Art. **28**,

$$t' = \frac{230,000}{8 \times 18,000 \times 4.5} = .355, \text{ or, say, } \tfrac{3}{8} \text{ in. Ans.}$$

(*b*) Considering a single stiffener, the value in bearing on a $\frac{3}{8}$-inch angle, as given in Table XL, is 7,220 lb.; and in single shear, 6,600 lb. The latter is the smaller. The stress in one stiffener is 230,000 ÷ 8 = 28,750 lb., and the required number of rivets is 28,750 ÷ 6,600 = 4.4.

Taking two stiffeners, it is necessary to consider the value of the rivet in double shear at 13,230 lb., in bearing on two stiffeners at 14,440 lb., and in bearing on the combined thickness of web and two reinforcing plates $1\frac{3}{4}$ in. The last is evidently greater than either of the other two values; the value in double shear is the smallest, and must be used. Since the stress in two stiffeners is twice that in one, and the value in double shear is twice that in single shear, the number of rivets will be the same as in the first case considered, that is, 4.4, or, say, 5 rivets in each stiffener. Ans.

33. Rocker Bearings.—The ends of spans over 75 feet in length are supported on rockers and supplied at one end with rollers, as explained in *Bridge Members and Details*. When it is necessary, on account of high water, to keep the bridge seat close to the bottom of the girders, the arrangement represented in Fig. 16 is used. The outstanding legs of the lower flange angles are cut away for a short distance at each end to allow the lower flange to enter between the vertical plates of a pedestal. The web of the

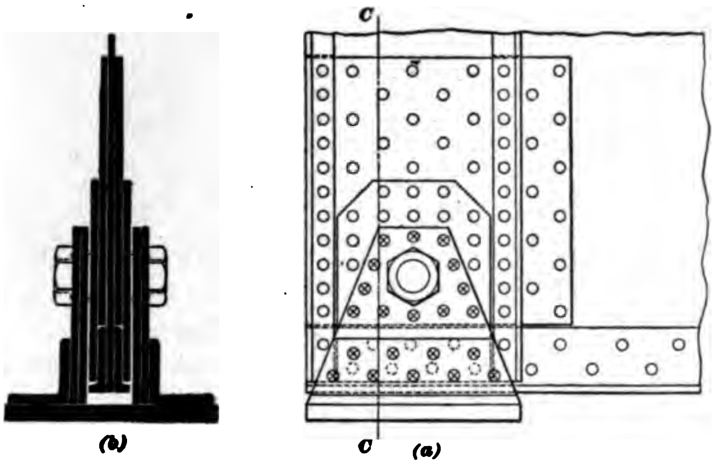


Fig. 16

girder is reinforced at the end, and a pin is passed through the pedestal and web; the pin should never be less than 6 inches in diameter. The design of the pin, pin plates, and pedestal is made by the same general principles that apply to pin-connected trusses, as explained in another Section.

EXAMPLES FOR PRACTICE

1. If the maximum reaction at the end of the plate girder represented in Fig. 13 is 157,000 pounds, stiffeners 5 in. × $3\frac{1}{2}$ in., and allowable intensity of bearing 18,000 pounds per square inch, what is the required thickness of the stiffener angles? Ans. $\frac{1}{2}$ in.

2. If in example 1 the value of a rivet in single shear is 6,600 pounds, and in bearing on the web 15,000 pounds, how many rivets must be used in each stiffener to transmit the stress to the web? Ans. 6 rivets

3. If the maximum reaction at the end of the plate girder represented in Fig. 15 is 275,000 pounds, stiffeners 5 in. × $3\frac{1}{2}$ in., and allowable intensity of bearing 18,000 pounds per square inch, what is the required thickness of the stiffener angles? Ans. $\frac{7}{16}$ in.

DESIGN OF PLATE GIRDERS

(PART 2)

DESIGN OF AN I-BEAM HIGHWAY BRIDGE

1. Introduction.—In this and in the following Sections will be given complete designs of several classes and types of bridges. The designs will be made according to the rules given in *Bridge Specifications* (a title that, for convenience, will be abbreviated to *B. S.*). These examples will familiarize the student with the principles involved and the methods used, which he can without any difficulty extend to forms and conditions not specifically covered in the present instruction.

2. Data.—As a first example, an I-beam highway bridge will be designed from the data given in the data sheet on page 4. The words in *Italics* are supposed to have been written to fill out the general form, which contains only the words printed in Roman type (see *B. S.*, Art. 226).

3. Determination of Span.—It is first necessary to determine the location of the abutments. According to *B. S.*, Art. 18, no part of a bridge should be less than 7 feet from the center line of the nearest track, nor less than 22 feet above the base of the rail. This condition applies also to abutments and underneath clearance lines for overhead bridges. As the faces of abutments are usually rough and extend somewhat beyond the neat lines, it is well to locate the neat lines at the base of the rail 7 feet 6 inches from the center line of the track, making them (as there are two tracks 13 feet center to center) 7 feet 6 inches + 13 feet + 7 feet 6 inches = 28 feet apart. The faces of abutments are sometimes made plumb (vertical), in order to shorten

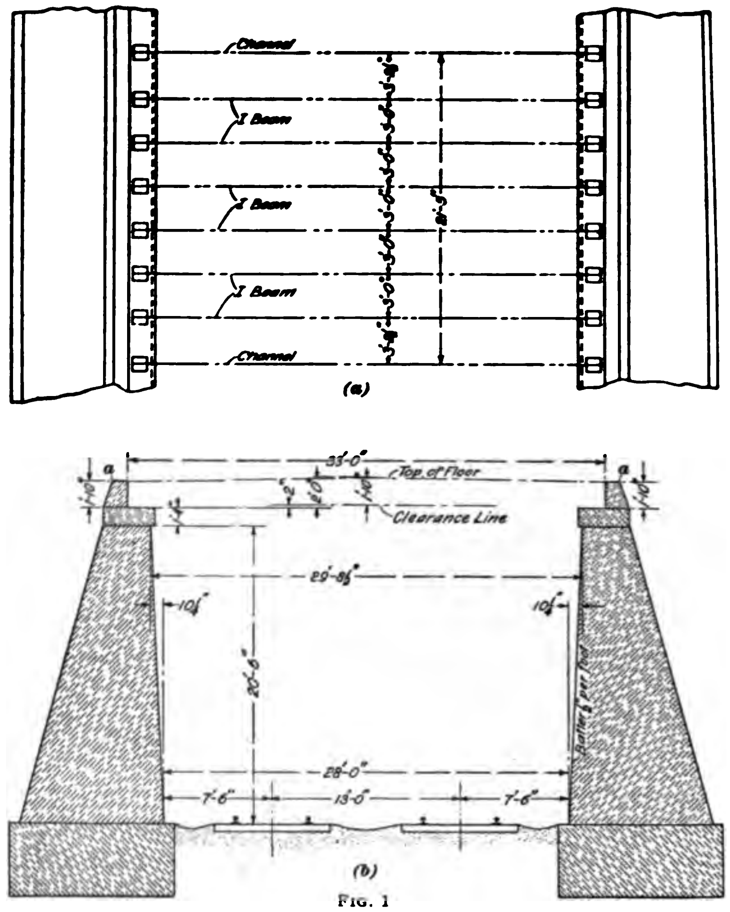


FIG. 1

the span, and sometimes battered, according to the judgment of the engineer. In the present case, allowance will be made for a batter of ½ inch per foot in each abutment; then, the distance between the faces will increase at the rate of $\frac{1}{2}+\frac{1}{2}=1$ inch for every foot above the base of the rail, as far as the bottom of the bridge seat or coping. Bridge seat stones are usually made from 12 inches thick for short spans to 24 inches, and in some cases more, for long spans. In the present case, a thickness of 16 inches will be sufficient. Allowing 1 inch for the thickness of the sole plate and 1 inch for the bedplate makes the distance from the underneath clearance line (the bottom of the I beam) to the bottom of the bridge seat $16+1+1=18$ inches, and the distance from the base of the rail to the bottom of the bridge seat 22 feet − 1 foot 6 inches = 20 feet 6 inches. Then, as the distance between the abutments increases 1 inch for each foot, it will increase $20\frac{1}{2}$ inches, or 1 foot $8\frac{1}{2}$ inches, in 20 feet 6 inches, and the distance between the neat lines under the bridge seat will be 28 feet + 1 foot $8\frac{1}{2}$ inches = 29 feet $8\frac{1}{2}$ inches, as represented in Fig. 1, which is the **bridge-seat plan**, that is, the drawing of the bridge seat. [It will be noticed that here the word *plan* is used in the sense of *drawing* or *drawings*, not in the sense of a top view. In reality, the drawings in Fig. 1 show both a top view (*a*) and a cross-section (*b*).]

4. For a span of this length, the edge of the bedplate should not come closer than 3 or 4 inches to the neat line, and should not be set much farther back than this, as it lengthens the span. In the present case, it will be set $4\frac{3}{4}$ inches back at each abutment, making the clear distance between bedplates 29 feet $8\frac{1}{2}$ inches + $4\frac{3}{4}$ inches + $4\frac{3}{4}$ inches = 30 feet 6 inches. Bedplates are seldom made less than 12 inches in length; this is long enough for this span, and makes the total length of I beams 30 feet 6 inches + 1 foot + 1 foot = 32 feet 6 inches, and the distance center to center of bedplates (the span) 31 feet 6 inches, or 378 inches. The parapets are usually set 3 inches from the ends of the beams; that makes them, in this case, 33 feet apart.

5. Depth and Spacing of Beams.—As this span is less than 35 feet, rolled beams will be used. According to *B. S.*, Art. **92**, they cannot be less than $378 \div 30 = 12.6$ inches in

GENERAL DATA

For bridge over *Delaware, Lackawanna, & Western Railroad*

at *Elmhurst, Pennsylvania*

Length and general dimensions *To span two tracks 13 feet center to center*

Skew or angle of abutments with center line of bridge *90°*

Width of bridge and location of trusses *20 feet clear width. No trusses*

Floor system *One layer of 3-inch oak plank on nailing pieces and steel beams*

Number and location of tracks *No tracks*

Loading *Art. 98 (3) Bridge Specifications*

Description of abutments *Cement-concrete abutments*

Distance from floor to clearance line *Not more than 3 feet*

" " " " high water

" " " " low water

" " " " river bottom

Character of river bottom

Usual season for floods

Name of nearest railroad station *Elmhurst, Pennsylvania, D., L. and W. R. R.*

Distance to nearest railroad station *2 miles*

Time limit *6 months from date of award of contract*

Name of Engineer *International Textbook Company*

Address of Engineer *Scranton, Pennsylvania*

Remarks *To have a tight board fence at each side at least 5 feet 6 inches above the top of the floor* depth. Table XIV* shows that the next depth of beam is 15 inches. In finding the spacing of beams, it is first necessary to determine the distance between the outside beams. In the present case, a roadway of 20 feet between wheel-guards is required; the fence will be placed 6 inches outside of the inner edges of the wheel-guards; then, the clear distance between the fences will be 21 feet. A tight board fence is specified to prevent injury to the traffic from cinders and sparks from the locomotives that will pass underneath.

Fig. 2 shows the cross-section of a good style of fence and connection to the beams; the posts are placed 5 or 6 feet apart. The outside beam is usually a channel, preferably of the same depth as the I beams. To assist in keeping the fence in place and preventing it from being blown over, the channel may be connected to the next beam, at intervals of about 10 feet, by means of the diaphragm shown in Fig. 2.

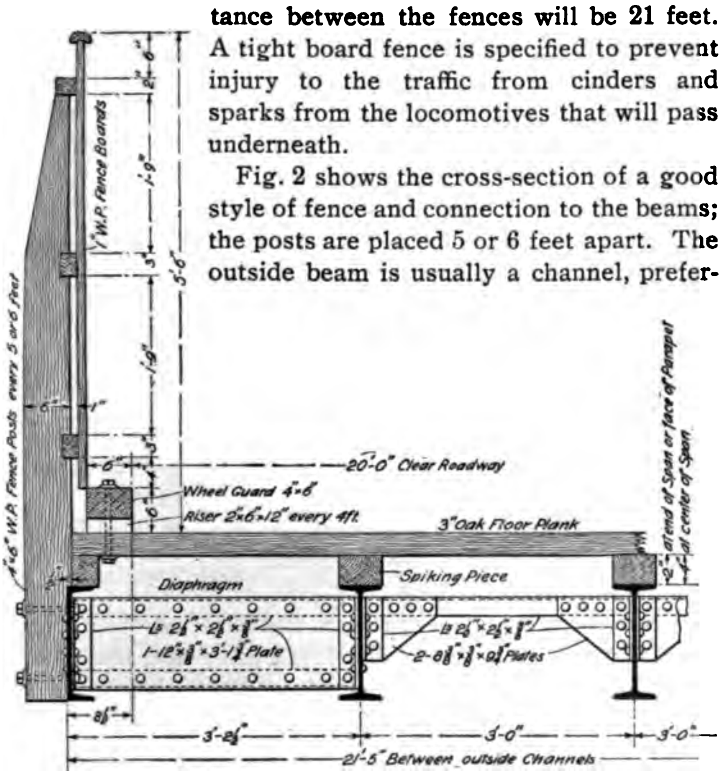


Fig. 2

*All tables referred to in this Section are found in *Bridge Tables*, unless otherwise stated.

This may be made of the lightest material allowed—in this case, as the bridge is over a railroad, ⅜ inch thick (*B. S.*, Art. **112**). With the arrangement represented in Fig. 2, the face of the channel is 8½ inches outside of the edge of the wheel-guard, making the channels 20 feet + 8½ inches + 8½ inches = 21 feet 5 inches from face to face. As there will be one layer of floor planks 3 inches thick, the beams cannot be more than 3 feet center to center (*B. S.*, Art. **123**). Five spaces at 3 feet makes 15 feet, leaving 21 feet 5 inches − 15 feet = 6 feet 5 inches, to be made up in the sides of the bridge. This requires the face of each channel to be placed one-half of 6 feet 5 inches, or 3 feet 2½ inches, from the center of the first beam; if a spiking piece 4 inches wide is used on the channel, its center will be just 3 feet from the center of the next beam. This spacing of beams will satisfy all conditions.

6. Live Load.—According to *B. S.*, Arts. **98** (3) and **110**, the live load should be *either* 80 pounds per square foot *or* a steam road roller acting on each beam as two concentrated loads of 5,000 pounds, 11 feet apart. For the former, as the beams are 3 feet apart, the load per linear foot is 3 × 80 = 240 pounds. The maximum bending moment occurs at the center, and, as the span is 31.5 feet, is

$$\frac{240 \times 31.5 \times 31.5}{8} = 29{,}770 \text{ foot-pounds}$$

The maximum bending moment due to the two concentrated loads occurs under one of the loads when that load and the center of gravity of the two loads are equidistant from the center of the beam, and, as explained in *Stresses in Bridge Trusses*, Part 4, is equal to $\frac{10{,}000 \times 13 \times 13}{31.5} = 53{,}650$ foot-pounds at 2.75 feet from the center of the span. As the bending moment due to the road roller is the greater, it is unnecessary to further consider that due to the uniform load. The shear will not be considered in this example, as it has been found in practice that, in all ordinary cases in bridge work, an I beam or channel that is strong enough to resist the bending moment is also strong enough to resist the shear.

The bending moment on the channel may be taken equal to one-half that on the I beams, or $53,650 \div 2 = 26,820$ foot-pounds, as the load on the channel is practically one-half the load on the beam.

7. Dead Load.—As 4.5 pounds per board foot, the weight stated in *B. S.*, Art. **97**, is rather high for the timber used in bridge floors, we shall assume it to include the weight of the spikes that hold the floor down, and the bolts that hold the spiking pieces to the beams. As the floor plank is 3 inches thick, its weight is $3 \times 4.5 = 13.5$ pounds per square foot, or, since the beams are 3 feet apart, $3 \times 13.5 = 40.5$ pound per linear foot of beam. The floor plank will be fastened down by spiking it to wooden nailing pieces 2 to 6 inches thick that are bolted to the tops of the I beams (see *B. S.*, Art. **122**). In order to prevent water from standing on the floor, as it would do if the floor were level, it is customary to make the floor a little higher at the center of the span than at the ends; this is done by varying the depth of the nailing pieces. In the present case, the nailing pieces will be made 2 inches deep at the ends of the span and 4 inches deep at the center. As the top of the nailing piece will be curved, its average depth will be about $3\frac{1}{2}$ inches. Its width should be at least equal to the flange of the beam; it will be assumed that 6 inches is sufficient. The average cross-section is then $3\frac{1}{2}$ in. $\times$ 6 in., equivalent to $1\frac{3}{4}$ board feet; this makes the weight per linear foot $1.75 \times 4.5 = 7.9$ pounds. The total weight of timber supported by each beam is, then, very nearly $40.5 + 7.9 = 48.4$ pounds per linear foot. As the maximum bending moment due to the live load occurs 2.75 feet from the center, or 13 feet from the end of the span, it is necessary to find the dead-load bending moment at that point. For the floor plank and nailing pieces, it is

$$\frac{48.4 \times 31.5}{2} \times 13 - (48.4 \times 13) \times \tfrac{13}{2} = 5,820 \text{ foot-pounds}$$

for each beam, and approximately one-half of this, or 2,910 foot-pounds, for each channel.

8. There is still to be considered the bending moment due to the weight of the diaphragms represented in Fig. 2. The distance from the center of the I beam to the back of the channel is 3 feet 2⅛ inches; the diaphragm will be about 3 feet 1½ inches long, and, if 15-inch beams are used, about 12 inches deep. Then, as the weight of a 12″ × ⅜″ plate is 15.3 pounds per linear foot, the weight of 3 feet 1½ inches is 15.3 × 3.125 = 47.8 pounds. The total length of angle is 3 feet 1½ inches + 3 feet 1½ inches + 12 inches + 12 inches = 8 feet 3 inches, or 8.25 feet. As the weight of a 2½″ × 2½″ × ⅜″ angle is 5.9 pounds per linear foot, the total weight of angles is 5.9 × 8.25 = 48.7 pounds. In Fig. 2, it may be seen that there are thirty ⅝-inch rivets in one diaphragm, and the

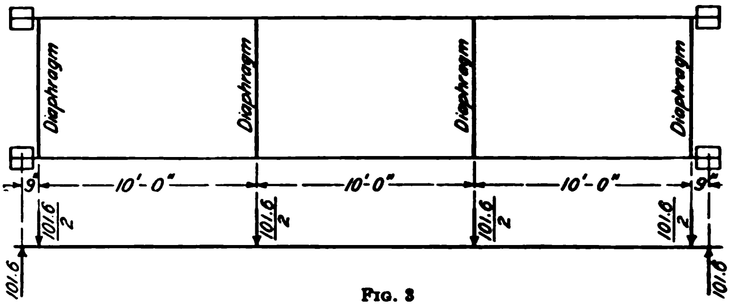


FIG. 3

weight of their heads must be found. Table XXI gives 8.5 pounds for the weight of one hundred rivet heads for ⅝-inch rivets; then, the weight of sixty rivet heads will be $\frac{60}{100} \times 8.5 = 5.10$ pounds. The weight of one diaphragm is, therefore, 47.8 + 48.7 + 5.1 = 101.6 pounds. Using four of these placed symmetrically on the span at distances of about 10 feet, as shown in Fig. 3, the bending moment on the I beam and on the channel due to them is, since half of each goes to one beam,

$$101.6 \times 10.75 - \frac{101.6}{2} \times 10 = 580 \text{ foot-pounds}$$

The bending moments due to the weight of the beams and channels cannot be found until their weights are known. It is well to assume some value, however, and, if necessary, correct it later. An experienced designer will come very close the first time. It has been shown that the depth cannot be less than 15 inches; we shall use the weight of the lightest 15-inch beam and channel, 42 and 33 pounds per linear foot, respectively, as given in Tables XIII and XIV. For the I beams, the bending moment at the point of maximum moment, 2.75 feet from the center, is

$$\frac{42 \times 31.5}{2} \times 13 - (42 \times 13) \times \tfrac{13}{2} = 5{,}050 \text{ foot-pounds}$$

And for the channel,

$$\frac{33 \times 31.5}{2} \times 13 - (33 \times 13) \times \tfrac{13}{2} = 3{,}970 \text{ foot-pounds}$$

The guard timber and fence will be found to weigh very nearly 45 pounds per linear foot, and, as this weight is almost all carried by the channel, the moment on the channel due to it is

$$\frac{45 \times 31.5}{2} \times 13 - 45 \times 13 \times \tfrac{13}{2} = 5{,}410 \text{ foot-pounds}$$

The total maximum bending moment on an I beam is, then, 53,650 + 5,820 + 580 + 5,050 = 65,100 foot-pounds, or 781,200 inch-pounds.

The total maximum bending moment on a channel is 26,820 + 2,910 + 580 + 3,970 + 5,410 = 39,690 foot-pounds, or 476,300 inch-pounds.

9. Allowable Working Stress.—The allowable intensity of stress given in *B. S.*, Art. **103**, for the compression flange is $20{,}000 - 200 \times \frac{l}{w}$. It is not considered good practice to assume that a wooden floor gives lateral support to the top flanges of the I beams; so, if no bracing is placed between the beams, the unsupported length will be $31.5 \times 12 = 378$ inches. Suppose that a 15-inch beam were used, then, as the flange is about 5.5 inches wide, the ratio $\frac{l}{w}$ would be $\frac{378}{5.5} = 68.7$, and the allowable intensity of stress would be $20{,}000 - 200 \times 68.7 = 6{,}260$ pounds per square inch. If, however, small struts are placed between the beams near the top flanges, in the same relative positions as the diaphragms already referred to, the unsu lengths may be taken as 10 feet, or 120 inches, allowable intensity of stress for the I beam will

$$20{,}000 - 200 \times \frac{120}{5.5} = 15{,}640 \text{ pounds}$$

In general, when the ratio $\frac{l}{w}$ exceeds 40, the top should be supported laterally.

As the bending moment is 781,200 inch-pour required value of the section modulus is 781,200 - = 49.95. Consulting Table XIV, it is found that the 15-inch beam—that is, a 15-inch 42-pound beam—ha tion modulus of 58.9. This beam, therefore, can be used.

10. Assuming that the lightest 15-inch channel for which the flange is 3.4 inches wide, the allowable i of stress is

$$20{,}000 - 200 \times \frac{120}{3.4} = 12{,}940 \text{ pounds per square}$$

Then, as the maximum bending moment is 476,3 pounds, the required value of the section mo 476,300 ÷ 12,940 = 36.81. Consulting Table XIII, it that a 15-inch 33-pound channel has a section modulus and so this will be used.

11. The bracing angles will add a little to the d on the I beams; but, as the section modulus of the 42-pound I beam is larger than required, it is not n to consider the effect of those angles.

NOTE.—In case the required value of the section modulus l out larger than that corresponding to the assumed size of t it would have been necessary to revise the design. Suppos had come out 86.0. Then, if a 15-inch beam were used, it necessary to use a 15-inch 70-pound beam, for which the secti lus is 88.5; the same strength could be had, however, by 18-inch 55-pound beam, for which the section modulus is 88.4 getting the same strength with a lighter beam. It would necessary to recompute the bending moment and allowable of stress for the 18-inch beam.

12. Depth of Floor.—The distance from the c line and from the top of the bridge seat to the to

floor can now be found. The vertical distances at the center of the span are 3 inches of plank, 4 inches of nailing strip, and 15 inches for the I beams and channels, making 22 inches for the depth of floor at the center, as represented in Fig. 1 (*b*). In Art. **3,** 2 inches was allowed for sole plates and bedplates under the beams; the bridge seat is, therefore, 24 inches below the floor at the center of the span, and, since the nailing pieces are 2 inches deep at the ends, the bridge seat is 22 inches below the floor at the ends. If the tops of the parapets *a*, *a*, Fig. 1 (*b*), are made level with the floor, they must be 22 inches high.

13. General Plan or Detail Drawings.—Fig. 4 is a detail drawing of the bridge that has just been designed, and gives all the information necessary for its manufacture. It is customary, in drawing the plan and elevation, to show a portion of the floor and fence in its finished condition, as at (*a*) and (*b*), and the remainder with the floor and fence removed, as at (*c*) and (*d*). One end of a channel and several beams are usually shown with the top flange cut away, in order to show the detail of the connection of the beams to the sole plates. The diaphragms, sometimes called **frames,** are marked F_1, and the cross-struts, which also are called frames, are marked F_2. Frames of both kinds are shown to a larger scale at (*f*) and (*g*).

The holes for the anchor bolts at one end are made circular and $\frac{1}{4}$ inch larger in diameter than the bolts; at the other end, they are made $1\frac{1}{4}$ inches wide and $1\frac{5}{8}$ inches long, allowing $\frac{3}{8}$ inch for expansion and contraction.

In a bridge of this size, the diaphragms or frames are sometimes bolted instead of riveted to the beams, to avoid the expense of setting up a riveting plant for so small a number of rivets.

DESIGN OF AN I-BEAM RAILROAD BRIDGE

14. Data.—An I-beam railroad bridge will now be designed from the data given on page 13.

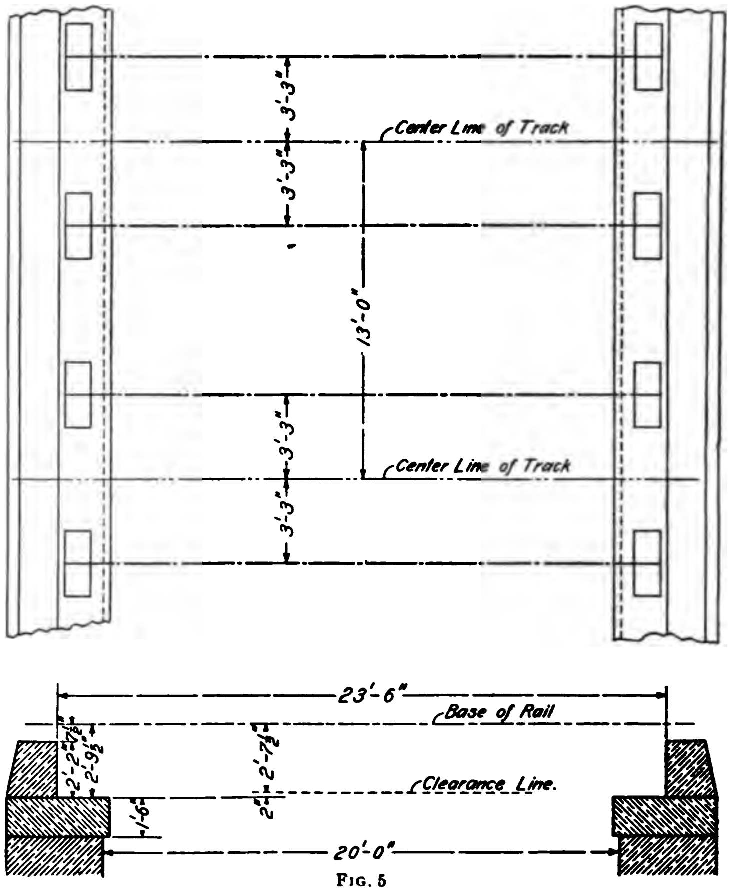


Fig. 5

15. Determination of Span.—The distance between neat lines under bridge seats is 20 feet. If the bearing

GENERAL DATA

For bridge over *French Creek*

at *Redlands, California*

Length and general dimensions *20 feet between neat lines under bridge seats*

Skew or angle of abutments with center line of bridge *90°*

Width of bridge and location of trusses *No trusses*

Floor system *Standard tie-floor on steel stringers or I beams*

Number and location of tracks *2 tracks 13 feet center to center*

Loading *Cooper's E50, as represented in Bridge Specifications, Art. 24*

Description of abutments *Cement concrete*

Distance from floor to clearance line *Not greater than distance to high water*

Distance from floor to high water *8 feet*

" " " " low water *13 feet*

" " " " river bottom *15 feet*

Character of river bottom *2 feet gravel, 3 feet shale, then solid rock 20 feet below base of rail*

Usual season for floods *April and May*

Name of nearest railroad station *Redlands, California*

Distance to nearest railroad station *5 miles*

Time limit *90 days*

Name of Engineer *Henry Jones*

Address *Berkeley, California*

Remarks

plates are made 12 inches wide, and set so that their edges are 6 inches back of the neat line, the total length of the beams will be 20 feet + 1 foot 6 inches + 1 foot 6 inches = 23 feet; and the distance center to center of bearings (the span), 1 foot less, or 22 feet. If 3 inches is left at each end, the parapets will be 23 feet 6 inches apart. The bridge seats will be made 1 foot 6 inches thick. The bridge-seat plan is represented in Fig. 5. The distance from the base of the rail to the top of the bridge seat cannot be found until after the bridge is designed. The top of the parapet is made $7\frac{1}{2}$ inches below the base of the rail.

16. Depth and Spacing of Beams.—The depth and the spacing of beams depend on the maximum bending moment and required value of the section modulus. It is considered better practice to place two beams under each rail than one; where necessary, three are used. The beams under each rail are bolted together and made to act as one by means of cast-iron separators or spacers, as illustrated in Table XV; these are located about 5 or 6 feet apart along the beams, but are not to be assumed as supporting the top flanges laterally. Lateral support is furnished by lateral bracing so arranged that the ratio of unsupported length to width of flange shall not exceed 12 (see *B. S.*, Art. **87**).

17. Live Load.—The live load is represented in Fig. 3 of *B. S.* By applying the conditions for maximum moment, it is found that it occurs when there are four driving axles on the span, under the second (or third) driver when that driver is 1.25 feet from the center of the span, and is equal to

$$\frac{4 \times 50{,}000 \times 9.75 \times 9.75}{22} - 50{,}000 \times 5 = 614{,}200 \text{ foot-pounds.}$$

18. Impact and Vibration.—The formula for impact and vibration is $I = \frac{300}{L + 300} \times S$ (*B. S.*, Art. **25**). In this case, as the moment is required, the value of the moment found in the last article should be substituted for S; and, as the entire span must be loaded in order to produce the maximum moment, $L = 22$. Therefore,

$$I = \frac{300}{22 + 300} \times 614{,}200 = 572{,}200 \text{ foot-pounds}$$

19. Dead Load.—The weight w per linear foot of I-beam bridges of this class is given very closely by the formula $w = 25\,l$ (*B. S.*, Art. **242**). In this case, $l = 22$, and, therefore, $w = 25 \times 22 = 550$ pounds per linear foot. The weight of track can be taken as 400 pounds per linear foot (*B. S.*, Art. **23**). The maximum live-load moment occurs very near the center of the span, and it will be sufficiently accurate to find the dead-load moment at the center, which is

$$\frac{(550 + 400) \times 22 \times 22}{8} = 57{,}500 \text{ foot-pounds}$$

20. Wind Load.—It is unnecessary to compute the stresses in the laterals of I-beam bridges. If the smallest-sized angle allowed is used—in this case, $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in.—it is sufficiently strong to resist any wind stresses. According to *B. S.*, Art. **27**, the wind pressure on the train is 300 pounds per linear foot, applied 7 feet above the top of the rail. Ordinarily, the lateral system will be about 2 feet below the top of the rail, or about 9 feet below the center of wind pressure. Then, as the beams will be placed 6 feet 6 inches center to center, the additional load on the leeward beams (as explained in *Stresses in Bridge Trusses*, Part 5) will be $\frac{300 \times 9}{6.5} = 415$ pounds per linear foot, and the bending moment at the center due to it will be

$$\frac{415 \times 22 \times 22}{8} = 25{,}100 \text{ foot-pounds}$$

As this is so small, it is sometimes neglected. In the present case, it is less than 2 per cent. of the total moment, but, there being no reason why it should not be considered, it will be taken into account.

21. Total Moment.—The total moment is equal to the sum of the several moments just found, and is as follows:

614,200 + 572,200 + 57,500 + 25,100 = 1,269,000 foot-pounds = 15,228,000 inch-pounds.

22. Section Modulus.—As it is required that the flanges shall be supported laterally, the full value of 16,000 pounds per square inch can be used for the allowable intensity of stress, since the ratio of width to unsupported length will be less than 20 (see *B. S.*, Art. **29**). Then, the required value of section modulus is 15,228,000 ÷ 16,000 = 951.75.

If four beams are used (two under each rail), the required value of the section modulus for each will be 951.75 ÷ 4 = 237.94. Consulting Table XIV, it is found that the heaviest I beam made has a section modulus of 198.4, which is not enough. Therefore, more beams must be used. If six beams are used (three under each rail), the required value of the section modulus for each will be 951.75 ÷ 6 = 158.62. Consulting Table XIV, it is found that a 20-inch 95-pound I beam has a section modulus of 160.7, and a 24-inch 80-pound I beam has a section modulus of 174. If it were necessary to keep the distance from the base of the rail to the underneath clearance line as small as possible, the 20-inch beams would be used. In the present case, as there are practically no restrictions, the 24-inch beams will be used, as they are lighter and therefore more economical. On account of their larger value of section modulus, the 24-inch beams are also stronger than the 20-inch beams.

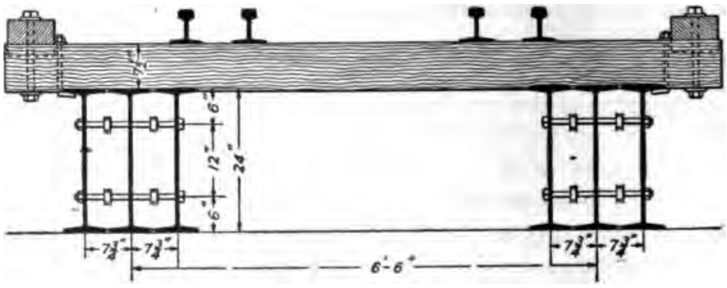
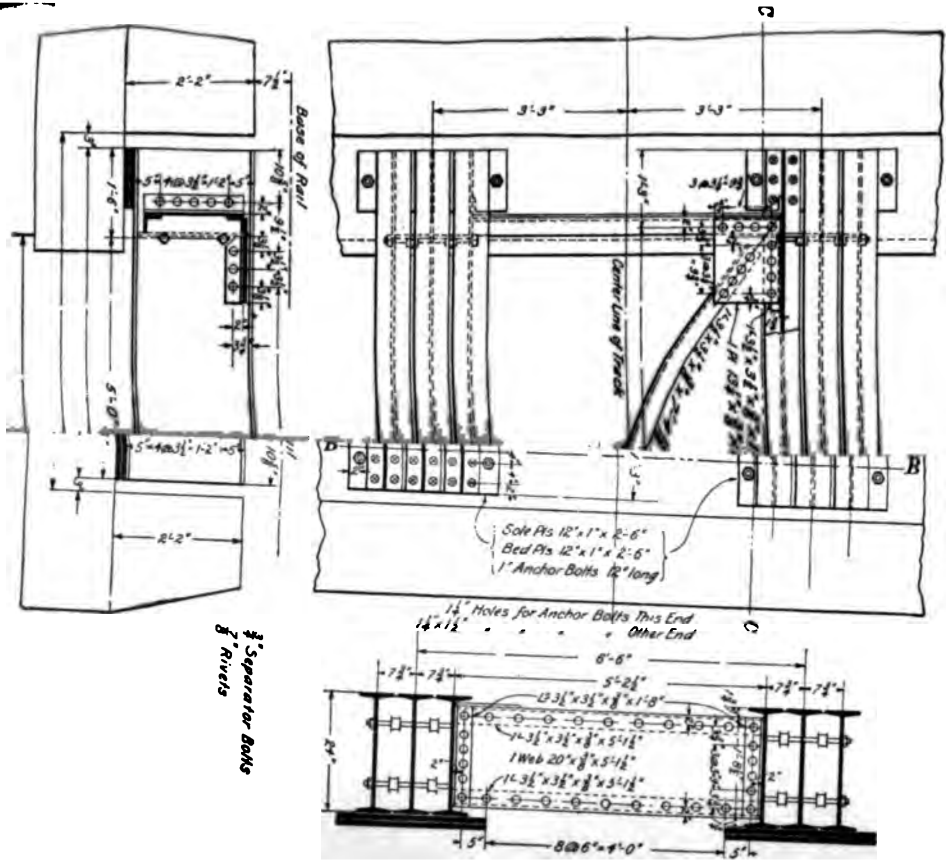


FIG. 6

23. Depth of Bridge.—In *B. S.*, Art. **48**, it is stated that standard ties are framed to 7½ inches in depth over



Cross Section on B-B

stringers and girders 6 feet 6 inches center to center. Then, the distance from the base of rail to the underneath clearance line, as represented in cross-section in Fig. 6, is 7½ inches + 24 inches = 2 feet 7½ inches. Allowing 1 inch for the sole plate and 1 inch for the bedplate gives 2 feet 9½ inches from the base of the rail to the top of the bridge seat, as represented in Fig. 5.

24. Plan.—Fig. 7 is a detail drawing of the bridge that has just been designed, and shows the customary method of arranging the lateral system. The lateral angles are connected to plates that are riveted to angles attached to the inside beams. The lateral truss is placed as high as is possible without interfering with the top flanges of the beams. The two sets of I beams are connected near the ends by diaphragms or frames, and at the panel points of the lateral truss by means of single angles. It is customary to show the top view of the I beams, and to consider a portion of the top flange removed at each lateral connection and at the end of one set of beams, in order to show more clearly the detail of the connection of the laterals to the beam and of the sole plates to the lower flanges.

For I-beam bridges longer than about 18 feet, three panels are used in the lateral systems; for shorter bridges, two panels are employed. The panels are usually made equal. In locating the rivets in the lateral connection plates, great care must be taken that the different angles connecting to the plate do not interfere with one another. It is well to lay out each connection plate on a large sheet full or half size and draw each angle in its proper position, leaving about ¼ inch clearance between the different angles that come close together.

The process of laying out the lateral system is frequently perplexing to a beginner, but is very simple after a little experience has been had. The different steps will be briefly outlined. The end frames are first located near the ends in such a position that they will not interfere with the anchor bolts nor with the rivets that connect the sole plates to the beams. This can be done by locating the frames about 1 inch from the sole plates. In Fig. 7, the backs of the angles that serve as flanges for the frames are 1 foot 1 inch from the ends of the I beams. These angles are $3\frac{1}{2}$ inches wide, and, according to Table XII, the gauge lines are 2 inches from the back edges. This makes the gauge lines of the end frames 1 foot 3 inches from the ends of the span and 23 feet – 1 foot 3 inches – 1 foot 3 inches = 20 feet 6 inches from each other. This distance is divided into three equal spaces of 6 feet 10 inches each, thus locating the gauge lines of the angles that act as struts between the beams at the intermediate points.

The three beams that form each side of the bridge are bolted together with their centers $7\frac{5}{8}$ inches apart, as given in Table XV, and each set of three is placed with its center 6 feet 6 inches from that of the other. This makes the inside beams 6 feet 6 inches – $7\frac{5}{8}$ inches – $7\frac{5}{8}$ inches = 5 feet $2\frac{3}{4}$ inches apart. Since the webs of these beams are $\frac{3}{4}$ inch in thickness (Table XIV), the clear distance between them is 5 feet 2 inches. The lug or hitch angles that are used to connect the plates to the webs of the inside I beams are $3\frac{1}{2}$ inches wide, with the gauge lines 2 inches from the webs. This makes these gauge lines 5 feet 2 inches – 2 inches – 2 inches = 4 feet 10 inches apart. The gauge lines of these hitch angles and those of the cross-frames or struts, located in the preceding paragraph, are commonly called **working lines** for the lateral trusses. They can be considered as the center lines of the chords and the panel points, respectively. The diagonal lines connecting the intersections of these gauge lines locate the diagonals of the lateral truss. In Fig. 7, they are taken as the gauge lines of the laterals. In some cases, the center of gravity of the angle is made to coincide with the diagonal between the working lines; in a bridge as small as that now under consideration, it matters little which method is used.

The next step is to find the length of the diagonal. Each of these lines, together with the working lines, forms a right triangle whose two legs are 6 feet 10 inches and 4 feet 10 inches, respectively. Then, the length of the diagonal is

$$\sqrt{(6'\,10'')^2 + (4'\,10'')^2} = 8 \text{ feet } 4\tfrac{7}{16} \text{ inches}$$

The rivets at the ends of the laterals are next located by laying out the connection plates full or half size, as previously described.

In detailing bridge work, it is frequently necessary to find the length of the hypotenuse of a right triangle when the legs, commonly called the **coordinates** in this work, are known. The principle for finding the hypotenuse is simple, but the arithmetical work is laborious, especially when the coordinates are given to small fractions, such as sixteenths of an inch. To shorten this work, tables are commonly employed. There are several books on the market that contain tables giving the squares of distances that occur in feet, inches, and fractions of an inch. In using such tables, the squares of the coordinates are copied and added; the length corresponding to the square root of the sum is then taken directly from the table.

DESIGN OF A PLATE-GIRDER RAILROAD BRIDGE

25. Data.—As the next example of practical design will be taken a deck plate-girder railroad bridge, the data sheet for the construction of which is given on page 20.

26. Determination of Span.—It is first necessary to determine the location of the abutments. It is required that the clear distance between them at the level of the curb be made 50 feet. There is one electric-car track in the center of the street, and it may be assumed that the top of the rail is level with the top of the curb. According to *B. S.*, Art. **94**, the lowest line of overhead bracing in through bridges for street railways shall not be lower than 15 feet from the top of the rail. This condition applies equally well to the underneath clearance of bridges over street-railroad tracks. In the present case, the underneath clearance line of the bridge will be placed 15 feet above the top

GENERAL DATA

For bridge over *Sumner Street*

at *Cincinnati, Ohio*

Length and general dimensions *One span over street 50 feet wide at the level of the curb, with one electric-car track at the center*

Skew or angle of abutments with center line of bridge *90°*

Width of bridge and location of trusses *No trusses. Space deck girders according to Bridge Specifications, Art. 16*

Floor system *Standard-tie floor. See Bridge Specifications, Art. 48*

Number and location of tracks *2 tracks 13 feet center to center*

Loading *Cooper's E50, as represented in Bridge Specifications, Art. 24*

Description of abutments *Granite abutments; front faces even with street lines*

Distance from floor to clearance line *Not more than 6 feet 6 inches*

Distance from floor to high water *No water*

" " " " low water " "

" " " " river bottom " "

Character of river bottom " "

Usual season for floods " "

Name of nearest railroad station *Cincinnati, Ohio*

Distance to nearest railroad station *3 miles*

Time limit *90 days*

Name of Engineer

Address of Engineer

Remarks *Above-described bridge is to replace the bridge at present in use. New abutments will be built by the Railroad Company. Contractor will furnish bridge-seat plan within 10 days from award of contract* of the rail, and the top of the bridge seat will be made level with that line. Bridge seats for this length of span should be not less than 18 inches thick; using this thickness, the distance from the top of the rail to the neat line under the bridge seat will be 15 feet − 1 foot 6 inches, or 13 feet 6 inches. If the face of each abutment is battered $\frac{1}{2}$ inch per foot, the abutments will be $13\frac{1}{2}$ inches farther apart under the bridge seat than at the top of the curb, or, in this case, 50 feet + 1 foot $1\frac{1}{2}$ inches = 51 feet $1\frac{1}{2}$ inches. Bedplates for deck plate girders are usually made about 20 inches long for spans of 50 feet, and about 24 inches long for spans 75 feet long, with the front edge from 6 to 9 inches behind the neat line. For the span under consideration, the plates will be made 22 inches in length and set with the front edges $6\frac{1}{4}$ inches behind the neat line. The center of the bedplate is then 11 inches + $6\frac{1}{4}$ inches = $17\frac{1}{4}$ inches, behind at each end of the span; the span is 51 feet $1\frac{1}{2}$ inches + 1 foot $5\frac{1}{4}$ inches + 1 foot $5\frac{1}{4}$ inches = 54 feet; and the total length of the girder is 22 inches more than this, or 55 feet 10 inches. For a girder of this length, the parapets should not come nearer than 4 inches to the ends. In this case, they will be made 56 feet 6 inches apart.

27. Depth and Spacing of Girders.—Deck girders are usually made about one-ninth or one-tenth of the span in depth. When the distance from the base of the rail to the clearance line is specified, the depth of the girder must be chosen so as not to exceed it. The depth of the tie and the thickness of the flange plates will usually occupy about 1 foot of the depth. In the present case, as the specified distance is 6 feet 6 inches, the depth of the girder, or the width of the web, should not exceed 5 feet 6 inches. If the depth of the girder is made one-tenth of the span, it will be $54 \div 10 = 5.4$ feet. It is well, when possible, to use an even foot or half foot for the width of web; in this case, 5.5 feet, or 66 inches, will be used. The backs of the flange angles are usually placed $\frac{1}{4}$ inch farther apart, or, in this case, $66\frac{1}{4}$ inches. This allows for slight irregularities in the edges of the web-plates, and prevents the edges from inter fering with the flange plates. As the span is less than 70 feet, the girders will be 6 feet 6 inches center to center (see *B. S.*, Art. **16**).

The bridge-seat plan is represented in Fig. 8. The distance from the base of the rail to the clearance line or the

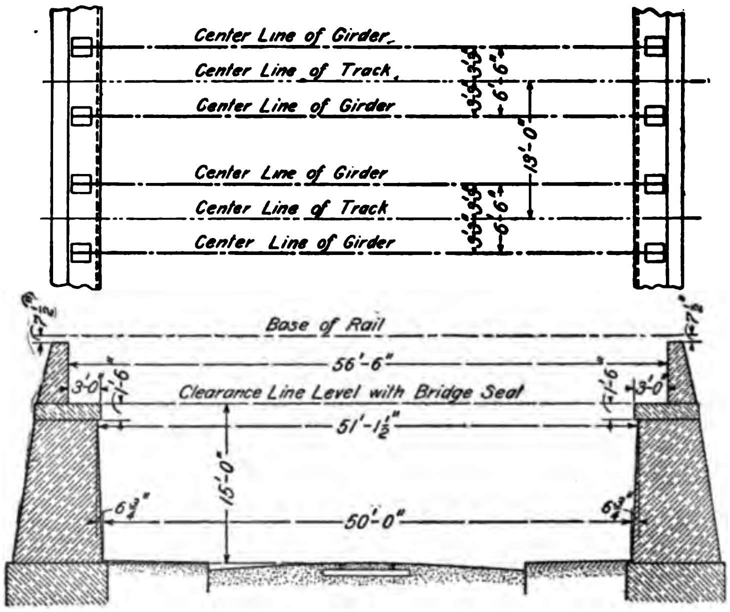


FIG. 8

bridge seat is not given, as this distance cannot be determined until after the girders have been designed.

28. Dead Load.—The formula for the weight per linear foot of a deck plate-girder railroad bridge for the specifications referred to in the data sheet, and for Cooper's E50 loading, is given in Art. **242**, *B. S.*, as

$$w = 500 + 8\,l = 500 + 8 \times 54 = 930 \text{ pounds, nearly}$$

The weight of the track will be taken as 400 pounds, making the total dead-weight 930 + 400 = 1,330 pounds per linear foot for one track (two girders). As explained in *Design of Plate Girders*, Part 1, it is necessary to calculate the moments and shears at several points along the girder. In the present case, they will be calculated at the center and at distances of 5, 10, 15, and 20 feet from the end of the span; besides, the shear at the end must be computed.

The dead-load moments, in foot-pounds, are as follows:

At the center,

$$\frac{1{,}330 \times 54 \times 54}{8} = 484{,}900$$

At 20 feet from the end,

$$\frac{1{,}330 \times 54}{2} \times 20 - \frac{1{,}330 \times 20 \times 20}{2} = 452{,}200$$

At 15 feet from the end,

$$\frac{1{,}330 \times 54}{2} \times 15 - \frac{1{,}330 \times 15 \times 15}{2} = 389{,}000$$

At 10 feet from the end,

$$\frac{1{,}330 \times 54}{2} \times 10 - \frac{1{,}330 \times 10 \times 10}{2} = 292{,}600$$

At 5 feet from the end,

$$\frac{1{,}330 \times 54}{2} \times 5 - \frac{1{,}330 \times 5 \times 5}{2} = 162{,}900$$

The dead-load shears, in pounds, are as follows:

At the end, $$\frac{1{,}330 \times 54}{2} = 35{,}910$$

At 5 feet from the end,

$$35{,}910 - 5 \times 1{,}330 = 29{,}260$$

At 10 feet from the end,

$$35{,}910 - 10 \times 1{,}330 = 22{,}610$$

At 15 feet from the end,

$$35{,}910 - 15 \times 1{,}330 = 15{,}960$$

At 20 feet from the end,

$$35{,}910 - 20 \times 1{,}330 = 9{,}310$$

At the center,

$$35{,}910 - 27 \times 1{,}330 = 0$$

29. Live Load.—The live load consists of Cooper's E50, represented in Fig. 3 of *B. S.* By applying the principles explained in *Stresses in Bridge Trusses*, Part 4, it is found that the greatest moment occurs under the third driver of the second engine, when that driver is .127 foot to the left of the center. The value of this moment is 2,703,200 foot-pounds. The maximum moment at the center, in this case, occurs when the same driver is at the center, and is 2,702,500 foot-pounds. As will be seen, these two values are very nearly equal. In general, it may be stated that *for spans over 50 feet it is sufficiently accurate to use in design the greatest moment that can occur at the center of the span, neglecting the consideration that involves the location of the center of gravity of the loads.* The maximum live-load moments and shears are then as follows:

At the center (under third driver, second engine), the live-load moment, in foot-pounds, is 2,702,500.

At 20 feet from the end (under third driver, first engine), 2,563,000.

At 15 feet from the end (under second driver, first engine), 2,247,200.

At 10 feet from the end (under second driver, second engine), 1,703,700.

At 5 feet from the end (under first driver, second engine), 976,900.

At the end (first driver, second engine), the live-load shear, in pounds, is 228,900.

At 5 feet from the end (first driver, second engine), 195,400.

At 10 feet from the end (first driver, first engine), 163,100.

At 15 feet from the end (first driver, first engine), 130,900.

At 20 feet from the end (first driver, first engine), 101,600.

At the center (first driver, first engine), 65,200.

30. Impact and Vibration.—The formula for impact and vibration is $I = \frac{300}{L + 300} \times S$ (*B. S.*, Art. **25**). In the present case, the allowance in terms of the moments and shears must be found. For the moments, it is necessary to load the entire span; then, $L = 54$, and

$$I = \tfrac{300}{354} \times M = .84746\, M$$

The allowances are:

Location	Moment, in Foot-Pounds
Center	$.84746 \times 2{,}702{,}500 = 2{,}290{,}300$
20 feet from the end	$.84746 \times 2{,}563{,}000 = 2{,}172{,}000$
15 feet from the end	$.84746 \times 2{,}247{,}200 = 1{,}904{,}400$
10 feet from the end	$.84746 \times 1{,}703{,}700 = 1{,}443{,}800$
5 feet from the end	$.84746 \times 976{,}900 = 827{,}900$

As the length of track that must be loaded to produce the greatest shears is different for different sections, the allowance for impact and vibration will be a different proportion of the maximum shear in each case. It has been found that the maximum shear at each section occurs when the first driver is at the section, for which position the first wheel is 8 feet beyond the section. The length of loaded track for sections within 8 feet of the end of the span is, therefore, 54 feet; for other sections it is equal to the distance of the section from the other end of the span plus 8 feet. The proportional allowances are as follows:

At the end and 5 feet from the end, the length L of loaded portion is 54; the proportional allowance, $I = .84746\ S$.

At 10 feet from the end, $L = 52$; $I = \frac{300}{352} S = .8523\ S$

At 15 feet from the end, $L = 47$; $I = \frac{300}{347} S = .8646\ S$

At 20 feet from the end, $L = 42$; $I = \frac{300}{342} S = .8772\ S$

At the center, $L = 35$; $I = \frac{300}{335} S = .8955\ S$

The allowances for shear are, therefore, as follows:

Location	Shear, in Pounds
End	$.84746 \times 228{,}900 = 194{,}000$
5 feet from the end	$.84746 \times 195{,}400 = 165{,}600$
10 feet from the end	$.8523 \times 163{,}100 = 139{,}000$
15 feet from the end	$.8646 \times 130{,}900 = 113{,}200$
20 feet from the end	$.8772 \times 101{,}600 = 89{,}100$
Center	$.8955 \times 65{,}200 = 58{,}400$

31. Wind Pressure.—In this article, we shall consider only the increase in moments and shears caused in the leeward girder by the overturning effect of the wind; this increase is calculated by the formula $w = \frac{Ph}{b}$, in which P is the wind pressure per linear foot of train, and w is the vertical load per linear foot of girder, due to the wind pressure (see *Stresses in Bridge Trusses*, Part 5). The center of the wind pressure is 7 feet above the top of the rail (*B. S.*, Art. 27), and the top lateral bracing is generally about 2 feet below the top of the rail; then, h, the distance from the center of wind pressure to the top lateral bracing, is $7 + 2 = 9$ feet; and b is 6.5 feet. Therefore,

$$w = \frac{300 \times 9}{6.5} = 415 \text{ pounds per linear foot}$$

The moments due to a uniform load of 1,330 pounds per linear foot have already been found; those due to a uniform load of 415 pounds may be found by multiplying the former moments by $\frac{415}{1330}$, or .31203. The results are as follows:

LOCATION	MOMENT, IN FOOT-POUNDS
Center	$.31203 \times 484{,}900 = 151{,}300$
20 feet from the end	$.31203 \times 452{,}200 = 141{,}100$
15 feet from the end	$.31203 \times 389{,}000 = 121{,}400$
10 feet from the end	$.31203 \times 292{,}600 = 91{,}300$
5 feet from the end	$.31203 \times 162{,}900 = 50{,}800$

As the wind pressure under consideration is that on a moving train, the shears will be found as for a moving load, that is, by loading the portion of the span on one side of a section. They are as follows:

LOCATION	SHEAR, IN POUNDS
End	$\frac{415 \times 54}{2} = 11{,}200$
5 feet from the end	$\frac{415 \times 49 \times 49}{2 \times 54} = 9{,}200$
10 feet from the end	$\frac{415 \times 44 \times 44}{2 \times 54} = 7{,}400$
15 feet from the end	$\frac{415 \times 39 \times 39}{2 \times 54} = 5{,}800$
20 feet from the end	$\frac{415 \times 34 \times 34}{2 \times 54} = 4{,}400$
Center	$\frac{415 \times 27 \times 27}{2 \times 54} = 2{,}800$

32. Total Moments and Shears.—As the dead load, live load, impact and vibration, and wind pressure may act simultaneously, the total moments and shears may be found by adding the values that have been found for the different conditions. In doing so, however, it must be remembered that the moments and shears due to dead load, live load, and impact and vibration have been found for the load on the entire width of track, that is, on two girders, while those due to the wind pressure are the effects on one girder. Therefore, to find the total moment or shear at any section of one girder, that due to the wind pressure may be added to one-half the sum of those due to dead load, live load, and impact and vibration, as just found. The total moments and shears *on each girder* can now be found.

The total moments, in foot-pounds, at various positions on the girder are as follows:

At the center,

$$\frac{484{,}900 + 2{,}702{,}500 + 2{,}290{,}300}{2} + 151{,}300 = 2{,}890{,}200$$

At 20 feet from the end,

$$\frac{452{,}200 + 2{,}563{,}000 + 2{,}172{,}000}{2} + 141{,}100 = 2{,}734{,}700$$

At 15 feet from the end,

$$\frac{389{,}000 + 2{,}247{,}200 + 1{,}904{,}400}{2} + 121{,}400 = 2{,}391{,}700$$

At 10 feet from the end,

$$\frac{292{,}600 + 1{,}703{,}700 + 1{,}443{,}800}{2} + 91{,}300 = 1{,}811{,}400$$

At 5 feet from the end,

$$\frac{162{,}900 + 976{,}900 + 827{,}900}{2} + 50{,}800 = 1{,}034{,}700$$

The total shears, in pounds, at the different sections of the girder are:

At the end,

$$\frac{35{,}910 + 228{,}900 + 194{,}000}{2} + 11{,}200 = 240{,}600$$

At 5 feet from the end,

$$\frac{29{,}260 + 195{,}400 + 165{,}600}{2} + 9{,}200 = 204{,}300$$

At 10 feet from the end,

$$\frac{22{,}610 + 163{,}100 + 139{,}000}{2} + 7{,}400 = 169{,}800$$

At 15 feet from the end,

$$\frac{15{,}960 + 130{,}900 + 113{,}200}{2} + 5{,}800 = 135{,}800$$

At 20 feet from the end,

$$\frac{9{,}310 + 101{,}600 + 89{,}100}{2} + 4{,}400 = 104{,}400$$

At the center,

$$\frac{0 + 65{,}200 + 58{,}400}{2} + 2{,}800 = 64{,}600$$

33. Design of Web.—A $\frac{7}{16}$-inch web will be tried first. The gross section is $66 \times \frac{7}{16} = 28.875$ square inches. As the total shear at the end is 240,600 pounds, the intensity of the shearing stress is $240{,}600 \div 28.875 = 8{,}330$ pounds per square inch. Consulting Table XXXVI, finding the point on the curve for the $\frac{7}{16}$-inch web corresponding to an intensity of stress of 8,330 pounds per square inch, and looking horizontally to the right, it is found that the stiffeners must be spaced about 16 inches apart. According to *B. S.*, Art. **55**, the spacing of stiffeners should be not less than one-third the depth of the web. In this case, therefore, the stiffeners should be not less than 22 inches apart. As the spacing given in Table XXXVI is 16 inches, it is necessary to try a thicker web. A web $\frac{1}{2}$ inch thick will be tried next. The gross section is $66 \times \frac{1}{2} = 33$ square inches, and the intensity of shearing stress, $240{,}600 \div 33 = 7{,}290$ pounds per square inch. Consulting Table XXXVI, finding the point on the curve for the $\frac{1}{2}$-inch web corresponding to an intensity of stress of 7,290 pounds per square inch, and looking horizontally to the right, it is found that the stiffeners must be spaced 22 inches apart. As this thickness of web satisfies the conditions as far as stiffener spacing at the end is concerned, the required spacing of stiffeners at other sections will next be found. The intensities of shearing stress are:

Location	Intensity of Shearing Stress, in Pounds per Square Inch
End	240,600 ÷ 33 = 7,290
5 feet from the end	204,300 ÷ 33 = 6,190
10 feet from the end	169,800 ÷ 33 = 5,150
15 feet from the end	135,800 ÷ 33 = 4,120
20 feet from the end	104,400 ÷ 33 = 3,160
Center of span	64,600 ÷ 33 = 1,960

34. Spacing of Stiffeners.—Consulting Table XXXVI, finding the points on the curve for $\frac{1}{2}$-inch web corresponding to the intensities just found, and looking horizontally to the right or left (whichever is nearer), the required spacings of stiffeners are found to be as follows:

Location	Spacing of Stiffeners, in Inches
End	22
5 feet from the end	27
10 feet from the end	32
15 feet from the end	38
20 feet from the end	46
Center of span	62

The stiffener spacing at other points on the girder may be found by interpolating between the values just found. These distances will not give the actual distances between the stiffeners, but simply the distances that must not be exceeded at the various sections. The actual distances between stiffeners depend on other details, such as rivet spacing, and are usually found by the detailer or draftsman when the plans are being made.

35. Pitch of Flange Rivets.—The required spacing of the rivets that connect the flanges to the web at any section is found by the following formula, given in *Design of Plate Girders*, Part 1:

$$p = \frac{K h_r}{V}$$

In the present case, the rivets are in double shear and in bearing on the $\frac{1}{2}$-inch web-plate. (They are also in bearing on the two flange angles, but it may be assumed that the thickness of the latter is greater than that of the web; it is found in practice that this is invariably true. For this reason, the bearing on the flange angles need not be considered.) The rivets used in deck plate-girder railroad bridges are $\frac{7}{8}$ inch in diameter for all spans. Those in the flanges are always shop-driven rivets and, according to *B. S.*, Art. **29**, the values in Table XL will be used. Consulting Table XL, the value of one $\frac{7}{8}$-inch rivet in double shear is found to be 13,230 pounds, and in bearing on a plate $\frac{1}{2}$ inch thick, 9,630 pounds. The latter value, being the smaller, must be used in the formula. As the flanges have not yet been designed, it is not known what size of angles will be used, so that the value of h_r cannot be calculated. In actual practice, however, the designer soon learns what sizes of angles are generally used for spans of different lengths. In the type of bridge now under consideration, 6″ × 6″ angles are used for all spans up to about 70 feet, and 8 in. × 8 in. for longer spans. In the present case, 6″ × 6″ angles will be used. Consulting Table XII, it is found that there will be two rows of rivets in each leg, the line midway between the two rows being $2\frac{1}{4} + \frac{2\frac{1}{4}}{2} = 3\frac{3}{8}$ inches from the back of the angle, as represented in Fig. 9. The distance between the backs of the angles in the flanges has already been found to be $66\frac{1}{4}$ inches. The distance h_r is, therefore, $66\frac{1}{4} - 3\frac{3}{8} - 3\frac{3}{8} = 59\frac{1}{2}$ inches. By substituting the proper values in the formula, and using the intensities of

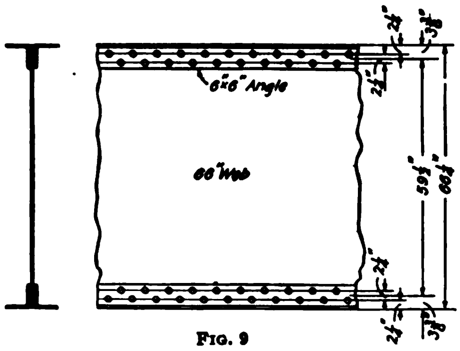


FIG. 9

shearing stress found in Art. **33,** the following pitches are obtained:

LOCATION	RIVET PITCH, IN INCHES Bottom Flange	Top Flange
End	$\frac{9{,}630 \times 59.5}{240{,}600} = 2.38$;	$2.38 \times .9 = 2.14$
5 feet from the end	$\frac{9{,}630 \times 59.5}{204{,}300} = 2.80$;	$2.80 \times .9 = 2.52$
10 feet from the end	$\frac{9{,}630 \times 59.5}{169{,}800} = 3.37$;	$3.37 \times .9 = 3.03$
15 feet from the end	$\frac{9{,}630 \times 59.5}{135{,}800} = 4.22$;	$4.22 \times .9 = 3.80$
20 feet from the end	$\frac{9{,}630 \times 59.5}{104{,}400} = 5.49$;	$5.49 \times .9 = 4.94$
Center	$\frac{9{,}630 \times 59.5}{64{,}600} = 8.87$;	$8.87 \times .9 = 7.98$

As a deck railroad bridge is under consideration, the pitch in the bottom flange, found from the formula $p = \frac{K h_r}{V}$, must be multiplied by .9 to get the required pitch in the top flange (see *B. S.*, Art. **57**). The pitch is sometimes made the same in both flanges, that found for the top flange in the manner just explained being used for the bottom as well. As in the case of stiffener spacing, the foregoing values will not represent the actual spacing of rivets at any section, but simply the values that must not be exceeded at the different sections. The spacing at other points may be found by interpolating between the given values. As the pitch at 20 feet from the end came out greater than that allowed in *B. S.*, Art. **57,** there was no necessity for computing the pitch at sections nearer the center.

36. Design of Flanges.—The required area of cross-section of the flanges at any section is found by means of the formula $A = \frac{M}{s h_g} - \frac{t h}{8}$ (*Design of Plate Girders*, Part 1). It is impossible to calculate the value of h_g, as the areas of the flanges are not yet known; for a first trial, however, h_g will be assumed equal to the depth h of web, the flanges at the center of the span will be designed on this basis, using the bending moment at the center of the span, and the distance h_g between the centers of gravity of the trial flanges will be computed. With this corrected value for h_g, the areas of the flanges will again be found, and the necessary correction made. It will seldom be found necessary to change the cross-section of the flanges as found by assuming $h_g = h$. In the present case, $h = 66$ inches, $s = 16{,}000$ pounds, and $t = \frac{1}{2}$ inch. To apply the formula, the bending moment already found must be multiplied by 12, to reduce it to inch-pounds. For the first trial cross-section at the center of the span, we have, therefore,

$$\frac{2{,}890{,}200 \times 12}{66 \times 16{,}000} - \frac{1}{8} \times \frac{1}{2} \times 66 = 32.84 - 4.12$$
$$= 28.72 \text{ square inches}$$

This is the value for the gross area of the top flange and the net area of the bottom flange.

37. The actual choice of the sizes of angles and plates is wholly a matter of practice. The designer usually follows certain established rules and relies to a great extent on his experience. It is considered bad practice to use very small or thin angles and a large number of plates. It is also considered bad practice to make the entire flange section of angles; this is not economical, as the entire section of flange must be continued the whole length of the girder. In *B. S.*, Art. **59**, it is required that one-third to one-half the flange area shall be composed of angles. In the present case, that would require from $\frac{28.72}{3}$, or 9.57, to $\frac{28.72}{2}$, or 14.36, square inches in two angles, or, as there are two angles, 4.79 to 7.18 square inches in each angle. 6″ × 6″ angles with thicknesses from $\frac{1}{2}$ to $\frac{11}{16}$ inch are commonly used in the flanges of plate girders for railroad bridges up to about 70 feet in length; the flange plates are never narrower than the total width of the two flange angles together with the thickness of the web, nor thicker than the flange angles. In the present case, the following sections will be used:

Top Flange	Section, in Square Inches
Two angles 6 in. × 6 in. × $\frac{1}{2}$ in. @ 5.75 .	11.5
One plate 14 in. × $\frac{3}{8}$ in.	5.25
One plate 14 in. × $\frac{7}{16}$ in.	6.125
One plate 14 in. × $\frac{7}{16}$ in.	6.125
Total gross area	29.0

In *B. S.*, Art. **60**, it is required that, when plates of different thicknesses are used, they shall diminish in thickness outwards from the flange angles. An exception is sometimes made to this rule, and will be made in this case, when it is required that one plate shall extend the full length of the top flange. This is done to keep water and dirt from working down between the flange angles and the web, and to give the girder a better finish. A thin plate serves the purpose just as well as a thicker one, and is more economical, as all that part beyond its theoretical end is wasted, so far as necessary flange section is concerned.

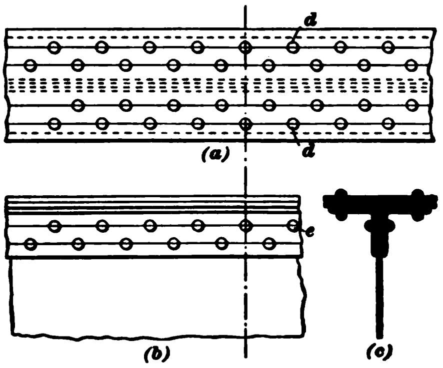


Fig. 10

For the bottom flange it is necessary to deduct from the gross section the areas of cross-section of the rivet holes. This is most easily done by deducting from each angle and plate the holes that are in the plate. Fig. 10 shows the method of riveting flange angles to the web and the flange plates to the angles. Each rivet *d* in the horizontal leg of any angle is directly opposite one *e* in the vertical leg; this brings two rivets *d*, *d* directly opposite each other in each plate. There are then two holes to be deducted from each angle and two from each plate. The following sections will be used:

Bottom Flange	Section, in Square Inches
Two angles 6 in. × 6 in. × ½ in.;	11.50 − 2 × 2 × .50 = 9.5 0
One plate 16 in. × ½ in.	8.0 − 2 × .5 = 7.0 0
One plate 16 in. × 7/16 in.	7.0 − 2 × .4375 = 6.1 2 5
One plate 16 in. × 7/16 in.	7.0 − 2 × .4375 = 6.1 2 5
	Total net area, 2 8.7 5

38. In the foregoing design, h_s was assumed. The location of the center of gravity of each flange and the distance between the two centers of gravity will now be computed, and the design of the flanges altered if necessary. For the center of gravity of the lower flange, the statical moment for each angle will be taken equal to the area of net section multiplied by the lever arm of the gross section, as the position of the center of gravity is practically the same for both sections. The position of the center of gravity of the gross section is taken from Table IX. Moments will be taken about the outer edge of the section in each case; the lever arms for the top flange are shown in Fig. 11, those for the bottom flange in Fig. 12.

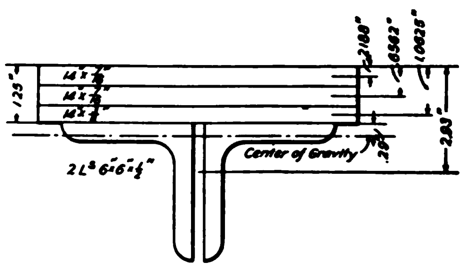


Fig. 11

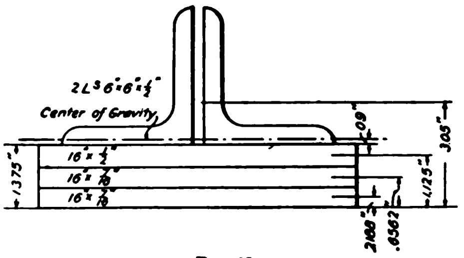


Fig. 12

The statical moments for the top flange are as follows:

6.1 2 5	× .2188	=	1.3 4 0
6.1 2 5	× .6562	=	4.0 1 9
5.2 5	× 1.0625	=	5.5 7 8
1 1.5	× 2.93	=	3 3.6 9 5
2 9.0			4 4.6 3 2

The distance of the center of gravity of the top flange is, therefore, $44.632 \div 29 = 1.54$ inches from the outside of the section. As the plates have a total thickness of $\frac{7}{16} + \frac{7}{16} + \frac{3}{8} = 1.25$ inches, the center of gravity of the top flange is $1.54 - 1.25 = .29$ inch below the back of the angles.

The statical moments for the bottom flange are as follows:

$$\begin{array}{lll} 6.125 \times & .2188 = & 1.340 \\ 6.125 \times & .6562 = & 4.019 \\ 7.0 & \times 1.125 = & 7.875 \\ \underline{9.50} & \times 3.05 = & \underline{28.975} \\ 28.75 & & 42.209 \end{array}$$

The distance of the center of gravity of this flange from the outside edge of the flange is, therefore, $42.209 \div 28.75 = 1.468$ inches from the outside of the section. As the plates have a total thickness of $\frac{1}{2} + \frac{7}{16} + \frac{7}{16} = 1.375$ inches, the center of gravity of the bottom flange is $1.468 - 1.375 = .09$ inch from the back of the angles.

As the distance back to back of the flange angles is 66.25 inches, the distance h_r between the centers of gravity of flanges is $66.25 - .29 - .09 = 65.87$ inches. This is very nearly equal to the assumed distance of 66 inches. Substituting this value of h_r in the formula for area, the result is

$$A = \frac{2{,}890{,}200 \times 12}{65.87 \times 16{,}000} - \frac{1}{8} \times \frac{1}{2} \times 66 = 32.91 - 4.12$$
$$= 28.79 \text{ square inches}$$

This is slightly greater than the net area of the trial bottom flange, but the difference is so slight (.04 square inch) that it is inadvisable to make any changes. Had the difference been greater—say, .2 or .3 square inch—it might have been advisable to increase the thickness of one of the flange plates by $\frac{1}{16}$ inch.

It will be seen that the flange plates are wider and thicker in the lower flange than in the upper. They are sometimes made the same width, in which case the lower flange plates must be still thicker, or else more plates must be used. If those in the lower flange are made about 2 inches wider than those in the top, and the same number of plates is used, it will usually be found that the theoretical lengths of corresponding plates in the two flanges are the same or very nearly so; this condition is very convenient in designing, especially if the same rivet spacing is used in both flanges.

Some engineers do not design the top flange, but make it the same size as the lower flange. This gives additional section, and therefore additional strength to the top flange, but is not economical. If this were done in the present case, as the gross area of the lower flange is 11.50 + 8 + 7 + 7 = 33.5 square inches, and the required gross area of the upper flange is 28.79 square inches, the difference, which is 4.71 square inches, would be wasted in the upper flange. Unless it is stated in the specifications that both flanges must have the same gross area, they should be designed separately.

39. Lengths of Flange Plates.—The flange angles in both flanges and the plate next to the flange angles in the top flange are continued the full length of the girder. The other plates are cut off where they are no longer needed. For this purpose, the areas required at the different sections at which the moments have been computed will be determined, and the curves of flange areas will be plotted. The distances between the centers of gravity of the flanges at sections other than at the center are not known, but they may be assumed for trial equal to that at the center, and corrected later. Using the bending moments found in Art. 32, the required flange areas, in square inches, are:

At the center,

$$\frac{2{,}890{,}200 \times 12}{65.87 \times 16{,}000} - 4.12 = 32.91 - 4.12 = 28.79$$

At 20 feet from the end,

$$\frac{2{,}734{,}700 \times 12}{65.87 \times 16{,}000} - 4.12 = 31.15 - 4.12 = 27.03$$

At 15 feet from the end,

$$\frac{2{,}391{,}700 \times 12}{65.87 \times 16{,}000} - 4.12 = 27.24 - 4.12 = 23.12$$

At 10 feet from the end,

$$\frac{1{,}811{,}400 \times 12}{65.87 \times 16{,}000} - 4.12 = 20.63 - 4.12 = 16.51$$

At 5 feet from the end,

$$\frac{1{,}034{,}700 \times 12}{65.87 \times 16{,}000} - 4.12 = 11.79 - 4.12 = 7.67$$

Fig. 13 shows the curves of flange areas: AB represents the half span; C, D, E, and F, the sections at 5, 10, 15, and 20 feet from A, respectively; CC', DD', EE', FF', and BB',

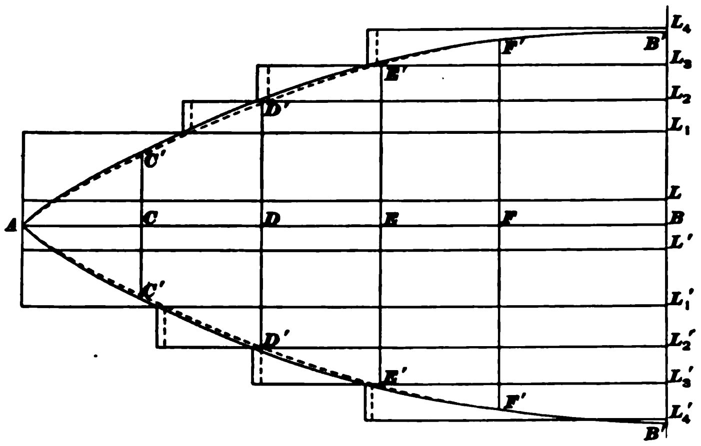


FIG. 13

the required areas of the flanges at C, D, E, F, and B, respectively, BL and BL' representing $\frac{th}{8}$, or the portion of web that is included in flange area; LL_1, L_1L_2, L_2L_3, and L_3L_4, the areas of the angles and plates in the upper flange; and $L'L_1'$, $L_1'L_2'$, $L_2'L_3'$, and $L_3'L_4'$, the areas of the angles and plates in the lower flange. Dotted curves are then drawn through A, C', D', E', F', and B'. Drawing lines through L, L_1, L_2, etc., and noting where they intersect the curves, the points at which the plates are no longer required are determined. At C, no plates are required; at D, one plate on each flange is required; at E, two plates on each flange are required; and at F and B, three plates on each flange are required. At F and B there is no need to revise the flange area, as the entire section is required. In some cases, as at E, the end of a plate is just included in the section, but, as the plate at that point does not carry much stress, it should not be counted as part of the flange area in calculating the distance between the centers of gravity of the flanges.

The actual location of the center of gravity at each section can be calculated in the same way as in Art. **38** for the entire flange; it is not necessary to repeat the numerical steps. They are, for the top flange, .64 inch at E, 1.09 inches at D, 1.68 inches at C, below the backs of the flange angles; and for the bottom flange, .43 inch at E, .86 inch at D, and 1.68 inches at C, above the backs of the flange angles. The distances between the centers of gravity of the flanges are:

At E, 66.25 − .64 − .43 = 65.18 inches
At D, 66.25 − 1.09 − .86 = 64.30 inches
At C, 66.25 − 1.68 − 1.68 = 62.89 inches

The revised flange areas, in square inches, are, therefore, as follows:

At 15 feet from the end,

$$\frac{2{,}391{,}700 \times 12}{65.18 \times 16{,}000} - 4.12 = 27.52 - 4.12 = 23.40$$

At 10 feet from the end,

$$\frac{1{,}811{,}400 \times 12}{64.30 \times 16{,}000} - 4.12 = 21.13 - 4.12 = 17.01$$

At 5 feet from the end,

$$\frac{1{,}034{,}700 \times 12}{62.89 \times 16{,}000} - 4.12 = 12.34 - 4.12 = 8.22$$

The curve of flange areas may now be corrected by plotting these values at C, D, and E, drawing the curves shown in full lines, and locating the theoretical ends of the flange plates. The distance scaled from the theoretical end of a plate to the center line is one-half the length of plate; in this case, the theoretical lengths of plates in the top flange are approximately 25, 34.5, and 40.5 feet, and in the bottom flange, 25.5, 34.75, and 43 feet. According to *B. S.*, Art. **60**, each plate shall extend 12 inches at each end beyond the theoretical end; this will increase the length of each plate by 2 feet. The first plate in the top flange and all the angles will continue the full length of the girder, that is, 55 feet 10 inches. The flanges are then made up as follows:

TOP FLANGE

Two angles 6 in. × 6 in. × $\frac{1}{2}$ in. × 55 ft. 10 in
One plate 14 in. × $\frac{3}{8}$ in. × 55 ft. 10 in.
One plate 14 in. × $\frac{7}{16}$ in. × 36 ft. 6 in.
One plate 14 in. × $\frac{7}{16}$ in. × 27 ft.

BOTTOM FLANGE

Two angles 6 in. × 6 in. × $\frac{1}{2}$ in. × 55 ft. 10 in.
One plate 16 in. × $\frac{1}{2}$ in. × 45 ft.
One plate 16 in. × $\frac{7}{16}$ in. × 36 ft. 9 in.
One plate 16 in. × $\frac{7}{16}$ in. × 27 ft. 6 in.

The actual lengths of the plates will probably be slightly different from the lengths given, the difference being due to rivet spacing in the flanges.

40. Splices.—As no plate or angle is longer than 70 feet, it is unnecessary to splice any flange member (*B. S.*, Art. **61**). Consulting Table V, it is found that the longest plate 66 in. × $\frac{1}{2}$ in. that it is possible to get is 34 feet long; it is therefore necessary to splice the web. It will be spliced at the center, making each half 27 feet 11 inches long, nearly. The size of the splice plates will first be determined. Consulting Table XI, it is found that for a 6″ × 6″ × $\frac{3}{8}$″ angle the nominal and actual widths are equal; then, the actual size of the leg of a 6″ × 6″ × $\frac{1}{2}$″ angle is $6\frac{1}{8}$ inches. As the distance back to back of the flange angles is $66\frac{1}{4}$ inches, the clear distance between the vertical legs is $66\frac{1}{4} - 6\frac{1}{8} - 6\frac{1}{8} = 54$ inches. Allowing $\frac{1}{8}$ inch clearance at top and bottom leaves $53\frac{3}{4}$ inches as the height of the splice plate. According to *B. S.*, Art. **56**, each splice plate shall have a sectional area equal to 75 per cent. that of the web. In the present case, that of the web is 33 square inches; then, the area of each plate must be $.75 \times 33 = 24.75$ square inches. As the plates are

53¾ inches deep, the required thickness is 24.75 ÷ 53.75 = .46 inch. The nearest standard thickness is ½ inch; then, each splice plate will be 53¾ in. × ½ in. A spacing of rivets will now be assumed; if it gives sufficient resistance, it will be used; if not, the number of rivets will be increased. The spacing shown in Fig. 14 will be assumed.

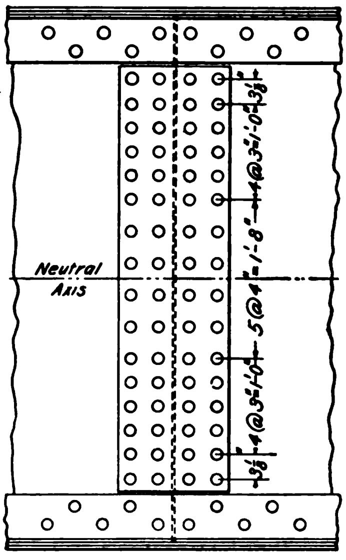


FIG. 14

The rivets are in double shear and in bearing on a ½-inch web-plate. It has already been found (Art. 35) that the latter gives the smaller value for a ⅞-inch rivet; this value is 9,630 pounds. As the distances of the rivets from the neutral axis are 2, 6, 10, 13, 16, 19, 22, and 25⅛ inches, and as there are four rivets at each of these distances, the moment of resistance of the rivets is, according to the formula given in *Design of Plate Girders*, Part 1,

$$\frac{9{,}630 \times 4 \times (2^2 + 6^2 + 10^2 + 13^2 + 16^2 + 19^2 + 22^2 + 25.125^2)}{25.125}$$

$$= 3{,}130{,}000 \text{ inch-pounds}$$

The moment that the web can bear is given by the formula $M = \frac{s t h^2}{8}$ (*Design of Plate Girders*, Part 1). In the present case, the value of s for bending stress is 16,000 pounds per square inch, t = ½ inch, and h = 66 inches; then,

$$M = \frac{16{,}000 \times .5 \times 66^2}{8} = 4{,}356{,}000 \text{ inch-pounds}$$

Since this is greater than the resisting moment that has been found, it is necessary to use more rivets. As the rivets farthest from the neutral axis are the most effective, we shall begin another row outside of the first two rows, and first compute the moment of resistance of a few rivets near the flanges. Let us first try two rivets at the top and two at the bottom, at distances of 25.125 and 22 inches from the neutral axis. The moment of resistance of these four rivets is

$$2 \times \frac{9,630 \times (22^2 + 25.125^2)}{25.125} = 855,000 \text{ inch-pounds,}$$

which, added to that already found, gives 3,130,000 + 855,000 = 3,985,000 inch-pounds. This is still too small. Let us try one more rivet at the top and one at the bottom in the same row, each 19 inches from the neutral axis. The moment of resistance of these two is

$$2 \times \frac{9,630 \times 19^2}{25.125} = 277,000 \text{ inch-pounds,}$$

which, added to that already found, gives 3,985,000 + 277,000 = 4,262,000 inch-pounds. This is still too small. Let us try one more rivet at the top and one more at the bottom in the same row, each 16 inches from the neutral axis. The moment of resistance of these two is

$$2 \times \frac{9,630 \times 16^2}{25.125}$$

= 196,000 inch-pounds, which, added to that already found, gives 4,262,000 + 196,000 = 4,458,000 inch-pounds. This is a little larger than the moment of the web, and is, therefore, sufficient. Three splice plates on each side of the web will now be used instead of one, the top and bottom plates having three rows of rivets on each side of the splice and the middle plate two rows. It is necessary to rearrange the spacing of the rivets, so that there will be the proper distance from the edge of each plate to

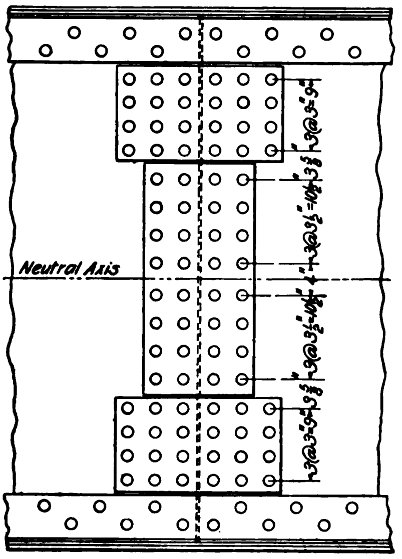


FIG. 15

the nearest rivet, and $\frac{1}{8}$ inch clearance between the plates (see *B. S.*, Art. **41**). The spacing represented in Fig. 15 will be used; this changes the location of the rivets, and it is well to recompute the moment of resistance of the rivets in the entire splice. That moment is as follows: $M = 9{,}630 \times [4 \times (2^2 + 5.5^2 + 9^2 + 12.5^2) + 6 \times (16.125^2 + 19.125^2 + 22.125^2 + 25.125^2)] \div 25.125 = 4{,}433{,}000$ inch-pounds, which is larger than the moment of resistance of the web, and therefore sufficient. It is unnecessary to calculate the moment of resistance of the splice plates; if the plates on each side have an area on a vertical section 75 per cent. that of the web, their moment of resistance will be greater than that of the web.

41. Bearings.—Since the abutments are granite, for which, according to *B. S.*, Art. **29**, the allowable intensity of pressure is 500 pounds per square inch, and the end shear, which is equal to the reaction, is 240,600 pounds, the required area of bearing is 240,600 ÷ 500 = 481.2 square inches. In Art. **26**, it was stated that the bedplates would be made 22 inches long; the required width is, therefore, 481.2 ÷ 22 = 21.9 inches. They will be made 22 inches long.

42. End Stiffeners.—The formula given in *Design of Plate Girders*, Part 1, for the required thickness of end stiffeners is $t' = \frac{R}{n s_b (b - \frac{1}{2})}$. In the present case, $R = 240{,}600$ pounds, and, according to *B. S.*, Art. **29**, the allowable intensity of bearing s_b is 18,000 pounds per square inch. The outstanding legs of the flange angles are 6 inches wide; then, according to *B. S.*, Art. **55**, the stiffeners will be 5 in. × 3½ in. It will first be assumed that there are four stiffeners; this gives

$$t' = \frac{240{,}600}{4 \times 18{,}000 \times (5 - \frac{1}{2})} = .743 \text{ inch}$$

When the required thickness of stiffeners comes out greater than $\frac{5}{8}$ inch, as in this case, it is generally considered advisable to use more stiffeners. Using eight gives

$$t' = \frac{240{,}600}{8 \times 18{,}000 \times (5 - \frac{1}{2})} = .371 \text{ inch, say, } \tfrac{3}{8} \text{ inch}$$

As this thickness is less than $\frac{3}{8}$ inch, it will be adopted, and eight stiffeners 5 in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{3}{8}$ in. with reinforcing plates under them will be used. The stiffeners will be riveted to the girder with $\frac{7}{8}$-inch rivets. The value in single shear, 6,610 pounds (Table XL), is found to be the smallest value. Then, the number of rivets required to connect each stiffener is

$$\frac{240,600}{8 \times 6,601} = 4.55\text{, or, say, 5 rivets}$$

43. Lateral System.—The lateral truss represented in Fig. 16 will be used for both the upper and the lower flange; the end frames will be about 52 feet apart, which makes it possible to use eight panels at 6 feet 6 inches each. The wind load is given in *B. S.*, Art. 27. The pressure on the lower half of the girder, about 3 feet in depth, is resisted by the lower laterals; the pressure of 50 pounds per square foot, or, in this case, $3 \times 50 = 150$ pounds per linear foot, evidently causes the greatest stresses in the lower lateral truss.

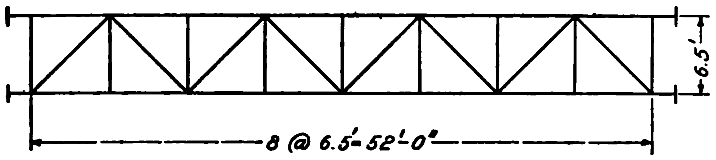


FIG. 16

Then, the panel load for the lower lateral truss is $150 \times 6.5 = 975$ pounds. The pressure on the upper half of the girder, and on the rails and ties, say 4 feet in depth, together with the pressure of 300 pounds per linear foot on the train, are resisted by the upper lateral system. The pressure on the train—together with 30 pounds per square foot, or $4 \times 30 = 120$ pounds per linear foot on the girders, ties, and rails—evidently causes greater stresses than 50 pounds per square foot on the girders, ties, and rails alone. The live wind panel load for the upper lateral system is, therefore, $300 \times 6.5 = 1,950$ pounds, and the dead wind panel load, $120 \times 6.5 = 780$ pounds.

44. The end panel of the upper lateral truss will first be considered. The live-load shear is $\frac{1{,}950 \times 7}{2} = 6{,}825$ pounds, and the dead-load shear is $\frac{780 \times 7}{2} = 2{,}730$ pounds. The total shear is, therefore, $6{,}825 + 2{,}730 = 9{,}555$ pounds. The panel length is the same as the distance center to center of girders; so the inclination of the laterals is about 45°; csc 45° = 1.414. The direct stress in the diagonal is $9{,}555 \times 1.414 = 13{,}510$ pounds, tension when the wind blows in one direction, and compression when in the other direction. According to *B. S.*, Art. **34**, the member must be designed for $13{,}510 + .8 \times 13{,}510 = 24{,}320$ pounds tension and compression. Dividing by 16,000 gives $24{,}320 \div 16{,}000 = 1.52$ square inches net section required to resist the tension.

45. According to *B. S.*, Art. **86**, the smallest angle that can be used for lateral bracing is $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in. In Table IX the gross area of this angle is given as 2.48 square inches. The number of holes to be deducted depends on the method of riveting the angle to the connection plate. Fig. 17 shows a connection frequently used: the short angle *a* riveted to the main angle at the end is called a **lug angle**, and serves the purpose of transmitting the stress from the leg *b* of the main angle to the connection plates. In practice, the number of rivets connecting the two angles is usually made one more than half the number required to connect the lateral to the connection plate. With rivets spaced as shown, 1.5 holes must be deducted, according to *B. S.*, Art. **33**. As the angle is $\frac{3}{8}$ inch thick, the area of cross-section of one hole is .375, and of 1.5 holes,

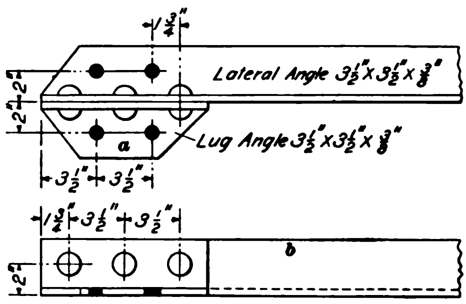


FIG. 17

$1.5 \times .375 = .5625$ square inch. Deducting this from 2.48 leaves 1.92 square inches net section. As only 1.52 is required, this angle is large enough so far as tension is concerned.

46. The distance center to center of girders, measured along a diagonal, is $6.5 \times 1.414 = 9.19$ feet = 110 inches, nearly. The ends of the laterals are riveted to the connection plates, so that the unsupported length may be taken as the distance between connections, or about 18 inches at each end shorter than the distance between girders, leaving $110 - 2 \times 18 = 74$ inches unsupported. Table IX gives the least radius of gyration as .69 inch; then,

$$\frac{l}{r} = \frac{74}{.69} = 107.25$$

Table XXXV gives the allowable intensity of compressive stress as 9,760 pounds; as the gross area is 2.48 square inches, the strength of the angle is $2.48 \times 9{,}760 = 24{,}200$ pounds. This is very nearly equal to 24,320, the required strength, and so this angle is sufficiently large.

47. As the span under consideration is less than 75 feet long, it will be shipped riveted up complete. That is, the rivets connecting the laterals to the lateral plates will be shop-driven, and, according to *B. S.*, Art. **29**, the values given in Table XL will be used. Table XL gives the value of a $\frac{7}{8}$-inch rivet in single shear as 6,610 pounds; the required number of rivets is then $24{,}320 \div 6{,}610 = 3.7$, or, say, 4 rivets. As the $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle is strong enough in the end panel of the upper lateral system, it is sufficient in all other panels, and so there is no need in this case of making any computations for the other angles.

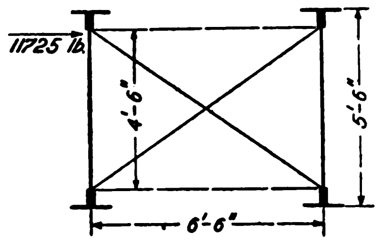


FIG. 18

48. The amount of wind pressure that is transmitted to the abutments by the end frames (a diagram of which is shown in Fig. 18) can be found

Number and location of tracks *One street-railway track at center of roadway*

Loading *As given in B. S., Art. 98 (2) and (5)*

Description of abutments *Cement-concrete abutments*

Distance from floor to clearance line *Not more than 6 feet*

" " " " high water *10 feet*

" " " " low water *20 feet*

" " " " river bottom *25 feet*

Character of river bottom *5 feet of mud and silt, then solid rock*

Usual season for floods *March and April*

Name of nearest railroad station *Scranton, Pennsylvania*

Distance to nearest railroad station *3 miles*

Time limit *6 months from award of contract*

Name of Engineer *International Textbook Company*

Address of Engineer *Scranton, Pennsylvania*

Remarks *Bridge to have ornamental railing 3 feet 9 inches high at each side of bridge. The top of bridge seat must be kept above high-water line*

2. Plans.—The plans for some parts of this bridge are shown in *Bridge Drawing*. The student is advised to consult those plates frequently during the study of this Section.

3. Kind of Bridge.—The first step is to determine the

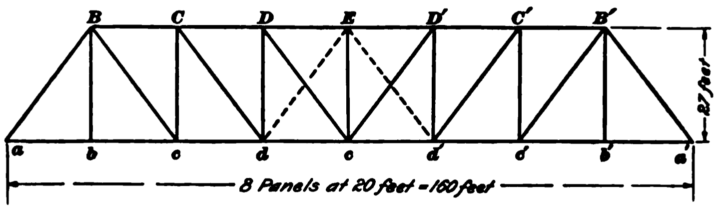


FIG. 1

kind of bridge. As the distance from the floor to the clearance line is limited to 6 feet, it is evident that a through

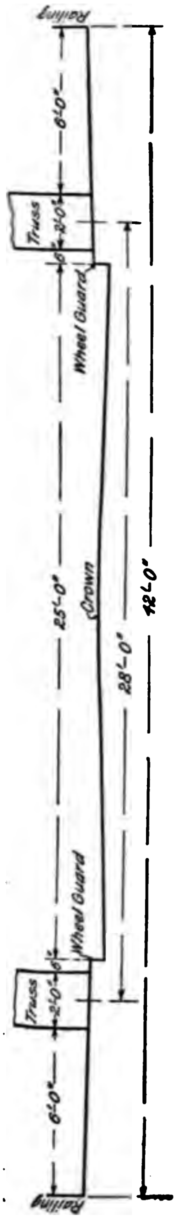


FIG. 2

FIG. 3

bridge must be used, there not being sufficient room for a deck bridge. As the span is 160 feet, pin-connected trusses must be used (*B. S.*, Art. **91**). According to *B. S.*, Art. **229**, panels from 20 to 30 feet in length are best for pin-connected trusses: eight panels 20 feet long will be used. If a simple Pratt truss is chosen, with a depth of 27 feet, which is about one-sixth of the span, the angle between the inclined web members and the lower chord will be about 53° 30′. This angle is greater than 50°, as required in *B. S.*, Art. **92**, and hence a depth of 27 feet will be adopted. The outline of the truss is shown in Fig. 1.

4. Width of Bridge. The data require that the trusses shall be located between the roadway and the sidewalks, and that the roadway shall have a clear width of 25 feet between wheel-guards. According to *B. S.*, Art. **125**, the edges of the wheel-guards toward the roadway will be 6 inches from the clearance lines of the trusses, making the clear distance between the latter 26 feet. The width occupied by a truss depends on the width of the widest members; for pin-connected trusses it is seldom less than 18 inches. In the present case, the width will be assumed to be 24 inches. If the design works out more or less than this, it may be slightly changed. With a width of 2 feet for each truss, the distance center to center of trusses is 28 feet, and the distance between the clearance lines of the trusses toward the sidewalks is 30 feet. As the data require 2 sidewalks 6 feet wide, the distance between the railings will be $30 + 6 + 6 = 42$ feet. Fig. 2 shows the assumed width and location of trusses and the location of railings.

DESIGN OF FLOOR SYSTEM

STRINGERS

5. Cross-Section of Floor.—The cross-section of floor usually employed for this type of bridge is shown in Fig. 3. The rails *h*, *h* for the street-railway track rest on $6'' \times 6''$ ties 8 feet long, marked *g*. When the rails are the same height as the thickness of the floor plank, the lower layer of plank is spiked directly to the top of the ties. When, as assumed in the figure, the height of the rail is greater than the thickness of the floor plank, longitudinal nailing pieces *f*, as required in *B. S.*, Art. **121**, are spiked to the top of the ties, and the lower layer of plank is spiked to the nailing pieces. For the remainder of the roadway, the lower layer of plank *d* is fastened to spiking pieces that are bolted to the tops of the steel beams. The upper layer *c* is then spiked to the top of the lower layer.

At each side of the roadway, $6'' \times 6''$ wheel-guards *e* are bolted to the top of the lower layer of plank; and the sidewalk plank *b*, for which one layer is sufficient, is continued right through the truss out over the wheel-guard. The sidewalk plank is supported at other points by steel beams with spiking pieces on top. To prevent the ends of the sidewalk plank *b* and the edges of the wheel-guards *e* from being worn by the wheels, a guard angle *a*, usually $2\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. $\times$ $\frac{1}{4}$ in., is fastened by screws to the ends of the sidewalk plank, as shown in Fig. 4.

It is customary, in order to facilitate drainage, to make the sides of the roadway from 3 to 6 inches lower than the center; they will be made 3 inches lower than the center in the present case. The top of the guard angle is required by *B. S.*, Art. **125**, to be 6 inches above the floor; this brings it 3 inches above the top of the floor at the center. The sidewalk plank rises from the wheel-guard toward the railing at the rate of $\frac{1}{4}$ inch per foot.

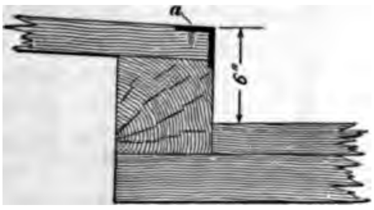


FIG. 4

6. Spacing of Stringers.—In *B. S.*, Art. **93**, it is specified that, for bridges carrying a street railway only, stringers shall be spaced 6 feet 6 inches center to center, and that for other bridges they shall be arranged to accommodate the traffic. In the present case, I beams will be placed 3 feet 3 inches on each side of the center line of bridge under the ties, and at intervals of 3 feet for the remainder of the floor, as shown in Fig. 5. An I beam *h* will also be placed at the center to carry any loads that move along the center of the car track. The beam *a, a*, at the outside of the sidewalk, under the railing, is frequently made deeper than the other sidewalk beams, and is composed of a web and one flange angle at top and bottom; it is then called a **fascia girder**. The actual location of the fascia girder depends on the type of fence railing that is used, and on the connection of the posts that support the fence. In the present case, the web of the fascia girder will be assumed to be 3 inches outside of the inner edge of the fence, making the web 3 feet from the next beam. If necessary, this distance can be changed slightly, when detailing, to conform to other details.

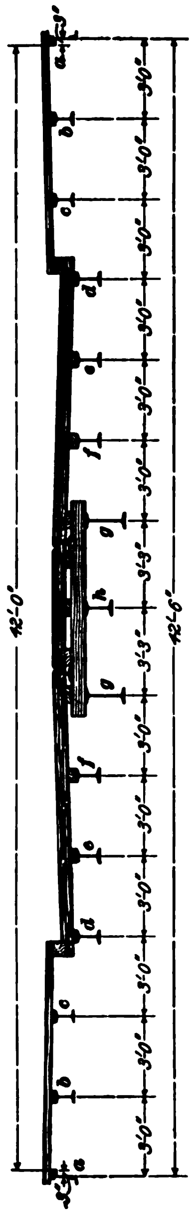


Fig. 5

7. In order to decrease the unsupported length of the upper flange of the stringers, so that a higher working stress may be used, cross-struts, similar to those placed between the I beams in the highway bridge designed in *Design of Plate Girders*, Part 2, will be placed between the stringers at the center of the panels. The unsupported length will then be 10 feet, or 120 inches. The weight of these struts is so small that it will be neglected in calculating moments and shears.

8. Design of Sidewalk Stringers.—In *B. S.*, Art. **98** (2), the live load is given as either 100 pounds per square foot or a road roller weighing 15 tons. The latter need not be considered on the sidewalk. Since the I beams are 3 feet apart, the live load due to the uniform load is 300 pounds per linear foot on beam *b*, Fig. 5. The floor plank 2 inches thick weighs $2 \times 4.5 = 9$ pounds per square foot (see *B. S.*, Art. **97**), and the part supported by beam *b* is $3 \times 9 = 27$ pounds per linear foot. The spiking piece will be assumed to weigh 8 pounds per linear foot, and the I beam 20 pounds per linear foot. The total load supported by the beam is, therefore, $300 + 27 + 8 + 20 = 355$ pounds per linear foot, and the maximum bending moment, since the floorbeams are 20 feet center to center, is $\frac{355 \times 20 \times 20}{8} \times 12$

$= 213{,}000$ inch-pounds

If a 9-inch 21-pound I beam is used, the width of the top flange is, by Table XIV, 4.33 inches; and, since the unsupported length is 120 inches, the allowable working stress, according to *B. S.*, Art. **103**, is

$$20{,}000 - 200 \times \frac{120}{4.33} = 14{,}460 \text{ pounds per square inch}$$

The required value of section modulus is 213,000 ÷ 14,460 = 14.73. As the section modulus of a 9-inch 21-pound I beam is greater than this (18.9, Table XIV), this beam will be used. An 8-inch 20.5-pound beam could have been used, but, as there is only .5 pound per foot difference between the two, and as the 9-inch beam is a standard beam, it is better to use the latter.

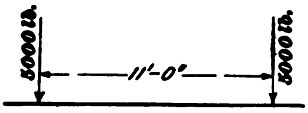


FIG. 6

The live load supported by beam *c* is somewhat less than that supported by *b*, as part of the width between beams *c* and *d* is occupied by the width of the truss. It is not customary to make allowance for this in the design of stringers, however, so that *c* will be made the same size as *b*.

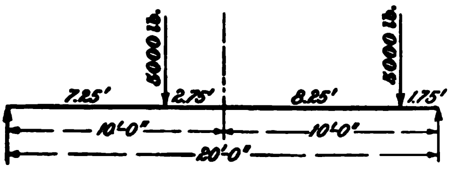


FIG. 7

It is unnecessary to design the fascia girder. The thinnest allowable web, $\frac{5}{16}$ inch (*B. S.*, Art. **112**), and the smallest allowable angles, $2\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × $\frac{5}{16}$ in. (*B. S.*, Art. **113**), will usually be found to give sufficient strength. These sections will be used in the present case.

9. Design of Roadway Stringers.—The road roller causes a greater bending moment on the roadway stringers than the uniform load. The live load on a stringer due to the road roller, according to *B. S.*, Art. **110** and Art. **98** (2), is represented in Fig. 6. The maximum bending moment on the beams *e* and *f*, Fig. 5, is found to occur under one of the loads when that load is 2.75 feet from the center of the panel, as shown in Fig. 7, and is 315,400 inch-pounds. Since the I beams *d*, *e*, and *f*, Fig. 5, are 3 feet apart, the dead load on *e* and *f* due to the floor plank is $3 \times 5 \times 4.5 = 67.5$ pounds per linear foot. The nailing piece will be assumed to weigh 10 pounds, and the I beam 30 pounds, giving a total dead load of 107.5 pounds per linear foot. The dead-load moment at the section of maximum live-load moment, 2.75 feet from the center, is 59,600 inch-pounds, making a total bending moment of $315,400 + 59,600 = 375,000$ inch-pounds. If a 10-inch 30-pound I beam is used, the width of the top flange is, by Table XIV, 4.8 inches, and the allowable intensity of working stress, since the unsupported length of flange is one-half a panel length, or 120 inches (Art. 7), is

$$20,000 - 200 \times \frac{120}{4.8} = 15,000 \text{ pounds per square inch}$$

The required section modulus is $375,000 \div 15,000 = 25$. As the section modulus of a 10-inch 30-pound I beam is greater than this (26.8, Table XIV), this beam will be used.

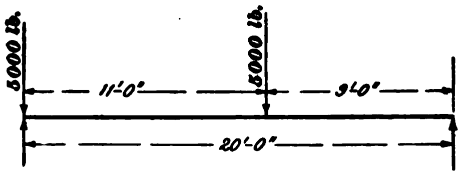


Fig. 8

The live load supported by the beam *d*, Fig. 5, is somewhat less than the live load supported by the beams *e* and *f*, but it is customary to make it the same size as the other roadway beams, and so a 10-inch 30-pound I beam will here be used for each of the beams *d*, *e*, and *f*.

10. The maximum reaction at the end of a stringer is found to occur, for the loading represented in Fig. 6, when the load occupies the position represented in Fig. 8. The live-load reaction is 7,250 pounds, and, since the dead load is 107.5 pounds per linear foot, the dead-load reaction is 1,075 pounds, giving a total reaction of 8,325 pounds. The connection to the floorbeam will be designed in *Design of a Highway Truss Bridge*, Part 2.

11. Design of Stringers Under Railway Track. The weight of the street car is given in *B. S.*, Art. **98** (4). One-half of it will be assumed to go to each rail; the rails will be taken 5 feet center to center. As the load on each axle is 20,000 pounds, the load on each wheel will be 10,000 pounds. The two wheels on each axle are located as shown in Fig. 9, with respect to the I beams *g* and *h*;

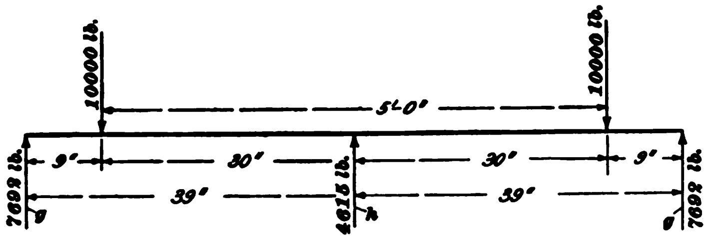


FIG. 9

$\frac{30}{39} \times 10{,}000$, or 7,692 pounds, goes to each of the beams marked *g*; and $2 \times \frac{9}{39} \times 10{,}000$, or 4,615 pounds, goes to the beam *h* at the center. The maximum bending moment on a stringer occurs when there are two wheels in a panel, one being 1.25 feet from the center and the other 3.75 feet on the other side of the center. The loading for beam *g* is shown in Fig. 10, the maximum moment being 706,700 inch-pounds, at 1.25 feet from the center. According to *B. S.*, Art. **99**, there must be added to this, to provide for impact and vibration, $\frac{3}{10} \times 706{,}700 = 212{,}000$ inch-pounds, giving for the total live-load moment, including the allowance for impact and vibration, 918,700 inch-pounds.

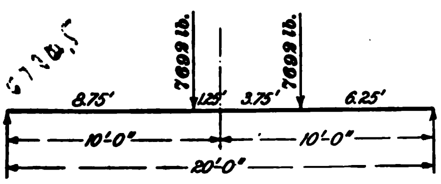


FIG. 10

The load on the leeward stringer is increased on account of the wind pressure on the side of the car. The wind pressure per linear foot, as given in *B. S.*, Art. **100**, is 250 pounds, applied 6 feet above the top of the rail. The length of the car will be assumed as 40 feet; the total pressure, therefore, is $40 \times 250 = 10{,}000$ pounds, and the overturning moment, according to *B. S.*, Art. **100**, is 10,000 × 6 = 60,000 foot-pounds. Since the rails are 5 feet center to center, the increase in the load on the leeward rail is 60,000 ÷ 5 = 12,000 pounds, one-quarter of which, 3,000 pounds, goes to each of the four wheels on the leeward side, and $\frac{17}{22}$ × 3,000 = 2,318 pounds goes to the stringer marked *g*. The bending moment due to this increase is found to be 212,000 inch-pounds, which makes the total bending moment 1.25 feet from the center, exclusive of dead load, 1,130,700 inch-pounds.

12. The dead-weight of the ties, nailing pieces, rails, and floor, including one-half the width of floor between the beams *g* and *i* (see Fig. 5), will be assumed to be equally distributed among the three beams *g*, *h*, and *g*, as shown to larger scale than before in Fig. 11. The ties, according to

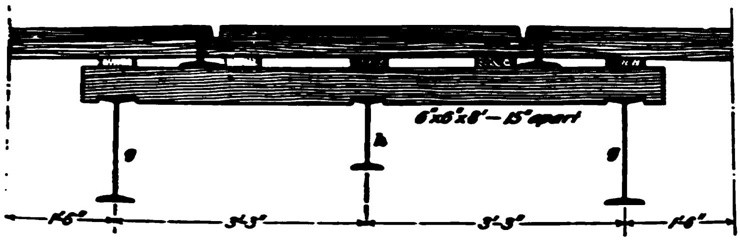


Fig. 11

B. S., Art. **121**, will be 6 in. × 6 in. × 8 ft., spaced 15 inches center to center. The weight of each tie is 108 pounds, corresponding to 86.4 pounds per linear foot of track (since the ties are spaced 1.25 feet center to center). The combined weight of the five spiking pieces will be assumed to be 25 pounds per linear foot, and of the two rails, 40 pounds per linear foot. Since the floor is 5 inches thick, and a width of 9 feet 6 inches is supported by the three beams, the weight of this portion is 5 × 4.5 × 9.5 = 213.75 pounds per linear foot, which makes the total dead-weight supported by the three beams, not including their own weight,

$$86.4 + 25 + 40 + 213.75 = 365.15 \text{ pounds per linear foot,}$$

and that supported by each beam one-third of this, or, say, 125 pounds per linear foot.

13. The weight of the beam *g* will be assumed to be 55 pounds per linear foot, making the total dead load on this beam $125 + 55 = 180$ pounds per linear foot. The dead-load moment at the section where the live-load moment is greatest, that is, at 1.25 feet from the center of the panel, is 106,300 inch-pounds. The total bending moment is, therefore, $1,130,700 + 106,300 = 1,237,000$ inch-pounds. If an 18-inch 55-pound beam is used, as the width of the flange (Table XIV) is 6 inches and the unsupported length of the top flange is 120 inches, the ratio $\frac{l}{w}$ will be 20. According to *B. S.*, Art. **103**, the allowable working stress is 16,000 pounds per square inch. Then, the required value of the section modulus is $1,237,000 \div 16,000 = 77.3$. In Table XIV, the section modulus of an 18-inch 55-pound beam is found to be 88.4, which is greater than the required value. This beam will be used for beams *g*.

14. The amount of load that goes to beam *g* from each wheel was found, in Art. **11**, to be 7,692 pounds. The maximum live-load shear is found to occur at the end, when two loads are in a panel, as represented in Fig. 12, and is equal to 13,460 pounds.

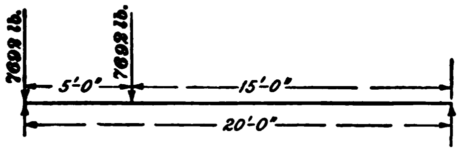


FIG. 12

The allowance for impact and vibration is $\frac{3}{10} \times 13,460 = 3 \times 1,346 = 4,040$ pounds; and the allowance for the overturning effect of the wind is 4,040 pounds. Since the dead load is 180 pounds per linear foot, the dead-load shear at the end is equal to

$$\frac{180 \times 20}{2} = 1,800 \text{ pounds,}$$

making the total shear at the end

$$13,460 + 4,040 + 4,040 + 1,800 = 23,340 \text{ pounds}$$

The connection to the floorbeam will be designed in *Design of a Highway Truss Bridge*, Part 2.

15. Design of Center Stringer.—The amount of load that goes to the beam *k*, Fig. 5, from each axle was found in Art. 11 to be 4,615 pounds. The maximum bending moment is found in the same way as for beam *g* at a section 1.25 feet from the center, and is 424,000 inch-pounds. The allowance for impact and vibration is $\frac{3}{10} \times 424{,}000 = 127{,}200$ inch-pounds. It is unnecessary to allow for any increase due to the wind: the overturning tendency of the wind increases the load on the leeward rail, but at the same time decreases the load on the windward rail by an equal amount, and, since beam *k* gets its load from both rails, the amount that goes to it is neither increased nor diminished.

The dead load supported by the beam *k* has been found to be 125 pounds per linear foot; the weight of the beam will be assumed to be 40 pounds per linear foot, making the total dead load 165 pounds per linear foot. The dead-load moment at the section where the live-load moment is greatest, that is, at 1.25 feet from the center of the panel, is 97,500 inch-pounds. The total bending moment is, therefore,

$$424{,}000 + 127{,}200 + 97{,}500 = 648{,}700 \text{ inch-pounds}$$

If a 12-inch 40-pound beam is used, as the width of flange (Table XIV) is 5.25 inches, and the unsupported length of the top flange is 120 inches, the allowable intensity of bending stress (*B. S.*, Art. 103) is

$$20{,}000 - 200 \times \frac{120}{5.25} = 15{,}430 \text{ pounds per square inch}$$

Then, the required section modulus is $648{,}700 \div 15{,}430 = 42.04$. In Table XIV, the section modulus of a 12-inch 40-pound I beam is found to be 44.8, which is greater than the required value, and this is the lightest beam that can be used. A 12-inch 40-pound I beam will be chosen for beam *k*.

16. The maximum live-load shear is found in the same way as for beam *g* to be 8,080 pounds. The allowance for impact and vibration is $\frac{3}{10} \times 8{,}080 = 2{,}420$ pounds. Since the dead load is 165 pounds per linear foot, the dead-load shear is 1,650 pounds, making the total shear at the end

$$8{,}080 + 2{,}420 + 1{,}650 = 12{,}150 \text{ pounds}$$

INTERMEDIATE FLOORBEAMS AND BRACKETS

17. Length and Depth.—The roadway is supported by floorbeams attached to the trusses at the panel points. The sidewalks are supported on brackets outside of the trusses. These brackets are sometimes continuations of the floorbeams and sometimes independent; in the latter case, they are riveted to the vertical posts of the trusses on the side opposite that to which the floorbeam is riveted, and the flanges of the brackets are connected with the flanges of the floorbeams in such a way that the brackets and floorbeam act as a single beam. The method of connection that will be employed in the present case will be discussed in *Design of a Highway Truss Bridge*, Part 2.

The total length of the beam, including the brackets, is the distance between the fascia girders; in the present case, this distance is 42 feet 6 inches, as shown in Fig. 5. The distance between the supports is equal to the distance center to center of the trusses, or 28 feet. The depth of floorbeam will be made equal to one-eighth the distance between trusses, or $28 \div 8 = 3.5$ feet. The load is transmitted to the beam by the stringers; their location a, b, c, etc., along the beam, and the location of the trusses A and B are shown in Fig. 13.

18. Dead Load.—The railing ordinarily weighs about 40 pounds per linear foot, and the fascia girder about 35 pounds per linear foot; in the present case, the two together will be assumed to weigh 75 pounds per linear foot. Since the distance center to center of the floorbeams is 20 feet, the amount of load that goes to the end of each beam or bracket is $20 \times 75 = 1{,}500$ pounds (a, Fig. 13). The dead load on each sidewalk stringer was found in Art. 8 to be 55 pounds per linear foot; therefore, the load that comes to the floorbeam at each sidewalk stringer is $20 \times 55 = 1{,}100$ pounds (b and c, Fig. 13). The dead load on each roadway stringer was found in Art. 9 to be 107.5 pounds per linear foot; therefore, the load that comes

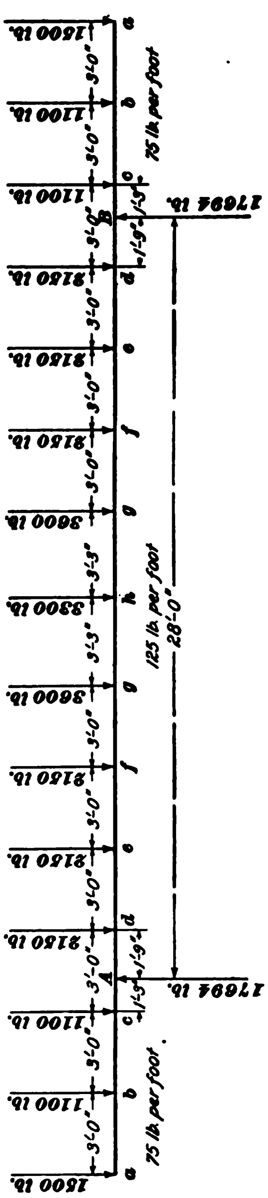


FIG. 13

to the floorbeam at each roadway stringer is 20 × 107.5 = 2,150 pounds (*d*, *e*, and *f*, Fig. 13). The dead load on each of the stringers *g* under the railway track was found in Art. **13** to be 180 pounds per linear foot; therefore, the load that comes to the floorbeam at each of these stringers is 20 × 180 = 3,600 pounds (*g*, Fig. 13). The dead load on the center stringer was found in Art. **15** to be 165 pounds per linear foot; therefore, the load that comes on the floorbeam at the center is 20 × 165 = 3,300 pounds (*h*, Fig. 13).

19. No rule or formula can be given by means of which the weight of brackets and floorbeams can be calculated. In every case it is necessary to estimate first, and then correct the estimate. An experienced designer will estimate closely the first time. In the present case, the weight of each bracket will be assumed to be 75 pounds per linear foot, and the weight of the floorbeam 125 pounds per linear foot. As the loading is symmetrical about the two supports *A* and *B*, Fig. 13, each reaction is equal to one-half the sum of all the loads on the beam and brackets, together with one-half the weight of beam and bracket, or 17,694 pounds. Fig. 13

shows the total dead load on the floorbeam and the dead-load reactions.

20. Dead-Load Shears.—As a rule, it is sufficient to find the shears on the bracket and on the floorbeam at sections close to the trusses. Considering the left truss *A*, Fig. 13, the shear on the bracket just to the left of *A* is negative and equal to 4,240 pounds, and that on the floorbeam just to the right of *A* is positive and equal to 13,450 pounds. If it is found necessary in the design of the web and the computation of rivet pitch to know the shears at other sections, they can be computed later at sections between the stringer connections.

21. Dead-Load Moments.—As a rule, it is sufficient to find the moment on the bracket where it connects to the truss, and on the floorbeam at the center of the bridge. The moment on the bracket where it connects to the truss is negative, and equal to 226,800 inch-pounds. The moment on the floorbeam at the center is positive, and equal to 1,029,500 inch-pounds. If it is found necessary in the design of the flanges to know the moments at other sections, they can be computed later at the sections where the stringers connect with the floorbeams.

22. Live Load.—The uniform live load is 100 pounds per square foot [*B. S.*, Art. 98 (2)]. According to *B. S.*,

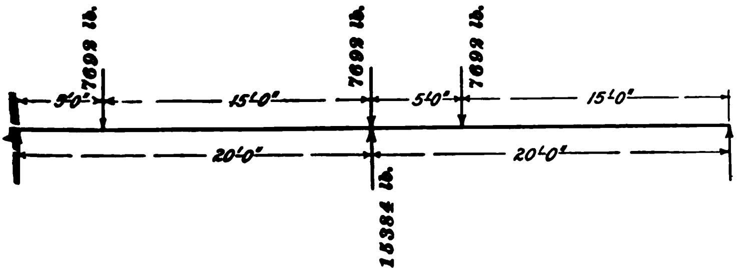


FIG. 14

Art. 98 (5), the uniform load is to be taken as covering the entire floor except a width of 10 feet for the car track. In the design of floorbeams in a bridge that carries a street-car track, it is customary to consider only the uniform load and the weight of a street-car, and to ignore the effect of the road roller, as it will probably never cross the bridge at the same time as a street car. The portion of the width occupied by the truss and wheel-guard—in this case, 2 feet 6 inches at each side—is not assumed to support any live load.

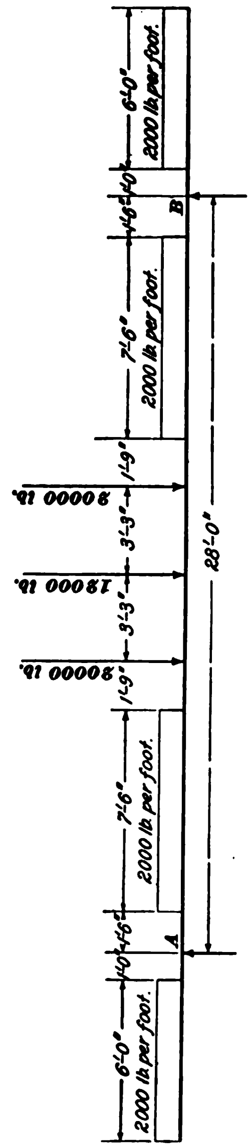


Fig. 15

The maximum load that comes to a floorbeam from the stringers marked *g*, Fig. 5, occurs when the loads in the panel ahead of and in the panel behind the floorbeam are located as represented in Fig. 14. The portion of the weight on one wheel that goes to a beam *g* has been found to be 7,692 pounds; then, the load on the floorbeam is 15,384 pounds. In like manner, the load on the floorbeam at the center stringer is found to be 9,230 pounds. As these loads come from the car, the stresses due to them must be increased to allow for impact and vibration; in the present case, the work will be much simplified and the results will be the same if the foregoing floorbeam loads are increased by the proper amount. The load, including impact and vibration on the floorbeam at the center stringer, is, then, $9,230 + \frac{3}{10} \times 9,230 = 12,000$ pounds, and at each of the stringers marked *g*, Fig. 5, $15,384 + \frac{3}{10} \times 15,384 = 20,000$ pounds, as shown in Fig. 15. The increase in load on the leeward stringer is not considered in the design of the floorbeam.

23. The clear width of each sidewalk is 6 feet. Since the panels are 20 feet in length, the uniform load on each bracket is 20 × 100 = 2,000 pounds per linear foot for a distance of 6 feet, and extends to within 1 foot of the center line of the truss, as represented in Fig. 15.

The clear width of roadway between wheel-guards is 25 feet. Allowing 10 feet for the load on the car track leaves a width of 15 feet, or 7.5 feet on each side of the track that is covered by the uniform load of 2,000 pounds per linear foot. Since the edge of the wheel-guard toward the roadway is 1 foot 6 inches from the center line of the truss, the uniform load will extend to within 1 foot 6 inches of the center line, as shown in Fig. 15.

24. It is unnecessary to compute the amount of load that comes to the floorbeam at each sidewalk and roadway stringer. If the moments and shears are found for the loading shown in Fig. 15, the results will be close enough to the actual values for all practical purposes. Since the loading

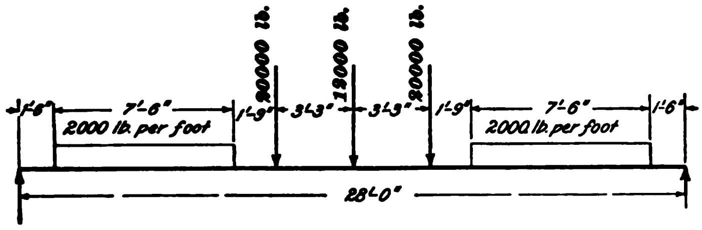


FIG. 16

is symmetrical, the reactions at A and B are each equal to one-half the total load on the beam, or, in this case, 53,000 pounds.

25. Live-Load Shears.—The live-load shear on the bracket just to the left of A is negative and equal to 12,000 pounds. The live-load shear on the floorbeam just to the right of A is positive and equal to 41,000 pounds.

26. Live-Load Moments.—The live-load moment on the bracket is negative and greatest at *A*; it is equal to 576,000 inch-pounds. The live-load moment on the floorbeam is greatest at the center, when there is no live load on the overhanging or sidewalk brackets, and there is a full load on the floorbeam, as shown in Fig. 16; it is positive and equal to 4,533,000 inch-pounds.

27. Design of Floorbeam Web.—The total shear on the floorbeam at the end is 13,450 + 41,000 = 54,450 pounds. It was decided in Art. 17 that a web 42 inches wide would be employed. It is good practice in this type of floor to use a thickness of web that will require no stiffeners between the stringer connections. Then, the smallest allowable unsupported distance is the clear distance between the flange angles, probably about 34 inches in this case, or the clear distance between stringer connections, which is about the same. The connection of stringer *d*, Fig. 5, to the floorbeam is so close to the end that no stiffeners will be required between the end of the floorbeam and this connection; hence, the intensity of shearing stress at the end of the floorbeam need not be computed. The total shear on the floorbeam in the next space, that is, between stringers *d* and *e*, must now be found, so that the allowable unsupported distance can be determined. The shear is practically constant from *d* to *e*, and if it is found half way between them, the result will be close enough. This shear is found by deducting from the end shear the sum of all the loads between the end and the point at which the shear is desired. The shear is as follows: $54{,}450 - (3.25 \times 125 + 2{,}150 + 1.75 \times 2{,}000) = 48{,}390$ pounds.

Using a web $\frac{3}{8}$ inch thick, the area of cross-section is 15.75 square inches; the intensity of shear is $48{,}390 \div 15.75 = 3{,}072$ pounds per square inch, and the allowable unsupported distance (Table XXXVI) is 35 inches. As this is greater than 34 inches, a $42'' \times \frac{3}{8}''$ web will be used, and no stiffeners will be provided except at the stringer connections, which are points of local concentrated loading (*B.S.*, Art. 128).

28. Design of Floorbeam Flanges.—The required area of flange will be found from the following formula, given in *Design of Plate Girders*, Part 1:

$$A = \frac{M}{s h_s} - \frac{t h}{8}$$

In the present case, $t h$ is 15.75 square inches, M is 1,029,500 + 4,533,000 = 5,562,500 inch-pounds, s is 16,000 pounds per square inch (*B. S.*, Art. **103**), and h_s is not known, but will be assumed for trial equal to the depth of web, or 42 inches. Then,

$$A = \frac{5{,}562{,}500}{16{,}000 \times 42} - \frac{15.75}{8} = 8.28 - 1.97 = 6.31 \text{ square inches}$$

For the top flange, two $4'' \times 4'' \times \frac{1}{2}''$ angles will be tried. The gross area is $2 \times 3.75 = 7.5$ square inches, and the center of gravity is 1.18 inches from the backs of the angles (Table IX). For the bottom flange, two $4'' \times 4'' \times \frac{9}{16}''$ angles will be tried. The gross area of one angle (Table IX) is 4.18 square inches; the area to be deducted for one $\frac{3}{4}$-inch rivet (Table XXVII) is .49 square inch. Then, the net area of one angle is $4.18 - .49 = 3.69$ square inches, and that of two angles is 7.38 square inches. The center of gravity of the angles is 1.21 inches from the backs of the angles. The distance center to center of gravity of the flanges is, therefore, $42.25 - 1.18 - 1.21 = 39.86$ inches. Using this value for h_s gives $A = \frac{5{,}562{,}500}{16{,}000 \times 39.86} - \frac{15.75}{8} = 8.72 - 1.97$ $= 6.75$ square inches.

As the flanges that have been tried have the required area, they will be used, that is, two $4'' \times 4'' \times \frac{1}{2}''$ angles for the top flange and two $4'' \times 4'' \times \frac{9}{16}''$ angles for the bottom flange. As there are no flange plates, and the angles are continued the full length of the floorbeam, it is not necessary to draw a curve of the flange areas nor to calculate the moment at any other section.

29. Flange Rivets in Floorbeams.—The pitch of rivets in the flanges will be found by the formula $p = \frac{K h_r}{V}$ (see

Design of Plate Girders, Part 1). In Table XII, the gauge line of an angle 4 inches wide is shown to be 2¼ inches from the back of the angle. Since the flange angles are 42.25 inches back to back, the distance

$$h_r = 42.25 - 2.25 - 2.25 = 37.75 \text{ inches}$$

The flange rivets are usually ⅞ inch in diameter and shop driven; according to *B. S.*, Art. **103**, the allowable working stresses are 11,000 pounds per square inch in shearing and 22,000 pounds per square inch in bearing. The rivets are in double shear and in bearing on the ⅜-inch web-plate. Consulting Table XL, the bearing value is found to be the smaller, the value being 6,190 pounds. At the end of the floorbeam, where the beam connects to the truss, $V = 54{,}450$ pounds; therefore,

$$p = \frac{6{,}190 \times 37.75}{54{,}450} = 4.29 \text{ inches at end of floorbeam}$$

Half way between *d* and *e*, the shear was found to be 48,390 pounds; therefore,

$$p = \frac{6{,}190 \times 37.75}{48{,}390} = 4.83 \text{ inches from } d \text{ to } e, \text{ Fig. 13}$$

Half way between *e* and *f*, the shear is equal to

$$54{,}450 - (6.25 \times 125 + 2 \times 2{,}150 + 4.75 \times 2{,}000)$$
$$= 39{,}870 \text{ pounds;}$$

therefore,

$$p = \frac{6{,}190 \times 37.75}{39{,}870} = 5.86 \text{ inches from } e \text{ to } f, \text{ Fig. 13}$$

Beyond *f*, the rivet pitch comes out greater than 6 inches, and it is unnecessary to calculate it, as the greatest allowable pitch is 6 inches (*B. S.*, Arts. **115** and **130**). Consequently, a pitch of 6 inches will be used for the remainder of the flange.

30. Design of Sidewalk Brackets.—It is customary to make the sidewalk bracket the same depth as the floorbeam where it connects with the truss, and to make it about 1 foot deep at the outside edge of the sidewalk, as shown in Fig. 17. The web shear, rivet pitch, and flange area should be computed at the point or section of maximum bending moment. This occurs where the bracket connects with the truss, which is for convenience assumed to be the center of the truss. As a rule, if the thinnest allowable web is used, and each flange is made of two of the smallest allowable angles—in this case, $2\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × $\frac{5}{16}$ in. (*B. S.*, Art. **113**) —the bracket will have sufficient strength.

The maximum total shear on the bracket is the sum of the dead shear, 4,240 pounds, as found in Art. **20**, and the live shear, 12,000 pounds, as found in Art. **25**; its value is, therefore, 16,240 pounds. If a $\frac{5}{16}$-inch web 42 inches deep at the truss is used, then, as the area of the web is 13.125 square inches, the intensity of shearing stress is 16,240 ÷ 13.125 = 1,240 pounds per square inch. Consulting Table XXXVI, the allowable unsupported distance is found to be about 50 inches. This web can therefore be used, and no stiffeners will be required except at stringer connections (*B. S.*, Art. **128**).

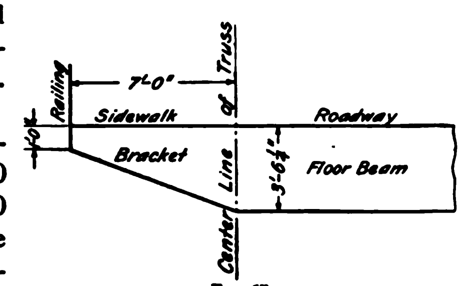


FIG. 17

The maximum bending moment is 226,800 + 576,000 = 802,800 inch-pounds. The area of flange is calculated by the formula $A = \frac{M}{s\,h_g}$ (see *Design of Plate Girders*, Part 1), as it is not customary to consider the effect of the web in a sidewalk bracket. The trial value for h_g, 42 inches, will first be used. This gives, since M = 802,800 foot-pounds and s = 16,000 pounds per square inch,

$$A = \frac{802{,}800}{16{,}000 \times 42} = 1.19 \text{ square inches}$$

The gross area of two $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angles is 2 × 1.46 = 2.92 square inches. Allowing one hole for a $\frac{3}{4}$-inch rivet in each angle, the net area is 2.92 − 2 × .273 = 2.37 square inches. As both of these are much greater than the trial value of the required area, it is unnecessary to compute the true values of h_g and A.

31. The required pitch of rivets is found by the formula $p = \frac{K h_r}{V}$. The rivets are in double shear and in bearing on a web $\frac{5}{16}$ inch thick. Consulting Table XL, the bearing value is found to be the smaller, and is 5,160 pounds. The shear V has been found to be 16,240 pounds. The distance back to back of angles at the deepest portion of the bracket is 42.25 inches. Consulting Table XII, the gauge line of a $2\frac{1}{2}$-inch angle is found to be $1\frac{3}{8}$ inches from the back of the angle, giving the value of $h_r = 42.25 - 1.375 - 1.375 = 39.5$ inches. Then,

$$p = \frac{5{,}160 \times 39.5}{16{,}240} = 12.6 \text{ inches}$$

As this is greater than the maximum allowable pitch [6 inches (*B. S.*, Art. **130**)], the latter will be used throughout the flanges.

END FLOORBEAM AND BRACKET

32. Length and Depth.—It is generally advisable to leave the design of the end floorbeam until after the design of the trusses, as the depth of the floorbeam must be made to conform to details that depend on the design of the truss. For convenience, however, the end floorbeam will now be designed, the depth being taken as $34\frac{1}{4}$ inches, as found in a subsequent article. The length and distances along the beam and bracket are the same as for the other floorbeams, as found in Art. **6**.

33. Dead Load.—The portion of the dead load that goes to the end floorbeam from the stringers is one-half that shown in Fig. 13; the weight of bracket will be assumed to be 75 pounds per linear foot, and of the floorbeam 100 pounds per linear foot. The dead loads are shown in Fig. 18.

34. Dead-Load Shears and Moments.—The shear on the floorbeam at its connection to the truss is 7,250 pounds, and the moment at the center of the floorbeam is 547,000 inch-pounds. It is not necessary to compute the shear and bending moment on the bracket, which will have sufficient strength if

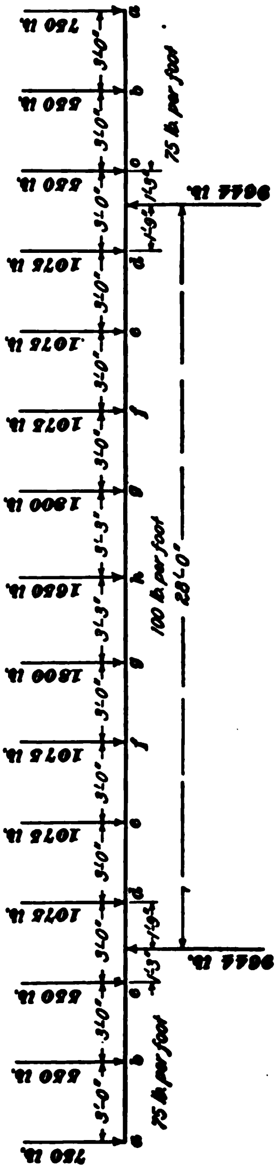

720 lb.
2220 lb.
1072 lb.
1800 lb.
1620 lb.
9644 lb.
75 lb. per foot
100 lb. per foot
28'-0"

FIG. 18

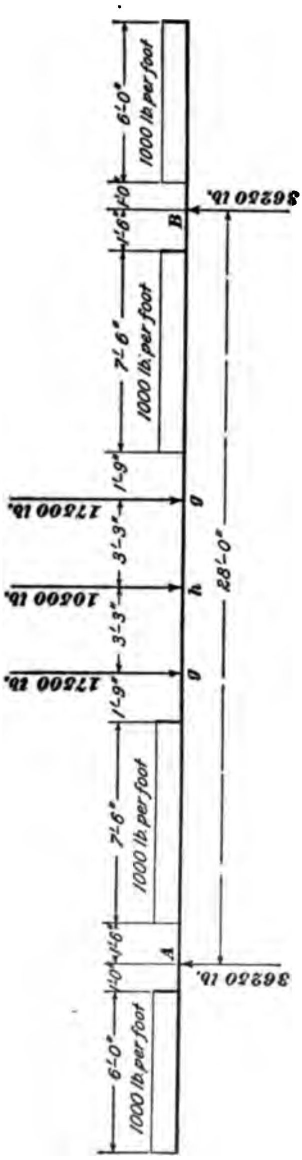

36250 lb.
17500 lb.
10500 lb.
1000 lb. per foot
28'-0"

FIG. 19

the web is made $\frac{5}{16}$ inch thick, and each flange is composed of two $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angles.

35. Live Load.—The live load on the end floorbeam, due to the uniform load on the floor, is one-half that shown in Fig. 16, or 1,000 pounds per linear foot. The live loads at the stringers g and h are more than one-half those shown in Fig. 16, and are greatest when the wheels are in the position shown in Fig. 12. The loads at g and h, including the allowance for impact and vibration, are, respectively, 17,500 and 10,500 pounds. The live loads on the end floorbeam are shown in Fig. 19.

36. Live-Load Shears and Moments.—The live-load shear on the end floorbeam at its connection with the truss is 30,250 pounds, and the bending moment at the center of the floorbeam (when there is no live load on the brackets) is 3,612,000 inch-pounds.

37. Design of Web.—The total shear on the floorbeam is $7{,}250 + 30{,}250 = 37{,}500$ pounds. If a $\frac{5}{16}$-inch web is used, the area of cross-section is $34 \times \frac{5}{16} = 10.625$ square inches, and the intensity of shearing stress is $37{,}500 \div 10.625 = 3{,}530$ pounds per square inch. Consulting Table XXXVI, it is seen that the allowable unsupported distance of the web is 27 inches. If 4-inch angles are used in the flanges, the clear distance between them will be 26 inches, and no stiffeners will be required.

38. Design of Flanges.—The required area of the flange will be found from the formula $A = \frac{M}{s h_g} - \frac{t h}{8}$. In the present case, $t h$ is 10.625 square inches, M is $547{,}000 + 3{,}612{,}000 = 4{,}159{,}000$ inch-pounds, s is 16,000 pounds per square inch, and h_g is not known. In the design of the intermediate floorbeams, it was found that the value of h_g was $1.18 + 1.21 = 2.39$ inches less than the distance back to back of flange angles. For trial, it will be assumed that h_g in the end floorbeam is the same amount less than the distance back to back of flange angles, or 34.25 − 2.39 = 31.86 inches. Then,

$$A = \frac{4,159,000}{16,000 \times 31.86} - \frac{10.625}{8} = 8.16 - 1.33$$
$$= 6.83 \text{ square inches}$$

For the top flange, two 4″ × 4″ × ½″ angles will be tried. The gross area is 2 × 3.75 = 7.50 square inches, and the center of gravity is 1.18 inches from the back of the angles. For the bottom flange, two 4″ × 4″ × $\frac{9}{16}$″ angles will be tried. The net area of each angle, if one ¾-inch rivet hole is deducted, is 4.18 − .49 = 3.69 square inches, and of two angles, 7.38 square inches. The center of gravity is 1.21 inches from the back of the angles. The distance h_e between the centers of gravity of the flanges is, then, 34.25 − 1.18 − 1.21 = 31.86 inches, as was assumed. The trial value of the required area found is, therefore, the actual flange area required.

39. Flange Rivets.—The pitch of rivets will be found by the formula $p = \frac{K h_r}{V}$. In the present case, the value of V is 7,250 + 30,250 = 37,500 pounds, and h_r = 34.25 − 4.5 = 29.75 inches. The rivets are in double shear and in bearing on the web $\frac{5}{16}$ inch thick; the latter value is the smaller, and is 5,160 pounds. Then,

$$p = \frac{5,160 \times 29.75}{37,500} = 4.09 \text{ inches}$$

This is very nearly the same as the pitch at the end of the intermediate floorbeam; hence, the same rivet spacing will be used for both.

40. Sidewalk Bracket.—There is no need to design the sidewalk bracket; the web will be made 34 inches deep at the truss and $\frac{5}{16}$ inch thick, and two 2½″ × 2½″ × $\frac{5}{16}$″ angles will be used in each flange.

DESIGN OF MAIN MEMBERS AND LATERAL SYSTEM

STRESSES IN MAIN MEMBERS

41. Live-Load Stresses.—According to *B. S.*, Art. 98 (2), the uniform load on the floor to be used in the design of the trusses is 80 pounds per square foot for bridges 100 feet in length, and 60 pounds per square foot for bridges 200 feet in length. The uniform load for the bridge under consideration (160 feet in length) is, therefore, 68 pounds per square foot. The total load on the two sidewalks is $2 \times 6 \times 68 = 816$ pounds per linear foot, and on the roadway, exclusive of a width of 10 feet for the car track [*B. S.*, Art. 98 (5)], is $2 \times 7.5 \times 68 = 1{,}020$ pounds per linear foot, making the total live uniform load on the floor $1{,}020 + 816 = 1{,}836$ pounds per linear foot. The panel loads are each

$$\frac{1{,}836 \times 20}{2} = 18{,}360 \text{ pounds,}$$

and the reaction for each truss when fully loaded (neglecting the half-panel loads at each end) is

$$\frac{18{,}360 \times 7}{2} = 64{,}260 \text{ pounds}$$

The stresses caused in the chord members by this portion of the live load are determined as explained in *Stresses in Bridge Trusses*, Part 2, and are as follows (see Fig. 20):

Member	Stress, in Pounds
ab, bc	47,600 (tension)
cd	81,600 (tension)
de	102,000 (tension)
BC	81,600 (compression)
CD	102,000 (compression)
DE	108,800 (compression)

The shears in the panels due to this portion of the load are as follows:

Panel	Shear, in Pounds	Panel	Shear, in Pounds
a b	64,260	*e d'*	13,770
b c	48,195	*d' c'*	6,885
c d	34,425	*c' b'*	2,295
d e	22,950	*b' a'*	0

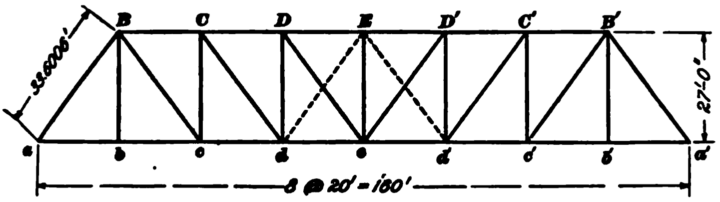


Fig. 20

42. According to *B. S.*, Art. **98** (4), the load on the car track for a span of 160 feet must be taken as 1,345 pounds per linear foot for the computation of the chord stresses. This gives a panel load for each truss of 13,450 pounds, and each reaction for a full load (neglecting the half-panel loads at the ends) is

$$\frac{13{,}450 \times 7}{2} = 47{,}075 \text{ pounds}$$

The stresses caused in the chord members by this portion of the live load are as follows:

Member	Stress, in Pounds
a b, b c	34,870 (tension)
c d	59,780 (tension)
d e	74,720 (tension)
BC	59,780 (compression)
CD	74,720 (compression)
DE	79,700 (compression)

43. These stresses must be increased to allow for the effect of impact and vibration. The increase is computed by the following formula (*B. S.*, Art. **99**):

$$I = \frac{300 - L}{1{,}000} \times S$$

As chord stresses are under consideration, the entire span is loaded when they are greatest; then, $L = 160$, and, therefore,

$$I = \frac{300 - 160}{1{,}000} = .14\,S$$

The total live-load chord stresses, in pounds, including impact and vibration, are as follows:

Member	Tension
ab, bc	$47{,}600 + 34{,}870 + .14 \times 34{,}870 = 87{,}350$
cd	$81{,}600 + 59{,}780 + .14 \times 59{,}780 = 149{,}750$
de	$102{,}000 + 74{,}720 + .14 \times 74{,}720 = 187{,}180$
	Compression
BC	$81{,}600 + 59{,}780 + .14 \times 59{,}780 = 149{,}750$
CD	$102{,}000 + 74{,}720 + .14 \times 74{,}720 = 187{,}180$
DE	$108{,}800 + 79{,}700 + .14 \times 79{,}700 = 199{,}660$

44. In computing the stresses caused in the web members by the load on the car track, the load at each floorbeam is to be taken as 1,600 p at $\frac{100}{p}$ floorbeams [*B. S.*, Art. 98 (4)]. In the present case, as the panel length p is 20 feet, this gives $1{,}600 \times 20 = 32{,}000$ pounds at each floorbeam, one-half of which, or 16,000 pounds, is the panel load on one truss; and the number of panel points that must be loaded is $100 \div 20 = 5$.* Those panel points must be loaded that will cause the desired stresses to be greatest. For example, to find the maximum shear caused by this portion of the load in the panel ab, the five joints b, c, d, e, and d' must be loaded; for the panel bc, the five joints c, d, e, d', and c', etc. must be loaded. When there are less than five panel points to the right of a panel, all the joints to the right are loaded, and the remaining loads are considered to be off the

*When $\frac{100}{p}$ does not give a whole number, the next larger whole number must be used. Thus, if $\frac{100}{p} = 5.34$, this should be called 6.

bridge. The live-load positive shears due to this loading are as follows:

Panel	Shear, in Pounds	Panel	Shear, in Pounds
ab	50,000	ed'	12,000
bc	40,000	$d'c'$	6,000
cd	30,000	$c'b'$	2,000
de	20,000	$b'a'$	0

45. These shears must be increased to allow for the effect of impact and vibration. As before, the increase is computed by the formula

$$I = \frac{300 - L}{1{,}000} \times S$$

As web stresses are under consideration, however, the length of track that is loaded to produce the maximum stresses is different for different members. It is customary to take for the loaded length in this case the distance from the right-hand end of the span up to the first panel load, or the panel load that is farthest to the left. Then, the loaded length for the panel ab is 140 feet; for the panel bc, 120 feet; for the panel cd, 100 feet; for the panel de, 80 feet; for the panel ed', 60 feet; for the panel $d'c'$, 40 feet; and for the panel $c'b'$, 20 feet. The total live-load positive shears in the different panels, including the allowances for impact and vibration, are as follows:

Panel	Shear, in Pounds
ab	$64{,}260 + 50{,}000 + .16 \times 50{,}000 = 122{,}260$
bc	$48{,}195 + 40{,}000 + .18 \times 40{,}000 = 95{,}395$
cd	$34{,}425 + 30{,}000 + .20 \times 30{,}000 = 70{,}425$
de	$22{,}950 + 20{,}000 + .22 \times 20{,}000 = 47{,}350$
ed'	$13{,}770 + 12{,}000 + .24 \times 12{,}000 = 28{,}650$
$d'c'$	$6{,}885 + 6{,}000 + .26 \times 6{,}000 = 14{,}445$
$c'b'$	$2{,}295 + 2{,}000 + .30 \times 2{,}000 = 4{,}895$
$b'a'$	0

46. Dead-Load Stresses.—The dead load consists of the weight of the floor, including the weight of the stringers and floorbeams, and the weight of the trusses, including the lateral system. The weight of the floor, as found in Art. **19** and illustrated in Fig. 13, is 17,694 pounds at each end of each floorbeam, that is, at each panel point. As the panels are 20 feet in length, this corresponds to a weight of $17{,}694 \div 20 = 884.7$, or nearly 900 pounds, per linear foot for one truss. The latter value will be used. Each panel load is then $900 \times 20 = 18{,}000$ pounds.

The approximate weight w of one truss is given by the formula

$$w = \frac{W}{12}\left[1 + 2\left(\frac{l - 90}{100}\right)^2\right] \quad \text{(See } B.\ S.\text{, Art. } \mathbf{243}\text{)}$$

in which l = span;

W = the total load per linear foot supported by the truss, exclusive of its own weight.

The dead load supported by the truss was found in the preceding paragraph to be 900 pounds per linear foot. The total live load on the floor, due to the uniform load, was found in Art. **41** to be 1,836 pounds per linear foot, or 918 pounds per linear foot per truss. The load on the car track was found in Art. **42** to be 1,345 pounds per linear foot, or 672.5 pounds per linear foot per truss. Since the stresses due to the last-named load are increased to allow for impact and vibration, it is well, in calculating the load supported by the truss, to increase the live load by the proper amount. This gives the total load coming to one truss from the car track, including allowance for impact and vibration, as $672.5 + .14 \times 672.5 = 766.65$ pounds per linear foot. Total load W supported by one truss is, then, $900 + 918 + 766.65 = 2{,}584.65$ pounds per linear foot; therefore,

$$w = \frac{2{,}584.65}{12} \times \left[1 + 2 \times \left(\frac{160 - 90}{100}\right)^2\right] = 426.5 \text{ lb. per lin. ft.}$$

The panel load due to the weight of the truss is, then, $426.5 \times 20 = 8{,}530$ pounds, of which one-half, or 4,265 pounds, is assumed to be applied at the loaded chord, and one-half at the unloaded chord (see *B. S.*, Art. **97**). The entire weight of the floor is taken at the loaded chord. The lateral system will be assumed to cause a panel load of 1,000 pounds, one-half at each chord. Then, each panel load of the unloaded chord is 4,265 + 500 = 4,765 pounds, and each panel load of the loaded chord is 4,765 + 18,000 = 22,765 pounds. The total dead panel load is 22,765 + 4,765 = 27,530 pounds, and each reaction is 96,355 pounds. The dead-load chord stresses are as follows:

MEMBER	STRESS, IN POUNDS
ab, *bc*	71,370 (tension)
cd	122,360 (tension)
de	152,940 (tension)
BC	122,360 (compression)
CD	152,940 (compression)
DE	163,140 (compression)

The dead-load shears are as follows:

PANEL	POSITIVE SHEAR, IN POUNDS	PANEL	NEGATIVE SHEAR, IN POUNDS
ab	96,355	*ed'*	13,765
bc	68,825	*d'c'*	41,295
cd	41,295	*c'b'*	68,825
de	13,765	*b'a'*	96,355

47. Wind Pressure.—In the design of trusses, it is customary to assume that the maximum wind pressure does not occur simultaneously with the maximum live load, so that the stresses caused in the chord members and end posts of the trusses by the wind pressure should not be added to the combined dead- and live-load stresses without some additional allowance. Practice varies as to the method of allowing for these wind stresses. It is frequently specified that, when they are less than 25 per cent. of the total combined dead- and live-load stresses, the wind stresses may be ignored; when greater, the members are designed for the sum of the maximum dead, live, and wind stresses, using working stresses 25 per cent. greater than those allowed for the dead- and live-load stresses alone. This method will be used here. As a rule, the members are first designed for the combined dead- and live-load stresses; the exposed area of the trusses and floor, the wind pressure on them, and the stresses due to this wind pressure are then calculated, and the cross-sections of the members corrected, if necessary, to allow for the wind stresses.

48. Longitudinal Force.—In this type of bridge, the longitudinal thrust of a suddenly stopping car is so distributed throughout the length and width of the bridge by the rails, stringers, and floor plank that it will not be considered. If the trusses were supported on a steel trestle, the force would be taken at the top of the bent.

49. Combined Stresses.—The combined dead- and live-load stresses in the chord members are as follows:

Member	Stress, in Pounds
a b, *b c*	87,350 + 71,370 = 158,720 (tension)
c d	149,750 + 122,360 = 272,110 (tension)
d e	187,180 + 152,940 = 340,120 (tension)
B C	149,750 + 122,360 = 272,110 (compression)
C D	187,180 + 152,940 = 340,120 (compression)
D E	199,660 + 163,140 = 362,800 (compression)

The combined dead- and live-load positive shears in the different panels are as follows:

Panel	Shear, in Pounds
a b	122,260 + 96,355 = 218,615
b c	95,395 + 68,825 = 164,220
c d	70,425 + 41,295 = 111,720
d e	47,350 + 13,765 = 61,115
e d'	28,650 − 13,765 = 14,885
d' c'	14,445 − 41,295 = − 26,850

As the combined shear in the panel *e d'* comes out positive. a counter is required in that panel. As the combined shear in the panel *d' c'* comes out negative, no counter is required in that panel.

The stresses in the diagonals can be found by multiplying the shears in the respective panels by $\csc H$. The length of a diagonal is $\sqrt{20^2 + 27^2} = 33.6006$, and $\csc H = 33.6006 \div 27 = 1.2445$. Then, the stresses in the diagonals are as follows:

DIAGONAL	STRESS, IN POUNDS
aB	218,615 × 1.2445 = 272,100 (compression)
Bc	164,220 × 1.2445 = 204,400 (tension)
Cd	111,720 × 1.2445 = 139,000 (tension)
De	61,115 × 1.2445 = 76,100 (tension)
Ed'(*dE*)	14,885 × 1.2445 = 18,500 (tension)

The stresses in the verticals, except the hip vertical, are each equal to the sum of the shear in the panel to the right of the vertical and the dead load at the top joint (4,765 pounds, Art. **46**). The stresses are as follows:

MEMBER	STRESS, IN POUNDS
Cc	111,720 + 4,765 = 116,500 (compression)
Dd	61,115 + 4,765 = 65,880 (compression)
Ee	14,885 + 4,765 = 19,650 (compression)

According to *B. S.*, Art. **98** (2), the stress in the hip vertical must be found from the same loading as the stress in the floorbeam. In Art. **19**, the dead load on the truss from one floorbeam was found to be 17,694 pounds; and in Art. **24**

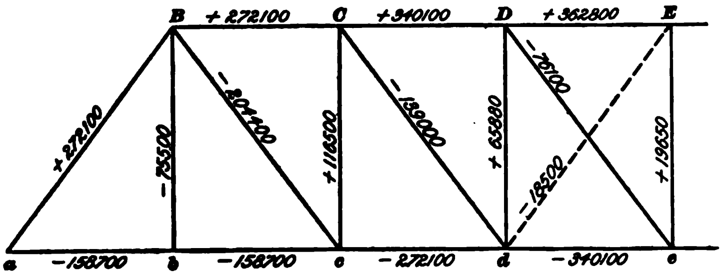


FIG. 21

the live load, including impact and vibration, was found to be 53,000 pounds. In addition, there is a dead load of 4,765 pounds at each panel point of the loaded chord, making the total stress in the hip vertical 17,694 + 53,000 + 4,765 = 75,459, or about 75,500 pounds, tension.

The total dead- and live-load stresses in the members are shown in Fig. 21.

DESIGN OF MAIN MEMBERS

50. Bottom Chord.—The bottom chord will be composed of eyebars (see *B. S.*, Art. **139**). The working stress in tension is given in *B. S.*, Art. **103**, as 16,000 pounds per square inch. Then, the required net areas of the bottom chord members are as follows:

Member	Required Net Area, in Square Inches
a b, *b c*	158,700 ÷ 16,000 = 9.92
c d	272,100 ÷ 16,000 = 17.01
d e	340,100 ÷ 16,000 = 21.26

The required areas can be made up in a number of ways by using different widths of eyebars, and no fixed rule can be given for the width that should be used. In general, however, eyebars from 4 to 8 inches in width are preferable for chord members of highway-bridge trusses from 150 to 175 feet in length. The smallest thicknesses that can be used are given in Table XXX. The actual thickness should, in general, be not greater than about twice the minimum thickness, although it is better not to exceed the minimum thickness by a very large amount. The bending moments on the pins are as a rule less when thin eyebars are used than when thick bars are used.

1. *Member d e.*—In the present case, four bars 5 in. × $1\frac{1}{16}$ in. will be used for the member *d e*. The area of one bar of this size is 5.3125 square inches, and of four bars, 21.25 square inches, which is near enough to the required area.

2. *Member c d.*—For the member *c d*, four bars 5 in. × $\frac{7}{8}$ in. will be used. The area of one bar of this size is 4.375 square inches, and of four bars, 17.5 square inches, which is slightly greater than required.

3. *Members a b and b c.*—For the members *a b* and *c d*, it is specified in *B. S.*, Art. **139**, that the bars composing them must be connected to each other by latticing; this is to make these members capable of resisting a small amount of compression, if for any reason, such as a heavy wind storm, the stresses in them are reversed. The method of connecting

them by latticing is shown in Fig. 22: an angle is riveted to the inside of each eyebar, and the outstanding legs are connected by single latticing. The area of each bar so connected must be reduced by the area of one rivet hole; $\frac{3}{4}$-inch rivets will be used. If two bars 6 in. × 1 in. are used, the gross area of each bar is 6 square inches, and the net area is 6 − .875 = 5.125 square inches; then, tne area of two bars is

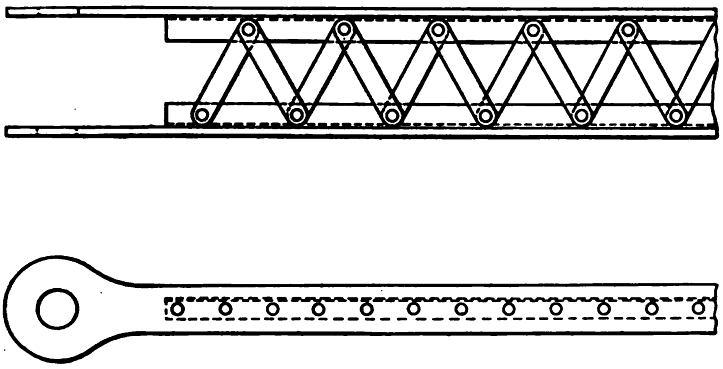


FIG. 22

10.25 square inches. As this is slightly greater than the required area, these bars will be used for *a b* and *c d*.

51. Inclined Web Members.—The required net areas of the inclined web members, not including the end post, which will be considered later, are as follows:

MEMBER	REQUIRED NET AREA, IN SQUARE INCHES
Bc	204,400 ÷ 16,000 = 12.775
Cd	139,000 ÷ 16,000 = 8.69
De	76,100 ÷ 16,000 = 4.76
dE	18,500 ÷ 16,000 = 1.16

1. *Member Bc.*—For *Bc*, two bars 6 in. × $1\frac{1}{8}$ in. will be used. The area of one bar is 6.75 square inches, and of two bars, 13.5 square inches, which is sufficient.

2. *Member Cd.*—For *Cd*, two bars 5 in. × $\frac{7}{8}$ in. will be used. The area of one bar is 4.375 square inches, and of two bars, 8.75 square inches, which is sufficient.

3. *Member De.*—For De, two bars 3 in. × $\frac{13}{16}$ in. will be used. The area of one bar is 2.4375 square inches, and of two bars, 4.875 square inches, which is sufficient.

4. *Member dE.*—The member dE is a counter, and, according to *B. S.*, Art. **141**, cannot have a sectional area less than 2 square inches, although but 1.16 square inches is required by the stress. One bar 3 in. × $\frac{3}{4}$ in. will be used; its area of cross-section is 2.25 square inches, which is greater than 2 square inches.

52. Vertical Web Members.—The form of cross-section that is best adapted to the compression web members is shown in Fig. 23. It is composed of two channels with the flanges pointing toward each other; the flanges are connected by tie-plates and lattice bars, as explained in *Bridge Members and Details*, Part 1. The size of channel that must

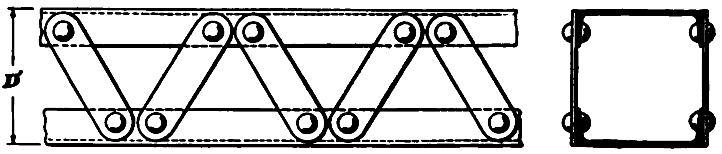


FIG. 23

be used for any member depends in general on the stress in the member, but for those verticals near the center of the span, in which the stress is comparatively small, other conditions frequently govern. For example, it is specified in *B. S.*, Art. **105**, that the value of $\frac{l}{r}$ shall not exceed 100 for main members. Since the length of each vertical from center to center of pins is 27 feet = 324 inches, the smallest allowable value of the radius of gyration r is 324 ÷ 100 = 3.24 inches. Consulting Table XIII, it is seen that the smallest channel having a value of r greater than 3.24 inches is a 9-inch 13.25-pound channel. The web of this is only .23 inch thick, and, as it is specified in *B. S.*, Art. **112**, that webs of channels shall be not less than $\frac{1}{4}$ inch thick, this channel cannot be used. A 9-inch 15-pound channel is found to be the smallest channel that can be used. Its radius of gyration is 3.4 inches, and $\frac{l}{r} = \frac{324}{3.4} = 95.3$. The gross area of two channels is (Table XIII) $2 \times 4.41 = 8.82$ square inches. Consulting Table XXXV, the allowable working stress corresponding to a value of $\frac{l}{r}$ of 95.3 is found to be 10,640 pounds per square inch. Since the gross area of two channels is 8.82 square inches, the total compressive stress that can be resisted by a member 27 feet long and composed of two 9-inch 15-pound channels is $8.82 \times 10{,}640 = 93{,}800$ pounds. As this is greater than the stress in either Dd or Ee, these channels will be used for these two members.

As the stress in Cc (+ 116,500 pounds) is greater than 93,800 pounds, the two channels just considered are not large enough for this member, and larger or heavier channels must be used. The radius of gyration of each of the heavier 9-inch channels is less than the smallest allowable value, 3.24 inches, as found in the preceding paragraph, so that the next heavier 10-inch channel, 20 pounds, will be tried. The radius of gyration of a 10-inch 20-pound channel is given in Table XIII as 3.66 inches; then, $\frac{l}{r} = \frac{324}{3.66} = 88.5$, and, by Table XXXV, the allowable working stress is 11,150 pounds per square inch. The gross area of two channels is (Table XIII) $2 \times 5.88 = 11.76$ square inches, and the total stress that can be resisted by two channels is $11.76 \times 11{,}150 = 131{,}100$ pounds. As this is greater than the stress in Cc, these channels will be used for this member.

53. Hip Vertical.—The stress in Bb is 75,500 pounds, tension, and, since the allowable working stress (*B. S.*, Art. **103**) is 16,000 pounds per square inch, the required net area of the member is $75{,}500 \div 16{,}000 = 4.7$ square inches. The same form of cross-section can be used for this member as for the other verticals, but that shown in Fig. 24 is frequently used, and will be employed here. The outstanding leg should be at least equal in width to the outstanding leg of the floorbeam connection angle. According to *B. S.*, Art. **126**, the connection angles cannot be less than 3 inches wide. Then, $3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angles will be tried, as they are the smallest that can be used. To get the net area of each angle, it will be sufficient to deduct the area of one hole for a $\frac{3}{4}$-inch rivet from each angle, that is, .27 square inch (Table XXVII). The gross area of one angle is (Table X) 1.62 square inches, and the net area, $1.62 - .27 = 1.35$ square inches. Then, the area of the member Bb, if composed of four angles, is $4 \times 1.35 = 5.4$ square inches, which is greater than the required value, and therefore sufficient. It will be found in connection with the details, however, that it is advisable to

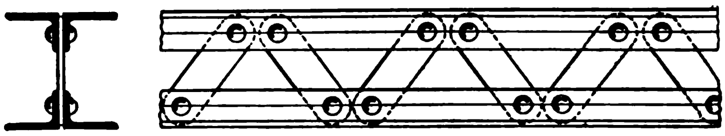


FIG. 24

use $3\frac{1}{2}$-inch angles for connecting the floorbeams to the verticals. For this reason, it is well to use four angles $3\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. $\times$ $\frac{5}{16}$ in. for Bb.

54. Width of Verticals.—The width of the hip vertical is usually made about the same as the compression verticals. The channels that compose the latter are placed far enough apart so that the radius of gyration about an axis parallel to the web is at least equal to that about an axis perpendicular to the webs. The distances between the webs of two channels that form a compression member, when the flanges are turned outwards, so that the radii of gyration in the two directions will be the same, is given in column 17, Table XIII. The distance D', Fig. 23, between the backs of the channels when the flanges are turned inwards or toward each other can be found by means of the formula

$$D' = D + 4x$$

in which D = distance found in column 17, Table XIII;

x = distance from back of channel to center of gravity, as found in column 12, Table XIII.

For two 10-inch 20-pound channels, $D' = 5.97 + 4 \times .61 = 8.41$ inches; and for two 9-inch 15-pound channels, $D' = 5.49 + 4 \times .59 = 7.85$ inches. They will be placed $8\frac{1}{2}$ inches apart in the present case.

55. Top Chord.—The form of cross-section shown in Fig. 25 is frequently used for the top chords and end posts of trusses. The pins are located near the center of gravity of the section, which is nearer the top than the bottom; and, in order to provide room between the pin and the top cover-plate to accommodate the eyebar heads, it is sometimes necessary to use deeper chord members than would otherwise be desirable. For this reason, this form of cross-section is not as desirable for light pin-connected trusses as the symmetrical form shown in Fig. 26, in which the center line is located a little below the center of gravity. This form will be used

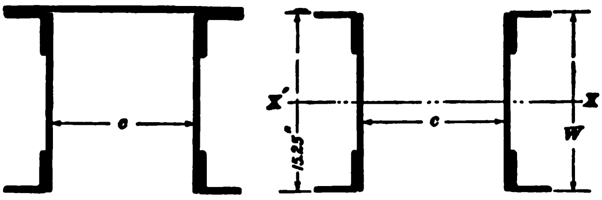


FIG. 25 FIG. 26

in the present case. The depth or width W should be at least great enough to allow the head of the largest eyebar to go inside. The largest eyebar that connects to the top chord is 6 inches wide. In Table XXX, the sizes of heads for 6-inch eyebars are given from $13\frac{1}{2}$ inches to $15\frac{1}{2}$ inches in diameter; a depth of 15 inches will be tried first.

The distance c between webs depends on the width of the vertical members and on the arrangement of the ends of the members with respect to each other. It is customary to place the heads of the eyebars that form the main diagonals between the vertical members and the inside of the chord members. As the width of the vertical members is $8\frac{1}{2}$ inches (Art. 54), the clear distance c between the webs of the top chord will be taken for the present as 12 inches. If necessary, this can be changed slightly when detailing.

In *Bridge Members and Details*, Part 1, it was explained that, when, as in this case, the distance c is greater than $\frac{3}{4}W$, the least radius of gyration is approximately equal to $\frac{1}{3}W$, in this case 5 inches. Since the unsupported length of the top chord members is 240 inches, the approximate value of $\frac{l}{r}$ is $240 \div 5 = 48$, and the allowable working stress (Table XXXV) is 14,180 pounds per square inch. Then, the trial values of the required cross-section of the top chord members are as follows (see Fig. 21):

Member	Trial Value, in Square Inches
BC	$272{,}100 \div 14{,}180 = 19.19$
CD	$340{,}100 \div 14{,}180 = 23.99$
DE	$362{,}800 \div 14{,}180 = 25.59$

The top chord is usually made in sections from 30 to 60 feet in length, which are spliced in the field. In the present case, the splices will be located near D and D', Fig. 20, and the chord will be composed of three parts.

1. *Member BC*.—For the member BC, the following sections will be tried:

	Square Inches
Two web-plates 15 in. × $\frac{5}{16}$ in.	9.38
Four angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in.	9.92
Total	19.30

The flange angles will be placed with the backs of their outstanding legs $15\frac{1}{4}$ inches apart, as shown in Fig. 26. The radius of gyration about axis $X'X$ through the center of gravity of the section is computed by the method explained in *Bridge Members and Details*, Part 1, and is found to be 5.67 inches; the value of $\frac{l}{r}$ is $240 \div 5.67 = 42.3$; and the allowable working stress is 14,550 pounds per square inch. Then, the corrected value of the required area of cross-section is $272{,}100 \div 14{,}550 = 18.70$ square inches. As this is so close to the area tried, the assumed section will be used for BC.

2. *Member CD.*—As CD is a continuation of BC, all the simple sections of which BC is composed are also parts of CD, and any additional area required for CD is provided by adding plates. The flange angles are far enough apart to allow an 8-inch vertical plate between the vertical legs of the angles, as shown at *a*, *a* in Fig. 27. The following section will be tried for CD:

	Square Inches
Two web-plates 15 in. × $\frac{5}{16}$ in.	9.38
Four angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in.	9.92
Two vertical plates 8 in. × $\frac{5}{16}$ in.	5.00
Total	24.30

The radius of gyration with reference to the axis $X'X$ through the center of gravity of the section is found to be 5.16 inches; the value of $\frac{l}{r}$ is $240 \div 5.16 = 46.5$; and the allowable working stress is 14,290 pounds per square inch. Then, the corrected value of the required area of cross-section is $340{,}100 \div 14{,}290 = 23.80$ square inches. As this is so close to the area tried, the assumed section will be used for CD.

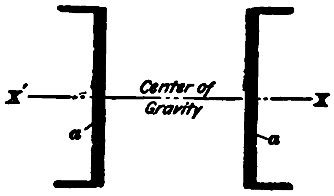


Fig. 27

3. *Member DE.*—For member DE, the same form will be used as for CD. The following section will be tried:

	Square Inches
Two web-plates 15 in. × $\frac{5}{16}$ in.	9.38
Four angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in.	9.92
Two vertical plates 8 in. × $\frac{7}{16}$ in.	7.00
Total	26.30

The radius of gyration with reference to $X'X$ is 5.01 inches; the value of $\frac{l}{r}$ is $240 \div 5.01 = 47.9$; and the allowable working stress is 14,170 pounds per square inch. Then, the corrected value of the required area of cross-section is

362,800 ÷ 14,170 = 25.60 square inches. As this is so close to the area tried, the assumed section will be used for *DE*.

56. End Post.—The end post will be made of the same general form as the top chords. The unsupported length of the end post is 33.6 feet, or 403.2 inches; the radius of gyration will be assumed equal to 5 inches. The value of the ratio $\frac{l}{r}$ is, then, 403.2 ÷ 5 = 80.6, and the allowable working stress is 11,760 pounds per square inch. Therefore, the trial value of the required area of cross-section is 272,100 ÷ 11,760 = 23.14 square inches. The same section will be used for this member as for the top chord member *CD*, unless it is necessary to revise the section later on account of the wind stresses.

DESIGN OF LATERAL SYSTEM

WIND STRESSES

57. Introductory Statement.—Before designing the details, it is advisable to compute the wind stresses, so that, if necessary, the sections of the main members may be changed. The lateral system will also be designed at this time.

58. Wind Pressure.—The wind pressure will be taken as 50 pounds per square foot on twice the exposed area of one truss together with the floor (*B. S.*, Art. 100). In calculating the exposed area of a member, it is customary to multiply its width by the distance between the centers of its connections. In tension members composed of eyebars, the width of the eyebar is used; in built-up members, 1 inch is added to the depth of the web or channel, or the width of angles, to allow for the latticing. The exposed widths of the members at one end of the truss are as follows (see Fig. 20): for *aB*, *BC*, *CD*, and *DE*, 16 inches; for *Bb*, 7 inches; for *Cc*, 11 inches; for *Dd* and *Ee*, 10 inches; for

Bc, 6 inches; for *Cd*, 5 inches; for *De* and *Ed*, 3 inches; for *ab* and *bc*, 6 inches; and for *cd* and *de*, 5 inches. Multiplying each of these widths by the length of the member, the exposed area of the upper chord of one truss is found to be 160 square feet; of the lower chord, 73.33 square feet; and of the web members, 333.3 square feet. Then, the wind pressure on twice the exposed area of one truss on the top chords is

$$2 \times 160 \times 50 = 16{,}000 \text{ pounds;}$$

on the bottom chords,

$$2 \times 73.3 \times 50 = 7{,}330 \text{ pounds;}$$

and on the web members,

$$2 \times 333.3 \times 50 = 33{,}330 \text{ pounds}$$

The exposed area of the floor is equal to the length of the span (160 feet) multiplied by the vertical distance from the

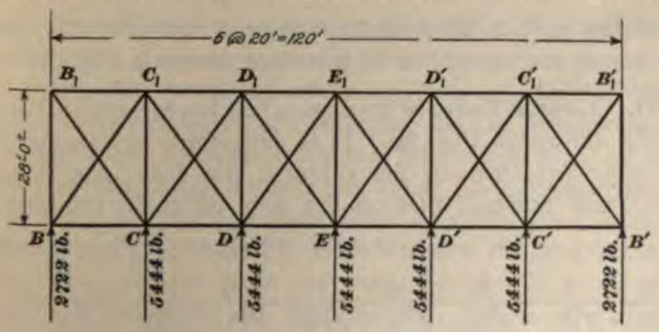


FIG. 28

highest portion of the floor (the edge of the sidewalk) to the bottom of the lowest **I** beam, about 3 feet, Fig. 5. It is, therefore, $160 \times 3 = 480$ square feet, and the wind pressure on it is $480 \times 50 = 24{,}000$ pounds. The wind pressure on the railings will be taken as 75 pounds per linear foot, or $75 \times 160 = 12{,}000$ pounds on the entire length of railings.

59. Upper Lateral Truss.—The upper lateral truss is shown in Fig. 28. It is customary to assume that the wind pressure on the top chord and one-half that on the web members—in this case, $16{,}000 + 33{,}330 \div 2 = 32{,}665$ pounds—is resisted by the top lateral truss. Since the top chord is

120 feet long, this corresponds to 32,665 ÷ 120 = 272.2 pounds per linear foot, and the load per panel is 272.2 × 20 = 5,444 pounds, as represented in Fig. 28. For convenience, the entire panel load is assumed to be applied at the windward side. There are also to be considered the two half-

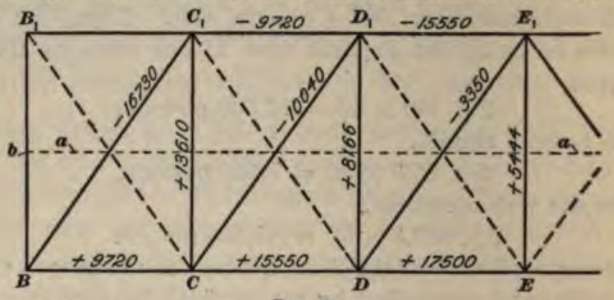


FIG. 29

panel loads of 2,722 pounds each at B and B'. The wind stresses in the upper lateral truss are shown in Fig. 29.

60. Lower Lateral Truss.—The lower lateral truss is clearly shown in Fig. 30. It is customary to assume that the wind pressure on the railings, the floor, the bottom chord, and one-half that on the web, which in this case is

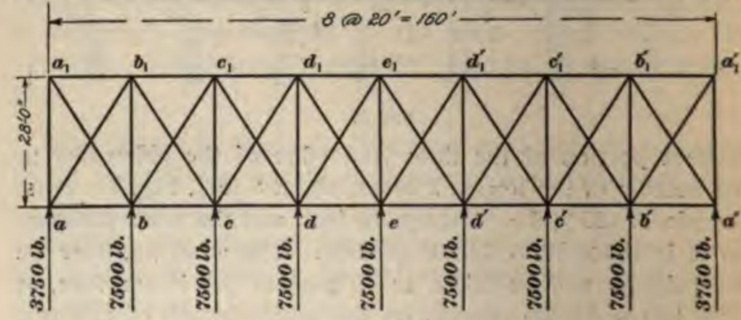


FIG. 30

12,000 + 24,000 + 7,330 + 33,330 ÷ 2 = 60,000 pounds, is resisted by the bottom lateral truss. As the bottom chord is 160 feet long, this corresponds to 60,000 ÷ 160 = 375 pounds per linear foot. The panel load is 375 × 20 = 7,500 pounds, as shown in Fig. 30. For convenience, all the panel load is assumed to be applied at the windward side. The half-panel loads at *a* and *a'* will be neglected; they are

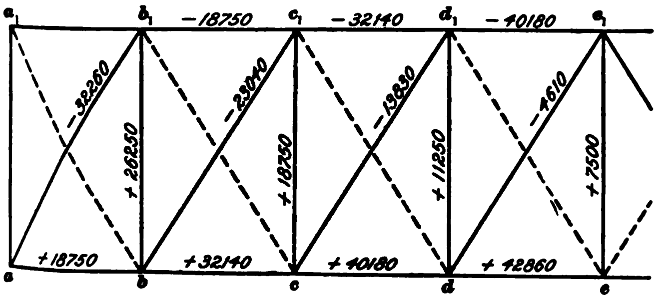


FIG. 31

transmitted directly to the abutments by the pedestals. The stresses in the lower lateral truss are shown in Fig. 31.

61. Portal and Transverse Frames.—The depth of the portal and transverse frames depends on the amount of room above the overhead clearance line, and this in turn depends on the position of the floor. In the present case, as the allowable distance from the top of the floor to the underneath clearance line is 6 feet (Art. **1**), the floorbeams will be connected to the vertical posts above the pins, leaving the lower chord to project a short distance below the floorbeams. To clear the main diagonals, which will be outside the vertical posts, as will be shown in *Design of a Highway Truss Bridge*, Part 2, the bottom of the floorbeam will be placed 1 foot above the center of the pins in the lower chord. It will be seen later, in connection with the details, that the top of the floor at the center of the bridge is 9 inches above the top of the floorbeam; as the floorbeam is 3 feet 6¼ inches deep, the top of the floor is 9 inches + 3 feet 6¼ inches + 1 foot = 5 feet 3¼ inches above the center of the lower chord, and 27 feet − 5 feet 3¼ inches = 21 feet 8¾ inches below the center of the top chord. As this is more than 20 feet, transverse frames are required at each panel point (*B. S.*, Art. **159**).

In *B. S.*, Art. **94**, the required headroom is specified as 15 feet. This leaves 6 feet 8¾ inches from the overhead clearance line to the center of the top chord. The top laterals are connected to the top of the top chord about 8 inches from

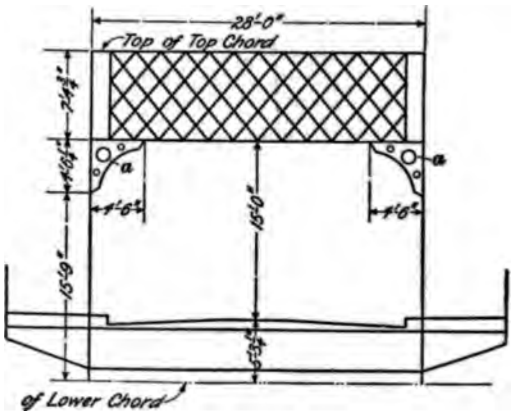


Fig. 32

the center of the chord, making the total depth of the transverse frames about 6 feet 8¾ inches + 8 inches = 7 feet 4¾ inches. Curved brackets will be placed at each end of

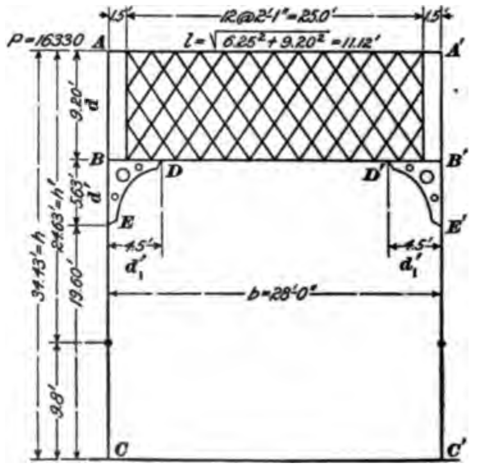


Fig. 33

each transverse frame. Since the edges of the wheel-guards are 1 foot 6 inches from the center of the trusses, the brackets will extend 4 feet 6 inches from the center of the trusses (*B. S.*, Art. **94**). They will also be made about 4 feet 6 inches in depth, as represented in Fig. 32, which is a cross-section of the bridge showing a type of frame and bracket adapted to the present case. The holes a, a in the curved brackets are simply for ornamentation.

The same general type of frame will be used for the portal, as represented in Fig. 33. The brackets will extend 4 feet 6 inches out from the center of the trusses; the other dimensions shown in Fig. 33 are found from those shown in Fig. 32 by multiplying the latter by csc H (1.2445).

62. The force that acts on the portal is one-half the total wind pressure on the upper chord, or 16,330 pounds. Making use of the formulas given in *Stresses in Bridge Trusses*, Part 5, assuming the point of inflection of the end post to be half way between the bottom of the bracket and the bottom of the post, the required stresses are found to be as follows:

Direct stress in end posts,

$$\frac{P h'}{b} = \frac{16{,}30 \times 24.63}{28} = 14{,}360 \text{ pounds}$$

Bending moment on end posts,

$$\frac{P}{2}(h' - d - d') = 8{,}165 \times 9.8 \times 12 = 960{,}200 \text{ inch-pounds}$$

Direct stress in each web diagonal,

$$\frac{P h' l}{n b d} = \frac{16{,}330 \times 24.63 \times 11.12}{6 \times 28 \times 9.20} = 2{,}894 \text{ pounds}$$

The stress in the top flange is

$$\frac{P h'}{2 d} + \frac{P}{2} - \frac{P h' x}{b d}$$

At the section opposite D, $x = 4.5$, and the stress is

$$\frac{16{,}330 \times 24.63}{2 \times 9.20} + \frac{16{,}330}{2} - \frac{16{,}330 \times 24.63 \times 4.5}{28 \times 9.20}$$
$$= 23{,}000 \text{ pounds, compression}$$

At the section opposite D', $x = 23.5$, and the stress is

$$\frac{16{,}330 \times 24.63}{2 \times 9.20} + \frac{16{,}330}{2} - \frac{16{,}330 \times 24.63 \times 23.5}{28 \times 9.20}$$
$$= -6{,}640 \text{ pounds, tension}$$

The stress in the bottom flange at D is

$$\frac{16{,}330 \times 24.63}{2 \times 9.20} - \frac{16{,}330 \times 24.63 \times 4.5}{28 \times 9.20}$$
$$= 14{,}800 \text{ pounds, tension}$$

The stress in the bottom flange at D' is

$$\frac{16{,}330 \times 24.63}{2 \times 9.20} - \frac{16{,}330 \times 24.63 \times 23.5}{28 \times 9.20}$$
$$= 14{,}800 \text{ pounds, compression}$$

There is also a direct stress caused in the bottom chord of the main truss by the direct stress in the end post; the former is found by multiplying the latter by cos H ($20 \div 33.6 = .5952$); this gives $14{,}360 \times .5952 = 8{,}550$ pounds. This stress is tension on the leeward side, which is the only side that will be considered, since the compression on the windward side simply decreases the tension in the chord.

DESIGN OF MEMBERS

63. Upper Lateral Truss.—Comparing the wind stresses in the top chord members with the combined dead- and live-load stresses, it is seen that the former are less than 25 per cent. of the latter, and need not be further considered (Art. **47**).

The diagonals are usually composed of one angle; in the present case, the required net area in the end panel is $16{,}730 \div 16{,}000 = 1.05$ square inches. The net area of a $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angle, after deducting one $\frac{7}{8}$-inch rivet hole, is 1.2 square inches, which is sufficient. If this angle is used, however, the bending stress due to its own weight is greater than the allowable intensity of bending stress; so that a larger angle must be used. For this reason one $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angle will be used for each diagonal in each panel, the longer leg being placed vertical.

64. The rivets connecting the laterals to the connection plates are $\frac{3}{4}$-inch rivets, field driven, and in single shear, the value being 3,980 pounds. Then, the number of rivets required at each end, since the greatest stress is 16,730 pounds, is $16{,}730 \div 3{,}980 = 4.2$, or, say, 5 rivets. In the next two panels, three rivets and one rivet are sufficient, respectively, but five rivets will be used, the same as in the end panel.

65. The maximum stress in a transverse strut is shown in Fig. 29 to be 13,610 pounds. It is customary to design the upper flange of the frames shown in Fig. 32 to resist this stress, and to make the lower flange the same size as the upper. In *B. S.*, Art. **105**, it is specified that no lateral member shall have a ratio of $\frac{l}{r}$ greater than 120. In the present case, since l is 28 feet, it is advisable to insert one $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angle connecting the center of each transverse strut with the intersections of the diagonals, as shown by the dotted lines *a a* in Fig. 29, thereby reducing the value of l to 14 feet, or 168 inches. Then, the smallest allowable value of r is $168 \div 120 = 1.4$ inches. The top flange of the frame will be made of two angles, with their backs spaced $\frac{5}{16}$ inch apart. Consulting Table XXVI, it is seen that the smallest angles that can be used are two angles 3 in. $\times$ $2\frac{1}{2}$ in. $\times$ $\frac{5}{16}$ in., for which the radius of gyration is 1.44 inches when the short legs are placed $\frac{5}{16}$ inch apart. These angles are found to contain sufficient area.

66. Lower Lateral Truss.—In the design of the upper lateral truss, it was found that the smallest angle that can be used for a diagonal is $3\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. $\times$ $\frac{5}{16}$ in. The net area of such an angle, after deducting one $\frac{3}{4}$-inch rivet hole, is 1.51 square inches, and its value in tension is $1.51 \times 16{,}000 = 24{,}160$ pounds. This angle can be used in all but the end panel.

The number of rivets required to connect the angle in the panel *b c* at each end, since the stress is 23,040 pounds and the value of one rivet ($\frac{3}{4}$-inch rivet, field driven, and in single shear) is 3,980 pounds, is $23{,}040 \div 3{,}980 = 5.8$, or, say, 6 rivets. Six rivets will be used in the panel *b c*, and also, to make the laterals alike, in the panels *c d* and *d e*.

The tension in the diagonal in the panel *a b* is 32,260 pounds, and the required net area is $32{,}260 \div 16{,}000 = 2.02$ square inches. One angle 4 in. $\times$ 3 in. $\times$ $\frac{3}{8}$ in. will be used;

the net area, after deducting one $\frac{3}{4}$-inch rivet hole, is 2.15 square inches. The number of rivets required to connect the lateral at each end, since the value of one rivet is 3,980 pounds, is 32,260 ÷ 3,980 = 8.1, or, say, 9 rivets.

Lug angles will be placed on all the laterals. The end of the lateral that has just been designed is shown in Fig. 34.

67. It is unnecessary in this case to consider the wind stresses in the floorbeams, such as + 26,250 pounds in $b\,b'$; they usually increase the intensity of stress in the floorbeam by a very small amount, which can be neglected.

68. In comparing the wind stresses in the lower chord members with those due to dead and live loads, it is necessary to add the stress in the leeward chord that is caused by the direct stress in the end post (8,550 pounds, tension) to the stresses in the leeward chord members due to their positions as members of the lower lateral stress. In the present case, none of the wind stresses in the lower chord members is so great as 25 per cent. of those due to combined dead and live loads, and so they will not be further considered.

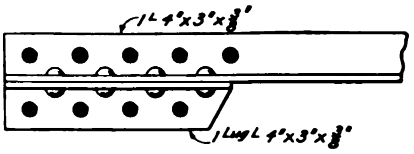


FIG. 34

69. Portal.—The direct stress in a lattice bar was found in Art. **62** to be 2,894 pounds. As the stress in any one of these bars is compression when the wind comes from one direction, and tension when it comes from the other, each must be designed for 2,894 + .8 × 2,894 = 5,209 pounds, tension and compression (*B. S.*, Art. **107**). Flat bars are sometimes used for the diagonals, but when the depth of portal is greater than about 3 feet it is better to use small angles, as they give greater stiffness to the web. In the present case, one 2″ × 2″ × $\frac{1}{4}$″ angle will be used for each diagonal. This is smaller than the smallest allowable angle, as given in *B. S.*, Art. **113**, but, since this may be considered an unimportant detail, it will be used. Each end will be connected by two $\frac{5}{8}$-inch rivets. The same-sized angles will be used for the lattice web of the transverse struts.

The stress in the top flange was found in Art. 62 to be 23,000 pounds compression at one end, and 6,640 pounds tension at the other end. When the wind blows in the opposite direction, these stresses are reversed. The top flange should therefore be designed for $23{,}000 + .8 \times 6{,}640 = 28{,}310$ pounds compression, and $6{,}640 + .8 \times 23{,}000 = 25{,}040$ pounds tension. The center of the top flange will be held by one $2\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angle in the same way as the transverse frames, as shown at *b*, Fig. 29, thereby reducing the unsupported length to 14 feet, or 168 inches. The smallest allowable value of r is, then, $168 \div 120 = 1.4$ inches. The smallest angles that can be used are two $3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angles, for which the radius of gyration is 1.44 inches. These angles are found to have sufficient area, and so will be adopted.

The stress for which the bottom flange should be designed is $14{,}800 + .8 \times 14{,}800 = 26{,}600$ pounds tension and compression. The same-sized angles will be used here as for the top flange, that is, two $3'' \times 2\frac{1}{2}'' \times \frac{5}{16}''$ angles, with the $2\frac{1}{2}$-inch legs $\frac{5}{16}$ inch apart and the 3-inch legs outstanding.

There is no need to design the bracket. It is customary to use a web about $\frac{5}{16}$ inch thick and angles on three sides about the same size as those in the flanges of the portal.

70. Effect of Wind Stresses on End Post.—On account of the bending moment on the end post, it is not sufficient to see that the direct stress due to the wind does not exceed 25 per cent. of the combined dead- and live-load stresses, but it must be seen in every case that the maximum intensity of stress due to the combined direct and bending stresses does not exceed the allowable working stress by more than 25 per cent. The formula used in the determination of the maximum intensity of stress s is

$$s = \frac{S}{A} + \frac{Mc}{I}$$

in which S = combined dead, live, and wind direct stresses;
A = gross area of member;
M = bending moment due to wind;
c = distance of extreme fiber from center of end post, measured at right angles to truss;
I = moment of inertia of cross-section of end post about an axis parallel to webs and half way between them.

In the present case, S = 272,100 (Art. **49**) + 14,360 (Art. **62**) = 286,500 pounds, and M = 960,200 inch-pounds (Art. **62**). For the section that was decided on in Art. **56**, A = 24.3 square inches, $c = 6 + \frac{5}{16} + 3\frac{1}{2} = 9\frac{13}{16}$ inches, and I = 1,108. Then, the actual intensity of stress s is

$$\frac{286{,}500}{24.3} + \frac{960{,}200 \times 9.81}{1{,}108} = 20{,}300 \text{ pounds per square inch}$$

The allowable working stress in the end post, without considering the effect of the wind, is given in Art. **56** as 11,760 pounds per square inch. When the wind is considered, the allowable working stress is 25 per cent. greater than this, or 11,760 + .25 × 11,760 = 14,700 pounds per square inch. As this is much less than the actual intensity of stress, 20,300 pounds per square inch, it is necessary to increase the section of the end post. The following section will be tried:

	Square Inches
Two web-plates 15 in. × $\frac{7}{16}$ in.	13.12
Four angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{1}{2}$ in.	13.00
Two side plates 8 in. × $\frac{1}{2}$ in.	8.00
Total	34.12

The least radius of gyration of this section is 5.03 inches, and since the unsupported length of the end post is 403.2 inches, the value of $\frac{l}{r}$ is 80.2. By Table XXXV, the allowable working stress for combined dead- and live-load stresses is 11,790 pounds per square inch. Increasing this 25 per cent. gives 14,740 pounds per square inch as the allowable working stress for combined dead, live, and wind stresses. The clear distance between webs will be made $11\frac{3}{4}$ inches, or $\frac{1}{4}$ inch less than the top chord; this will bring the vertical legs of the flange angles $11\frac{3}{4} + \frac{7}{16} + \frac{7}{16} = 12\frac{5}{8}$ inches apart, the same as in the top chord, and the extreme fiber $6.31 + 3.5 = 9.81$ inches from the center of the section. Then, the moment of inertia about an axis parallel to the web is 1,553. Substituting the new values in the formula gives

$$s = \frac{286{,}500}{34.12} + \frac{960{,}200 \times 9.81}{1{,}553} = 14{,}470 \text{ pounds per square inch}$$

As this is less than the allowable working stress, 14,740 pounds per square inch, the section that has been tried is

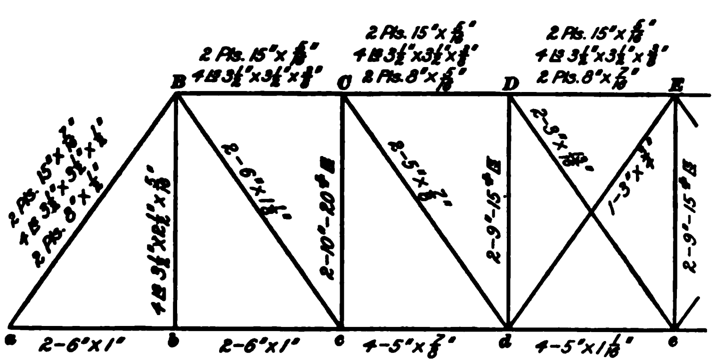


FIG. 35

sufficient, and will be adopted. It is not necessary to compute the shearing stress caused in the end post by the wind when the bending moment is considered.

71. The sections that have been decided on for the various members of the vertical trusses are, for convenience, shown on the members in Fig. 35.

DESIGN OF A HIGHWAY TRUSS BRIDGE

(PART 2*)

DETAILS

FLOOR CONNECTIONS

1. Cross-Section.—The location of the top of the floor with respect to the top of the floorbeam is shown in Fig. 1. It is customary to keep the top flange of the floor beam as high as it is possible to have it without interfering with the floor plank. This can be done by so placing the stringers *d*, *e*, and *f* under the roadway that their top flanges are very near the top of the floorbeam, and making the nailing pieces *i* on top of the stringers *d* at the edge of the roadway of sufficient thickness to bring the bottom of the floor plank at the lowest point *j* above the top of the floorbeam.

In Fig. 1, the three roadway stringers *d*, *e*, and *f* are placed with their tops 1 inch below the top of the floorbeam, and the top of the wheel-guard is placed 1 foot above the top of the floorbeam. This makes the nailing piece on stringer *d* about 2 inches thick, and brings the bottom of the 3-inch plank at *j* about 1 inch above the top of the floorbeam. As the sidewalk plank rises ½ inch per foot toward the railing, the distance from the top of the sidewalk bracket (level with

*The abbreviation *B. S.* stands for *Bridge Specifications*, to which Section frequent reference is made in this and in some of the following Sections. All tables referred to are found in *Bridge Tables*, unless otherwise stated.

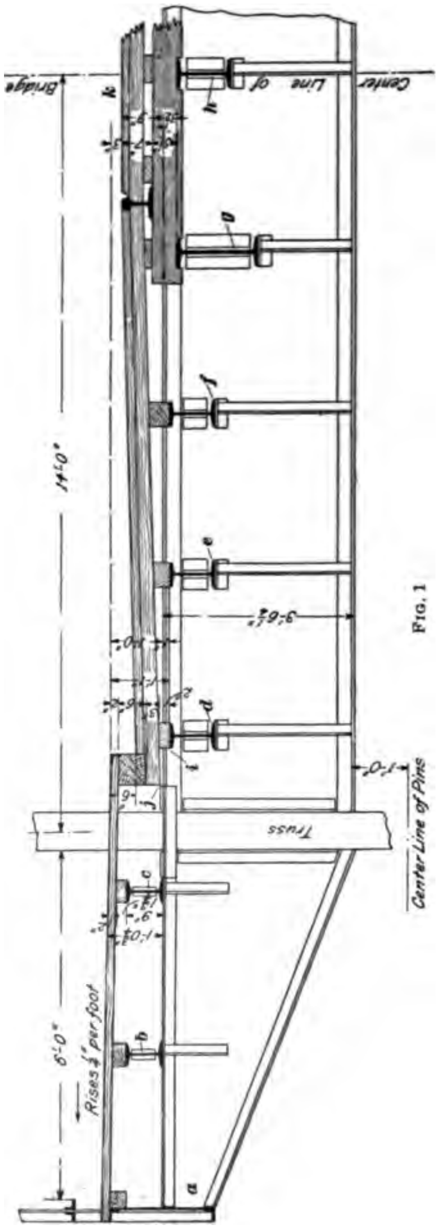


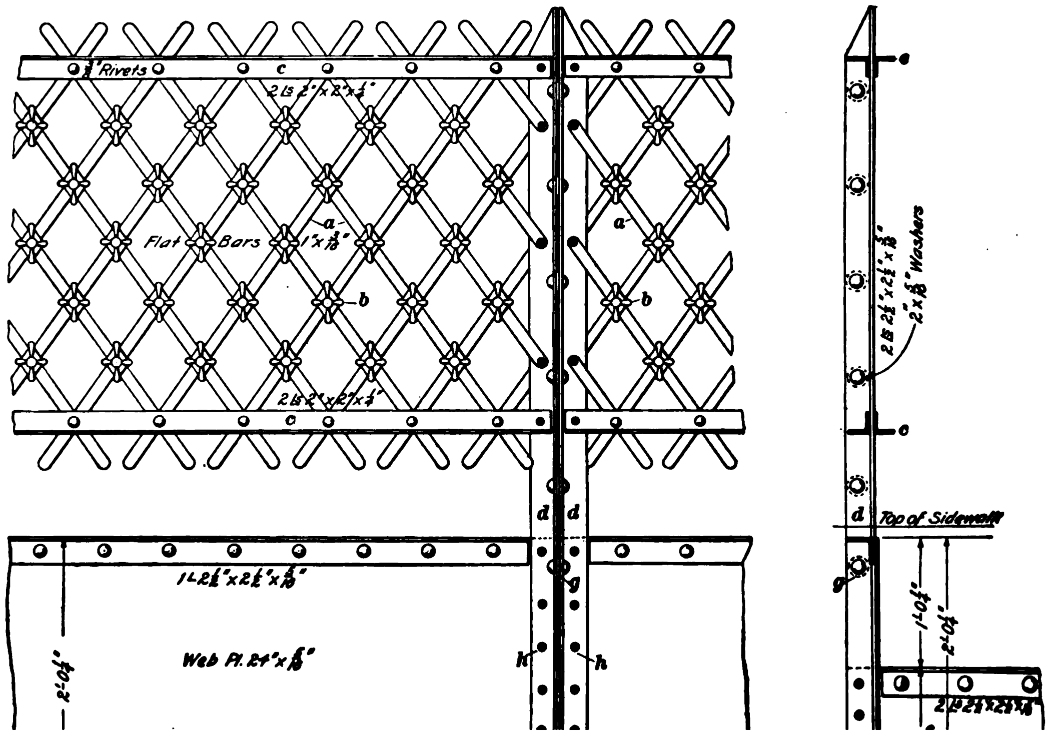
FIG. 1

the top of the floor-beam) to the top of the sidewalk plank at stringer *c* is about 12¾ inches; this makes it possible to place the sidewalk stringers *b* and *c*, 9 inches deep, on top of the top flange of the bracket, leaving room for a nailing piece about 1¾ inches in thickness on top of stringer *c*.

2. The location of the tops of the stringers *g* and *h* under the car track depends on the crown of the roadway and on the depth of rail and tie. It has been stated (*Design of a Highway Truss Bridge*, Part 1) that the wheel-guard would be placed 9 inches above the floor at the edge of the roadway, and the floor given a crown of 3 inches, bringing the top of the floor at the center of the bridge (level with the top of the rail) 3 inches below the wheel-guard; since the latter is 12 inches above the top of the floorbeam, the top of the rail is 9 inches above the top of the floorbeam, as shown in Fig. 1. If the rails are 7 inches high, and the ties are framed to $5\frac{1}{2}$ inches in depth where they bear on the stringers, the tops of the stringers will be $7 + 5\frac{1}{2} = 12\frac{1}{2}$ inches below the top of the floor, and $12\frac{1}{2} - 9 = 3\frac{1}{2}$ inches below the top of the floorbeam.

3. Connection of Fence and Fascia Girders to End of Bracket.—The detail of the connection of the fence and fascia girders to the end of the sidewalk bracket is shown in Fig. 2: (*a*) is the elevation of the fence and fascia girder, and (*b*) is the cross-section of fence and fascia girder and elevation of end of bracket. This is a very popular and serviceable type of fence; it consists of a number of flat bars *a*, *a* crossing each other at intervals to form a lattice web and held at their intersections by small ornamental castings *b*. Near the top and bottom of the fence are pairs of longitudinal angles *c*, *c*, about 2 in. × 2 in. × $\frac{1}{4}$ in. The fence posts *d*, *d* are usually composed of two angles $2\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × $\frac{5}{16}$ in., and enclose the web *e* at the end of the bracket; the fence and fascia girders are riveted to the outstanding legs of the fence angles, as shown at (*a*). The rivets are $\frac{3}{4}$ inch in diameter and spaced about $2\frac{1}{2}$ inches apart at *f*, the connection of the fence-post angles to the web of the bracket; above the bracket, the angles are connected to each other by means of a rivet every 10 or 12 inches, a washer *g* of the same thickness as the web *e* being inserted between the angles at each rivet. The rivets *h* in the connection of the fascia girder to the fence post are spaced about $4\frac{1}{2}$ inches apart; those in the top and bottom flanges of the fascia girder are spaced about 6 inches apart. In locating the rivet holes for the connection of the fence posts to the bracket, the clear width of sidewalk can be made just 6 feet, as required.

4. Connection of Sidewalk Stringers to Brackets. The sidewalk stringers *b* and *c*, Fig. 1, are made about $\frac{1}{4}$ inch shorter than the panel length, so as to leave some clearance between the ends of the stringers over the bracket. The ends of the stringers are connected to each other, as shown in Fig. 3, by means of two plates *d*, $\frac{5}{16}$ inch thick, riveted to



3/4" Rivets
2 Ls 2" x 2" x 1/4"
Flat Bars 1" x 3/16"
1 L 2 1/2" x 2 1/2" x 5/16"
Web Pl. 24" x 5/16"
2'-0 1/2"
Top of Sidewalk
2 Ls 2 1/2" x 2 1/2" x 5/16"
2" x 5/16" Washers
1'-0 1/2"
2'-0 1/2"

the webs of both beams. In the figure, (*a*) is the cross-section of the top flange of the bracket and the elevation of the adjacent ends of the stringers; (*b*) is a longitudinal section on the line *cc*, and shows the plan of the bottom flanges of the stringers and the top flange of the bracket. The lower flanges of the stringers are riveted to the upper flanges of the brackets, as shown at *e*. Under each sidewalk stringer, a pair of stiffener angles *f*, $2\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × $\frac{5}{16}$ in., and about 15 inches long, are riveted to the web of the bracket. The reaction from the stringers is so small that it is not necessary to calculate the number of rivets required in the stiffener; it is customary to place five or six rivets in each.

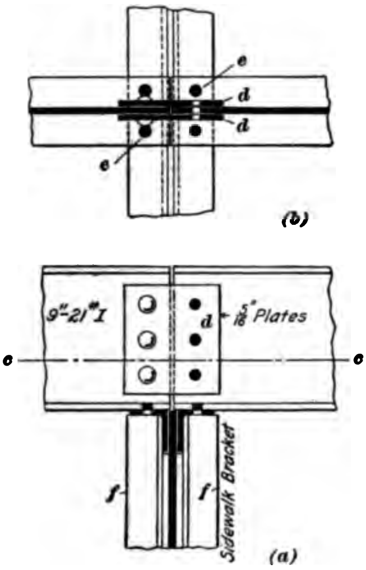


FIG. 3

5. Connection of Roadway Stringers to Floorbeam. The roadway stringers *d*, *e*, and *f*, Fig. 1, are riveted to the web of the floorbeam by means of connection angles *d*, Fig. 4; and short shelf angles *e*, *e*, and stiffeners *f*, *f* are placed under them. In Fig. 4, (*a*) is the cross-section of a part of the top of the floorbeam and an elevation of the ends of two stringers resting on the shelf angles; (*b*) is a longitudinal section on *cc* and a plan of the bottom flanges of the stringers. The upper parts of the stringers *i*, *i* are cut back to clear the vertical legs of the floorbeam flange angles *j*, and the connection angles *d* are placed below the flange angles.

With this kind of connection, it is customary to assume that the rivets in the connection angles transmit the entire

reaction (end shear on the stringer) to the floorbeam, ignore the supporting effect of the shelf angles and eners; the shelf angles and stiffeners, however, add con ably to the str of the connec and should alwa inserted when is room for under the strin

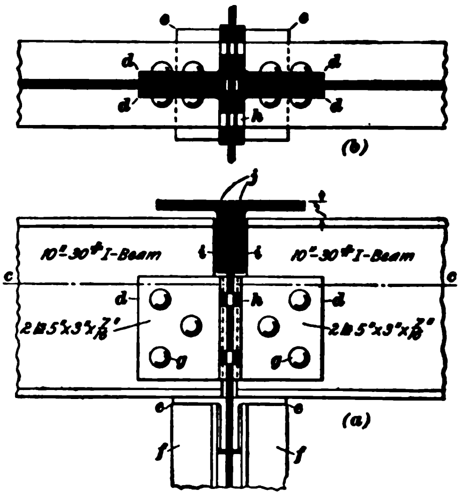


FIG. 4

6. In *Desi a Highway Bridge*, Part 1, i found that the mum end shear i of the roadway s ers is 8,325 po The rivets *g*, F that connect th gles to the we the stringers are $\frac{3}{4}$ inch in diameter, shop driven, in d shear, and in bearing both on the angles and on the w the I beam, which has a thickness of .45 inch (Table The bearing value of the web is the smallest; as the are $\frac{3}{4}$ inch in diameter and the allowable bearing str 22,000 pounds per square inch (*B. S.*, Art. **103**), the is $\frac{3}{4} \times .45 \times 22{,}000 = 7{,}425$ pounds. Then, the rec number of rivets is $8{,}325 \div 7{,}425 = 1.1$; three rivets w used to connect each pair of connection angles to th of the stringer (*B. S.*, Art. **155**).

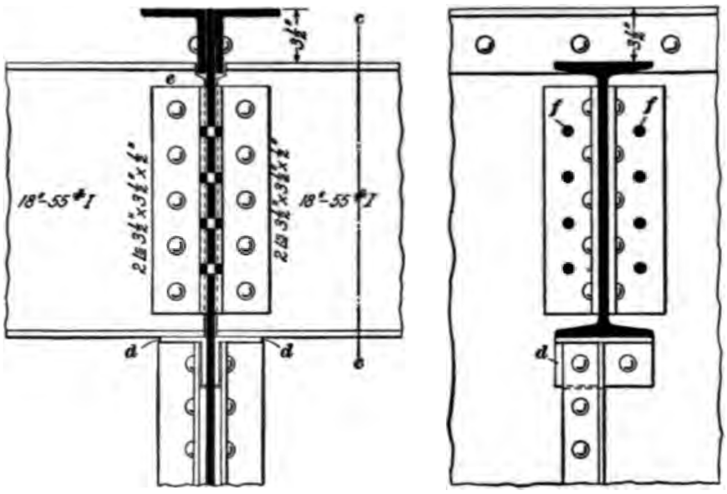
To find the number of rivets *h* required to conne connection angles to the web of the floorbeam, two must be considered. These rivets are field driven, a bearing both on the connection angles and on the w the floorbeam. When the load in one panel is consi the amount of stress transmitted to the floorbea 8,325 pounds, and the rivets are in single shear. The of the rivets for the allowable stresses given in

Art. **103**, are given in Table XXXIX. The value in single shear is the smallest, and is 3,980 pounds. Then, the required number of rivets for this condition is 8,325 ÷ 3,980 = 2.1. When the load in two adjacent panels is considered, the amount of stress transmitted to the floorbeam is 2 × 8,325 pounds = 16,650 pounds, and the rivets are in double shear. In this case, however, the bearing value of a rivet on the $\frac{3}{8}$-inch web of the floorbeam is the smallest value, and is 5,060 pounds. Then, the required number of rivets in this case is 16,650 ÷ 5,060 = 3.3. The next larger even number will be used, in this case four.

According to *B. S.*, Art. **126**, the connection angles cannot be smaller than 3 in. × 3 in. × $\frac{7}{16}$ in. If a leg 3 inches wide is placed next to the web of the stringer, it will be impossible to get three rivets in it without spacing them too close together; so a leg 5 inches wide will be used, and the connection angles will be made 5 in. × 3 in. × $\frac{7}{16}$ in. The shelf angles will be made 3 in. × 3 in. × $\frac{5}{16}$ in., and the stiffeners will be made $2\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × $\frac{5}{16}$ in. These are usually put in according to the judgment of the designer, and not by rule.

7. Connection of Stringers Under Railway Track. The stringers marked *g* in Fig. 1 are connected to the floorbeam in the same way as those just considered. The connection is shown in Fig. 5, in which (*a*) is the cross-section of the upper part of the floorbeam, and shows the elevation of the ends of two stringers, resting on shelf angles *d*, *d*; (*b*) is a cross-section of one of the stringers on the section *cc*, and shows the elevation of a part of the floorbeam. The stringers are cut back at *e* to clear the top flange angles of the floorbeam. In *Design of a Highway Truss Bridge*, Part 1, it was found that the maximum end shear in the stringer *g* is 23,340 pounds. The web of the I beam is .46 inch thick, and the value of one $\frac{3}{4}$-inch rivet that connects the angles to the stringer is .75 × .46 × 22,000 = 7,590 pounds. The required number of rivets is, then, 23,340 ÷ 7,590 = 3.1, or, say, 4.

To determine the number of rivets *f* required to connect the connection angles to the web of the floorbeam, two cases must be considered. These rivets are field driven, and in bearing both on the connection angle and on the web of the floorbeam. When the load in one panel is considered, the amount of stress transmitted to the floorbeam is 23,340 pounds, and the rivets are in single shear (value = 3,980 pounds). The required number of rivets for this condition is 23,340 ÷ 3,980 = 5.9, or, say, 6. When the load in two adjacent panels is considered, it is necessary to



(a) FIG. 5 (b)

compute the amount of load that is transmitted to the floorbeam. This is found in the same way as in *Design of a Highway Truss Bridge*, Part 1, except that, since stringer connections are under consideration, it is necessary to allow for the increase due to the overturning effect of the wind. The amount of dead load is 3,600 pounds, and of live load 15,384 pounds. The allowance for impact and vibration is $\frac{3}{10} \times 15,384 = 4,615$ pounds, and for the overturning effect of the wind, 4,615 pounds. Then, the total load is 28,215 pounds. The value of a rivet in bearing on the ⅜-inch floorbeam web (5,060 pounds) is the smallest; and the required number of rivets for this condition is $28{,}215 \div 5{,}060 = 5.6$, or, say, 6, as before. It is well, when possible, to add one or two rivets to the required number in stringers under railway tracks, and to use good-sized connection angles. For this reason, five and eight rivets, respectively, will be used, as shown in Fig. 5, and the connection angles will be made $3\frac{1}{2}$ in. $\times$ $3\frac{1}{2}$ in. $\times$ $\frac{1}{2}$ in., which is slightly larger than the smallest allowable connection angle (*B. S.*, Art. **126**).

8. Connection of Center Stringer.—The same connection will be used for the stringer marked *h*, Fig. 1, as for *g*, except that in the connection angles only four rivets will be driven in the legs that are in contact with the stringers, and six rivets in the legs that are in contact with the floorbeam web.

9. Connection of Bracket and Floorbeam to Truss. The connection of the bracket and that of the floorbeam to the truss are shown in Fig. 6, in which (*a*) is an elevation of the ends of the bracket and floorbeam that connect to the truss; (*b*) is a cross-section on *D D*; and (*c*) is a top view of the bracket and floorbeam. The floorbeam is connected to one side of the vertical post *E E*, and the bracket to the other. When, as in this case, the vertical post consists of two channels, a diaphragm *f*, consisting of a web and two angles at each side, is riveted between the channels. The leg of the angle adjacent to the web of the diaphragm is made $2\frac{1}{2}$ inches wide; the other is made the same width as the outstanding leg of the floorbeam connection angle.

10. The end shear on the floorbeam was found in *Design of a Highway Truss Bridge*, Part 1, to be 54,450 pounds. The value of one of the rivets *g* that connect the connection angle *h* to the web of the floorbeam ($\frac{3}{4}$-inch rivet, shop driven, in bearing on $\frac{3}{8}$-inch web) is 6,190 pounds. Then, the required number is $54{,}450 \div 6{,}190 = 8.8$, or, say, 9. The value of one of the rivets *i* that connect the angles *h* to the truss ($\frac{3}{4}$-inch rivet, field driven, in single shear) is 3,980 pounds. Then, the required number is $54{,}450 \div 3{,}980 = 13.7$, or, say 14. In the leg of the connection angle adjacent to the web, ten rivets *g* will be used; in the other leg, a rivet *i* will be

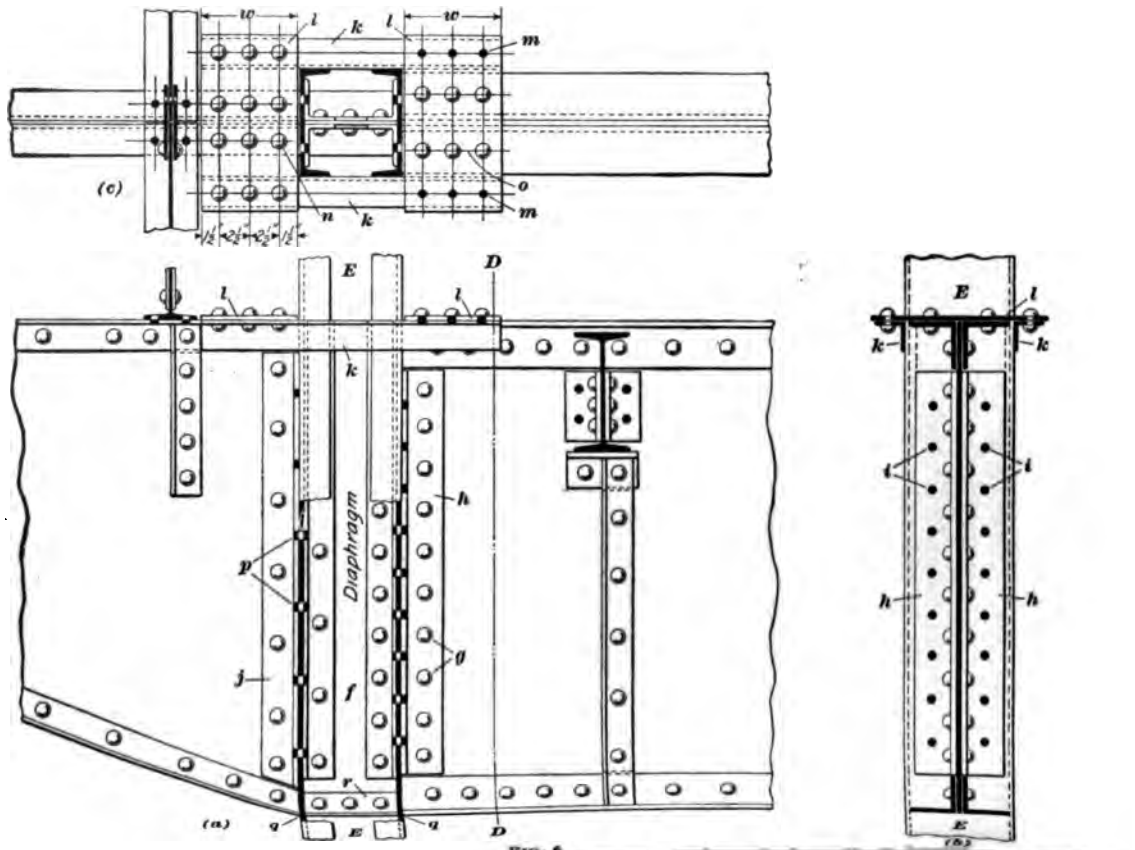


FIG. 6

placed half way between each two of the former, making nine rivets in each of the outstanding legs. It is customary, as in this case, to have a few extra rivets. For the floorbeam connection angles, angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{1}{2}$ in. will be used.

11. The shear on the bracket where it connects to the truss is 16,240 pounds (*Design of a Highway Truss Bridge*, Part 1). The number of rivets required in the leg of the angle *j* that is adjacent to the web of the bracket is 3.1, and in the outstanding legs, 4.1. The rivets in these connection angles, which will be made $3\frac{1}{2}$ in. × 3 in. × $\frac{7}{16}$ in., will be spaced as far apart as allowable, that is, 6 inches apart, thereby bringing seven rivets through the web and twelve through the vertical post.

12. Connection of Bracket to Floorbeam.—To assist in making the bracket and floorbeam act as a single beam, the top flanges should be connected to each other. A good method of connecting them, when the bracket and floorbeam are attached to different sides of a vertical post, is shown in Fig. 6. Two angles *k*, *k*, the same size as those in the top flange of the bracket, are placed close to the vertical post, one angle on each side, and their ends are connected to the ends of the bracket and floorbeam flange angles by means of plates *l*, *l*.

The stress transmitted by the angles *k* is found by dividing the greatest negative bending moment on the bracket by the depth of the bracket at the truss. In the present case, the stress is 802,800 ÷ 42 = 19,100 pounds, tension, and, as there are two angles, each transmits 9,050 pounds. The value of one of the rivets *m* that connect the end of an angle *k* to the plate *l*, Fig. 6 (*c*) ($\frac{3}{4}$-inch rivet, field driven, in single shear) is 3,980 pounds; then, the required number of rivets is 9,050 ÷ 3,980 = 2.3, or, say, 3. The same number is used to connect the other ends of the angles and to connect the plates to the top flanges at *n* and *o*.

The required thickness *t* of the plate *l* is given, approximately, by the formula

$$t = \frac{4\,S\,d}{s\,w^2}$$

in which S = stress transmitted by one angle k;
d = width of widest vertical member to which floorbeams are connected—that is, the clear distance between the angles k;
s = working stress in bending;
w = width of plate l, measured along bracket.

The width w is controlled by the rivet spacing in the plate; in the present case, it is 8 inches. Then,

$$t = \frac{4 \times 9{,}050 \times 10}{16{,}000 \times 8^2} = .354, \text{ or, say, } \tfrac{3}{8} \text{ inch}$$

The angles k and plates l are frequently omitted, and the rivets p that connect the bracket to the vertical post are depended on to transmit the stress in the bracket. This construction is not recommended, as it tends to overstrain the rivets and allow the brackets to sag at the ends.

The bottom flanges of the bracket and floorbeam are not connected, but are made to bear against the outsides of the vertical posts, the stress being transmitted from one to the other by means of angles r that are riveted to the bottom of the diaphragm.

13. Connection of End Floorbeam to Truss.—As there is no vertical member at the end joint, the end floorbeam cannot be connected to the truss in the same way as the other floorbeams. It will be supported on a chair riveted to the end post. The detail of the chair will be treated later, in connection with the design of the pin at the joint a. The portion of the end floorbeam directly over the truss is shown in Fig. 7, in which (a) is the elevation and (b) the cross-sec- tion of the floorbeam; the latter view shows also the relativ location of floorbeam, end post, and bottom chord. In orde that the bottom flange angles of the floorbeam shall n interfere with the top flange angles of the end post at e, is necessary to place the bottom of the floorbeam 1 fo 8 inches above the center line of the bottom chord, makin the floorbeam 8 inches shallower than the intermediate floo beams, that is, 34¼ inches, as assumed in *Design of a Hig way Truss Bridge*, Part 1.

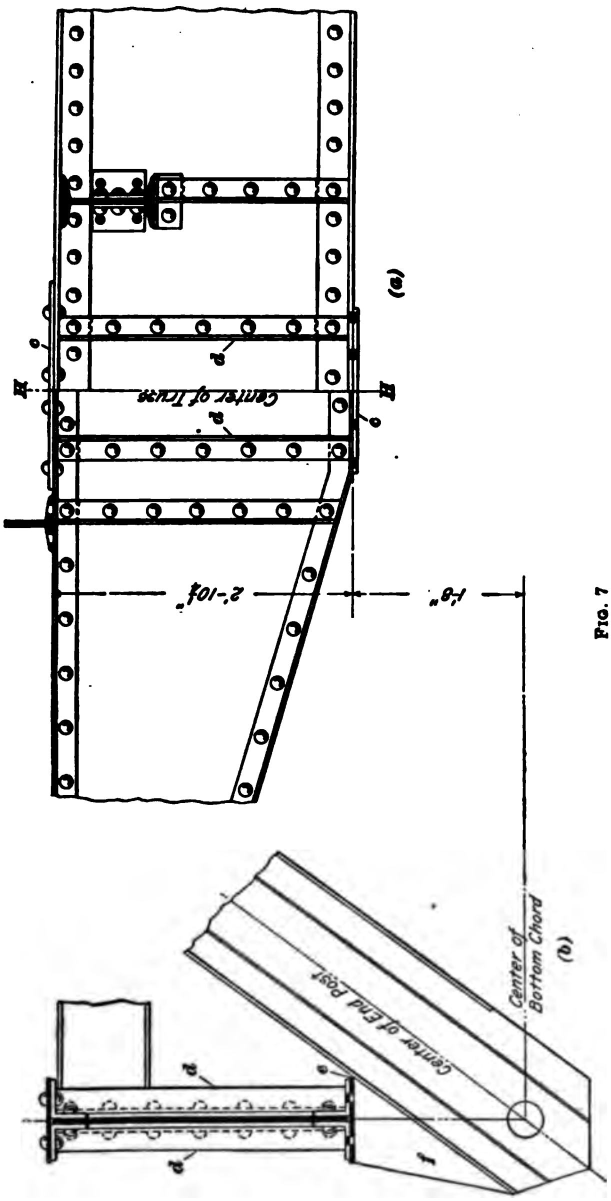

Center of Truss
2'-10½"
1'-6"
(a)
Center of End Post
Center of Bottom Chord
(b)

FIG. 7

The web of the bracket and floorbeam is in one piece extending the full length. Plates *c* having the same area as the flange angles of the bracket are riveted to the top and bottom flanges to splice the bracket flange angles to the floorbeam flange angles. Stiffeners *d* are riveted to the web where the beam rests on the chair at the truss. The stringers are connected to the end floorbeam and brackets in the same way as to the other floorbeams and brackets, except that there is a shelf angle and stiffener on but one side of the web.

SPLICES

14. The only splices that are necessary are in the top chord. In *Design of a Highway Truss Bridge*, Part 1, it was decided that the splices would be located in the panels CD and $D'C'$ close to the joints D and D', respectively. The form of cross-section used for the member CD is shown in Fig. 27 of the Section just referred to. This member will be

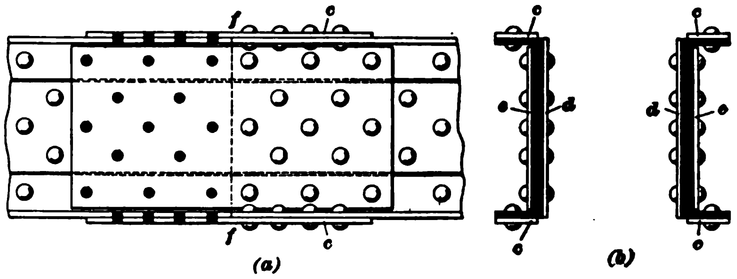


FIG. 8

spliced by means of four bars 3½ inches wide riveted to the outstanding legs of the flange angles, and by vertical splice plates inside and outside of the section, as shown in Fig. 8. The following splice plates will be used:

	Square Inches
Four plates 3½ in. × $\frac{7}{16}$ in., gross area =	6.125
Two plates 15 in. × $\frac{5}{16}$ in., gross area =	9.375
Two plates 14 in. × $\frac{5}{16}$ in., gross area =	8.75
Total gross area =	24.250

This section is slightly less than that of CD, but, as it is so little less, and as it is greater than the required area found for CD, these splice plates will be used. The rivets in the splice will be made $\frac{7}{8}$ inch in diameter. Each splice plate must have sufficient rivets on each side of the splice to transmit its stress to and from the ends of the members.

It is customary to rivet the splice plates to the end of one member in the shop, and connect the other end in the field. The required number of field rivets in a splice plate is found by dividing the product of the area of the plate and the working stress by the value of one rivet. In the present case, the working stress in CD, as found in Part 1, is 14,290 pounds per square inch. The smallest value of a rivet in this case is its value in single shear, or 5,410 pounds. Then, the number of field rivets required to connect each splice plate is as follows:

One plate $3\frac{1}{2}$ in. $\times \frac{7}{16}$ in., $\dfrac{1.531 \times 14{,}290}{5{,}410} = 4.05$, or, say, 4

One plate 15 in. $\times \frac{5}{16}$ in., $\dfrac{4.687 \times 14{,}290}{5{,}410} = 12.4$, or, say, 13

One plate 14 in. $\times \frac{5}{16}$ in., $\dfrac{4.375 \times 14{,}290}{5{,}410} = 11.5$, or, say, 12

As the value of a shop-driven rivet is greater than that of a field-driven rivet (*B. S.*, Art. **103**), fewer shop-driven rivets are required, but it is customary to place the same number of rivets on each side of the joint.

15. The splice is shown in Fig. 8, in which (*a*) is an elevation and (*b*) a cross-section of the top chord. The $3\frac{1}{2}$-inch splice plates are marked *c*, the 15-inch splice plates are marked *d*, and the 14-inch splice plates are marked *e*. The dotted line *ff* is the joint; the ends of the sections of chord are planed smooth on the line *ff* so that the ends will bear against each other and so transmit some of the stress. Some engineers depend on this bearing to transmit the entire stress in compression members, putting just enough rivets in the splice plates to hold the members in line. This is not the best practice; each splice, whether in tension or in compression, should be designed as stated, and sufficient section provided in the splice plates and sufficient rivets to fully transmit the stress. In some cases, tie-plates are made to serve as splice plates, instead of the $3\frac{1}{2}$-inch plates on the outstanding legs of the flange angles. In this case, the whole width of tie-plates should not be counted as splice plates, but only that portion in contact with and close to the outstanding legs of the flange angles.

DESIGN OF PINS AND PIN PLATES

16. Bending Moment on Pins.—In *Bridge Members and Details*, Part 2, it was stated that pins are designed to resist the greatest bending moment to which they are subjected. The bending moments on the pins are found from the stresses in the members that connect to the pins. In order to find the maximum bending moment on a pin, it is customary to compute the moment for several conditions of loading; for example, it may first be computed when the stresses in the web members that connect to the pin are greatest, that is, when there is a partial live load on the truss; and then computed when the stresses in the chord members that connect to the pin are greatest, that is, when there is a full live load on the truss. For each of these cases it is necessary to calculate the stresses that occur *simultaneously* in the other members; for example, when the chord stresses are greatest, it is necessary to compute the stresses that obtain in the web members when there is a full live load on the truss, and to use these stresses in finding the bending moment on the pin for that loading. Similarly, when the maximum stresses in the web members are considered, it is necessary to compute the stresses in the chord members for the loading that causes the greatest stresses in the web members.

In finding the forces that act on a pin, it is usually assumed that the stress in each member is evenly distributed among the several portions of the member bearing on the pin, and that the stress in each portion acts as a concentrated load at its center of bearing.

17. In finding the bending moment on a pin, it is convenient to resolve the forces that act on the pin into their vertical and horizontal components, and then to find, at the section where the moment is desired, the bending moments M_V and M_H, due to the vertical and to the horizontal forces, respectively. The resultant bending moment M on the pin at any section is then found by means of the formula

$$M = \sqrt{M_V^2 + M_H^2}$$

When the largest value of M has been determined, the pin having a resisting moment as great as M is found from Table XLI. How the details of this process are worked out will be explained presently.

Many designers find the largest pin required and make the others the same size. This simplifies the work in the shop somewhat, but there is no other reason for doing it.

The bending moments can be found by the graphic as well as by the analytic method. Whichever method is used, it is necessary to thoroughly understand the steps that are taken before the bending moment can be found. These steps can best be explained and illustrated in connection with the analytic method. For this reason, the bending moments on the pins discussed in the following pages are found by means of the analytic method. They can also be found by the graphic method by applying the principles explained and illustrated in *Graphic Statics*.

18. Width of Top Chord.—Before designing the pins, it is necessary to decide definitely on the distance between

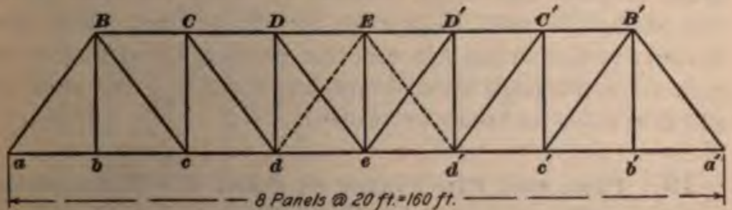


FIG. 9

the webs of the top chord. In *Design of a Highway Truss Bridge*, Part 1, this was assumed as 12 inches, but no calculation was made to verify the assumption. In a bridge of this kind, it is best to consider first the second joint of the top chord, in this case joint *C*, Fig. 9. The arrangement of the members at *C* is shown in Fig. 10, in which (*a*) is the elevation of the joint, and (*b*) is a cross-section of the top chord showing the relative positions of the vertical *Cc* and the diagonal *Cd*.

The width of the vertical has been taken as $8\frac{1}{2}$ inches (Part 1). If pin plates are required, they will be placed on the insides of the channels, as shown at *e*, Fig. 10, and the heads of the rivets that connect them to the channels will be flattened to a height of $\frac{3}{8}$ inch on the outside of the channel. This makes the vertical $8\frac{1}{2} + \frac{3}{8} + \frac{3}{8} = 9\frac{1}{4}$ inches wide over the rivet heads. The eyebars *f*, *f* that form the diagonal *Cd* are placed outside the vertical, and as close to it as possible. It is customary to allow $\frac{1}{16}$ inch between the eyebars and other members or rivets heads, so in this case their inner surfaces will be $9\frac{3}{8}$ inches apart; and since they are $\frac{7}{8}$ inch in thickness, their outer surfaces will be $9\frac{3}{8} + \frac{7}{8} + \frac{7}{8} = 11\frac{1}{8}$ inches apart. Allowing $\frac{1}{16}$ inch on each side for clearance, and $\frac{3}{8}$ inch for the height of the flattened rivet heads on the inside of the chord, gives $11\frac{1}{8} + \frac{1}{16} + \frac{1}{16} + \frac{3}{8} + \frac{3}{8} = 12$ inches for the distance between the webs of the chord, as shown in Fig. 10 (*b*).

In the following pages, the pins and pin plates will be designed. In the first joint, the diameter of the pin will be assumed at random, and the required diameter found by the usual method, that is, by successive trials. The same general method is used in practice for all the other joints; but in order to economize space in this Section, the diameter of each pin as obtained after several trials will be given at once, and this value will then be verified.

19. Pins and Pin Plates at Joint *C*.—The stresses in the members that meet at *C* are shown in Fig. 11. They are the *maximum* stresses in these members, and are not *simultaneous;* that is, the stress in the diagonal *Cd* is not a maximum when those in the chord members are greatest,

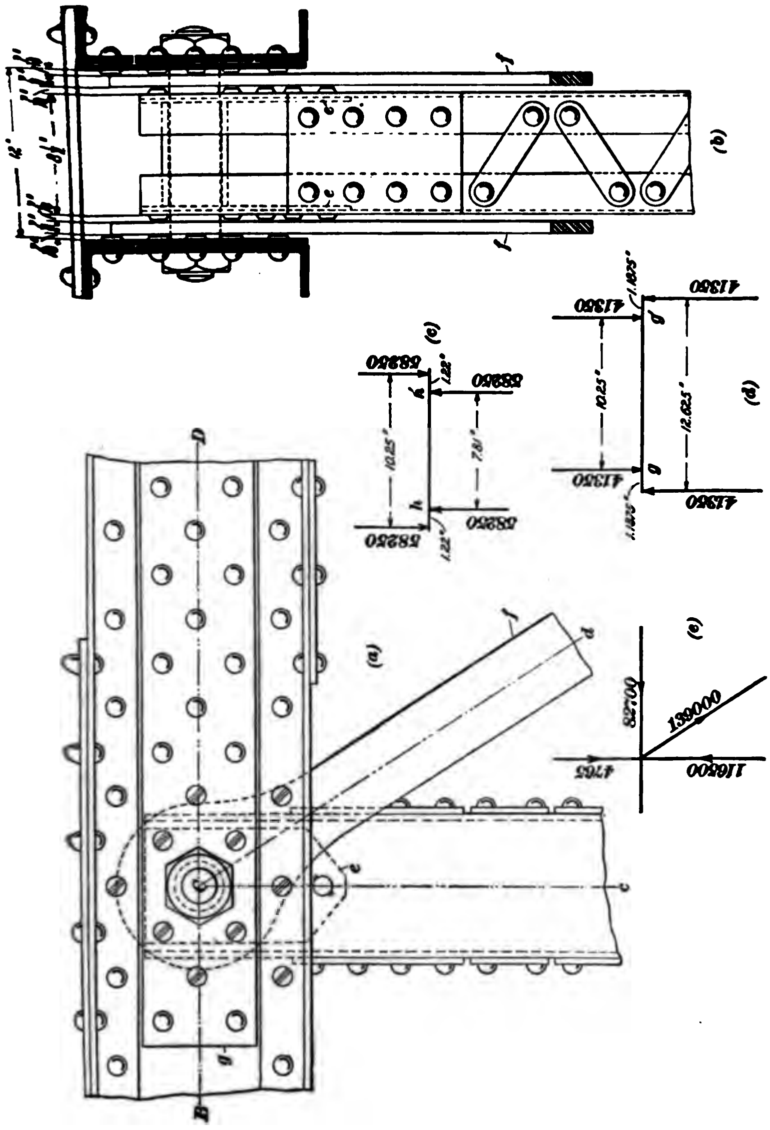

(a)
(b)
(c)
(d)
(e)
58850
10.25"
7.81"
1.22"
41360
12.625"
1.1875"
82700
4765
139000
116600

FIG. 10

and vice versa. In the design of top-chord pins, however, it is customary to ignore the maximum stresses in the chord members, as a part of the stress is transmitted from one panel to the next without passing through the pin. It is simply necessary to ascertain in each case the greatest amount of stress transmitted to the chord at the joint; in the present case, this is equal to the horizontal component of the stress in Cd, and is found by multiplying the maximum stress in Cd by cos H; in this case, it is $139{,}000 \times .595 = 82{,}700$. Then, the maximum stress in the vertical being known, the maximum forces acting on the pin are as shown

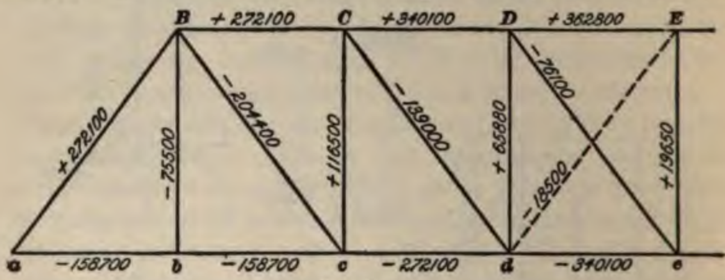


FIG. 11

in Fig. 10 (e). The dead panel load of 4,765 pounds is distributed over the pin by various members, but will for convenience be assumed to be applied at the centers of the eyebars.

The diameter of the pin will be assumed to be 4 inches. Then, since the total stress in the vertical is 116,500 pounds, and the working stress in bearing on the pin is 22,000 pounds per square inch, the required thickness of bearing for Cc is

$$\frac{116{,}500}{4 \times 22{,}000} = 1.324 \text{ inches}$$

The web of a 10-inch 20-pound channel (member Cc) is .38 inch thick, so that the thickness to be made up by pin plates is $1.32 - 2 \times .38 = .56$ inch, or .28 inch on each channel. Two pin plates e, e, Fig. 10 (a) and (b), $\frac{5}{16}$ inch thick, will be used. Then, the thickness of bearing for each channel will be $.38 + .31 = .69$ inch, the center of which is $.69 \div 2 = .345$ inch from the back of the channel. The centers of bearings for the two sides of the vertical member are, then, 8.5 − .345 − .345 = 7.81 inches apart, as shown at h and h' in Fig. 10 (c).

The required thickness of bearing of the top chord is

$$\frac{82{,}700}{4 \times 22{,}000} = .94 \text{ inch}$$

or .94 ÷ 2 = .47 inch on each side. If the $8'' \times \frac{5}{16}''$ side plates that form part of the member CD are continued a short distance beyond C, as shown at g, Fig. 10 (a), the thickness of bearing on each side will be $\frac{5}{8}$ inch, which is sufficient, and the centers of bearing for the two sides of the chord will be 12 + .3125 + .3125 = 12.625 inches apart, as shown in Fig. 10 (d).

As the eyebars are $9\frac{3}{8}$ inches apart and $\frac{7}{8}$ inch thick, the distance between their centers is $10\frac{1}{4}$ inches, as shown in Fig. 10 (c) and (d).

Assuming that the stress in each half of a member acts as a concentrated load at its center of bearing, the horizontal forces that act on the pin at C are shown in Fig. 10 (d) and the vertical forces in Fig. 10 (c).

It will be observed that the bending moment at any point between h and h', Fig. 10 (c), is equal to the moment of the couple formed by the two equal forces at either end of the pin. The lever arm of this couple is

$$\tfrac{1}{2} \times (10.25 - 7.81) = 2.44 \div 2 = 1.22 \text{ inches}$$

The moment is, therefore,

$$58{,}250 \times 1.22 = 71{,}065 \text{ inch-pounds } (= M_V)$$

It is evident that the moment at the left of h or at the right of h' is less than this.

Similarly, the bending moment at any point between g and g', Fig. 10 (d), is

$$41{,}350 \times \frac{12.625 - 10.25}{2} = 49{,}100 \text{ inch-pounds } (= M_H)$$

For the resultant moment we have, therefore,

$$M = \sqrt{71{,}065^2 + 49{,}100^2} = 86{,}400 \text{ inch-pounds}$$

Consulting Table XLI, since the working stress in bending on the pin is 22,000 pounds per square inch, it is seen that a pin $3\frac{1}{2}$ inches in diameter is the smallest that has the required value of resisting moment. The various steps will now be repeated, using $3\frac{1}{2}$ inches for the diameter of the pin.

The required thickness of bearing for the vertical is now

$$\frac{116{,}500}{3.5 \times 22{,}000} = 1.513 \text{ inches}$$

Two $\frac{3}{8}$-inch pin plates will be used, making the thickness of bearing on each side $.38 + .38 = .76$ inch, and the distance between the centers of bearings 7.74 inches, as shown in Fig. 12. The bending moment due to the vertical forces is now $58{,}250 \times 1.255 = 73{,}100$ inch-pounds; that due to the horizontal forces remains the same as before, since the thickness of bearing of top chord used before is sufficient. Then, the corrected value of the maximum bending moment on the pin is

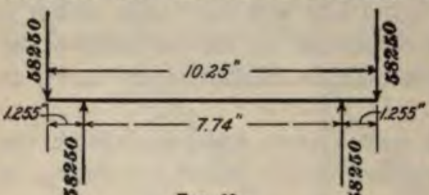


FIG. 12

$$\sqrt{49{,}100^2 + 73{,}100^2} = 88{,}100 \text{ inch-pounds}$$

A pin $3\frac{1}{2}$ inches in diameter has a sufficient resisting moment, and will be used.

20. The total width or thickness of bearing on each side of the vertical Cc, Fig. 10, is .76 inch. If the stress is assumed to be evenly distributed over this width, then, since the thickness of the pin plate is $\frac{3}{8}$ inch, the amount of stress transmitted by each plate is

$$\frac{.38}{.76} \times 58{,}250 = 29{,}125 \text{ pounds}$$

The rivets that connect the pin plates to the channel are $\frac{3}{4}$ inch in diameter and shop driven; the value in single shear, 4,860 pounds, is the smaller. The number of rivets required to transmit the stress in the pin plate to the channel is, therefore, $29{,}125 \div 4{,}860 = 6$. In addition to the required number, which will be placed below the pin, it is customary to put a few rivets above the pin to hold the pin plates firmly to the web of the channel, as shown in Fig. 13. Since the distance c (Table XIII) for a 10-inch 20-pound channel is $8\frac{1}{4}$ inches, the pin plates will be made 8 inches wide.

The section of the top chord is decreased in area by the pinhole, but in this case this is more than made up by the 8-inch plates on the sides.

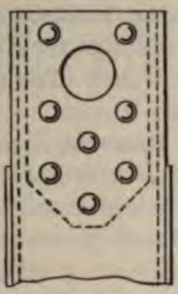


FIG. 13

FIG. 14

21. Pins and Pin Plates at Joints *D* and *E*.—The required sizes of the pins at D and E, Fig. 9, can be found by proceeding in the same way as for joint C. In the present case, and almost invariably in the kind of truss under consideration, they both work out smaller than that required at C, but for the sake of uniformity they are made the same size as at C. Pin plates a, Fig. 14, 7 in. $\times$ $\frac{5}{16}$ in., with four rivets below and two above the pinhole will be riveted to the insides of the channels of Dd and Ee; no pin plates are required on Ee, but it is customary in the best practice to provide at least one plate.

The upper chord is decreased in section by the pinholes at D and E; to restore the lost section, vertical plates a, Fig. 15, will be riveted to the insides of the webs. As

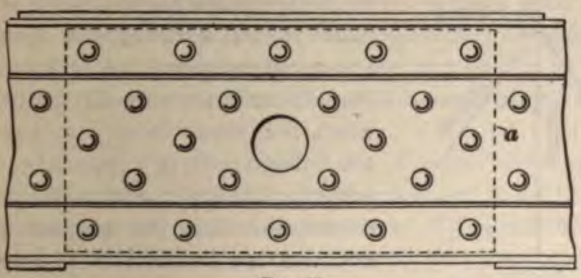


FIG. 15

the thickness of metal at D and E through which the pin passes is $\frac{3}{4}$ inch on each side of the chord, the lost section is $3\frac{1}{2} \times \frac{3}{4} = 2.625$ square inches. A plate 14 in. $\times$ $\frac{3}{8}$ in. will be added on each side; the net width, deducting the diameter of the pinhole, is $10\frac{1}{2}$ inches, and the area is 3.94 square inches. This is more than is necessary, but the plate is made thicker than required, on account of the fact that at D the rivets connecting the plate to the web will have to be countersunk on the inside, to make room for the eyebars composing De, and that the depth of a countersunk head of a $\frac{3}{4}$-inch rivet is $\frac{3}{8}$ inch (Table XIX). Hence, thinner plate cannot be used.

The working stress in DE is 14,170 pounds per square inch (see *Design of a Highway Truss Bridge*, Part 1); and, since the section is decreased 2.625 square inches on each side, the amount of stress that must be transmitted by each of these reinforcing plates is $2.625 \times 14{,}170 = 37{,}200$ pounds. The rivets that connect these plates to the web of the chord are $\frac{3}{4}$ inch in diameter and shop driven; the value in single shear, 4,860 pounds, is the smaller. The number of rivets required in each plate on each side of the pin is, then, $37{,}200 \div 4{,}860 = 7.7$, or, say, 8. The countersunk rivets in these plates are not considered as transmitting any stress (*B. S.*, Art. **109**).

22. Pin and Pin Plates at Joint *B*.—The stresses in the members that meet at B are shown in Fig. 11. They are the maximum stresses, and are not simultaneous; the stress in Bb is greatest when there is a full panel load at b, those in aB and BC are greatest when the truss is fully loaded, and that in Bc is greatest when the joints from c to the right are loaded. It is customary, in the design of the pin at the hip joint B, to assume that the stresses in aB and Bb are maximum at the same time, and then, using the stresses in these members shown in Fig. 11, to find by the method of joints the simultaneous stresses in BC and Bc, using the equations $\Sigma X = 0$ and $\Sigma Y = 0$. On this assumption, the simultaneous forces that act on the pin are shown in Fig. 16; it is unnecessary to consider the dead panel load at B.

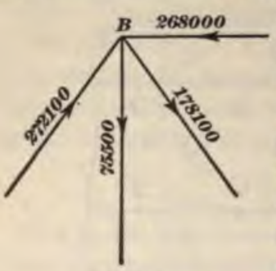


FIG. 16

The customary arrangement of members at joint B is shown in Fig. 17, in which (a) is the elevation of the joint; (b) is the cross-section of the top chord and side elevation of the pin, showing the arrangement of members; and (c) and (d) are half top views and half bottom views of the top chord and end post, respectively. The top chord and end posts are cut off on the line ee, leaving about $\frac{1}{8}$ inch clearance between them; the flange angles and webs of the two members are almost the same distance from the center of the truss, and the members bear against opposite sides of the pin. Pin plates f are riveted to the inside of the end post and extend beyond the pin up into the top chord; pin plates g are riveted to the outside of the top chord, and extend beyond the pin, enclosing the end of the end post. The eyebars b, b that form the diagonal Bc are placed inside the chord and the end post, as close as possible to the inside pin plates f of the end post, the rivets in this plate being countersunk on the inside of the plate to allow the eyebar to get close to it. The hip vertical i is attached to the pin by means of hanger plates j, j that are as close as possible to the inside of the eyebars. The plates k and l are tie-plates; the adjacent ends of the top tie-plates are covered by the bent plate m that is riveted to the end of the top chord. The two angles shown in cross-section at n are the top flange angles of the portal; it is customary to place them above and to continue them across the top chord, as shown, connecting them to it by means of the bent plates o and p.

23. In the discussion of joint C, the clear distance between the webs of the top chord was found to be 12 inches; it is customary to have the distance at B the same. Since the webs are $\frac{5}{16}$ inch thick, the distance between the vertical legs of the flange angles is $12\frac{5}{8}$ inches. The flange angles of the end post will also be placed $12\frac{5}{8}$ inches apart. Since the webs of the end post are $\frac{7}{16}$ inch thick, the clear distance between them will be $11\frac{3}{4}$ inches.

The required diameter of the pin, found by trial, is $4\frac{3}{4}$ inches. This value will now be verified. The maximum

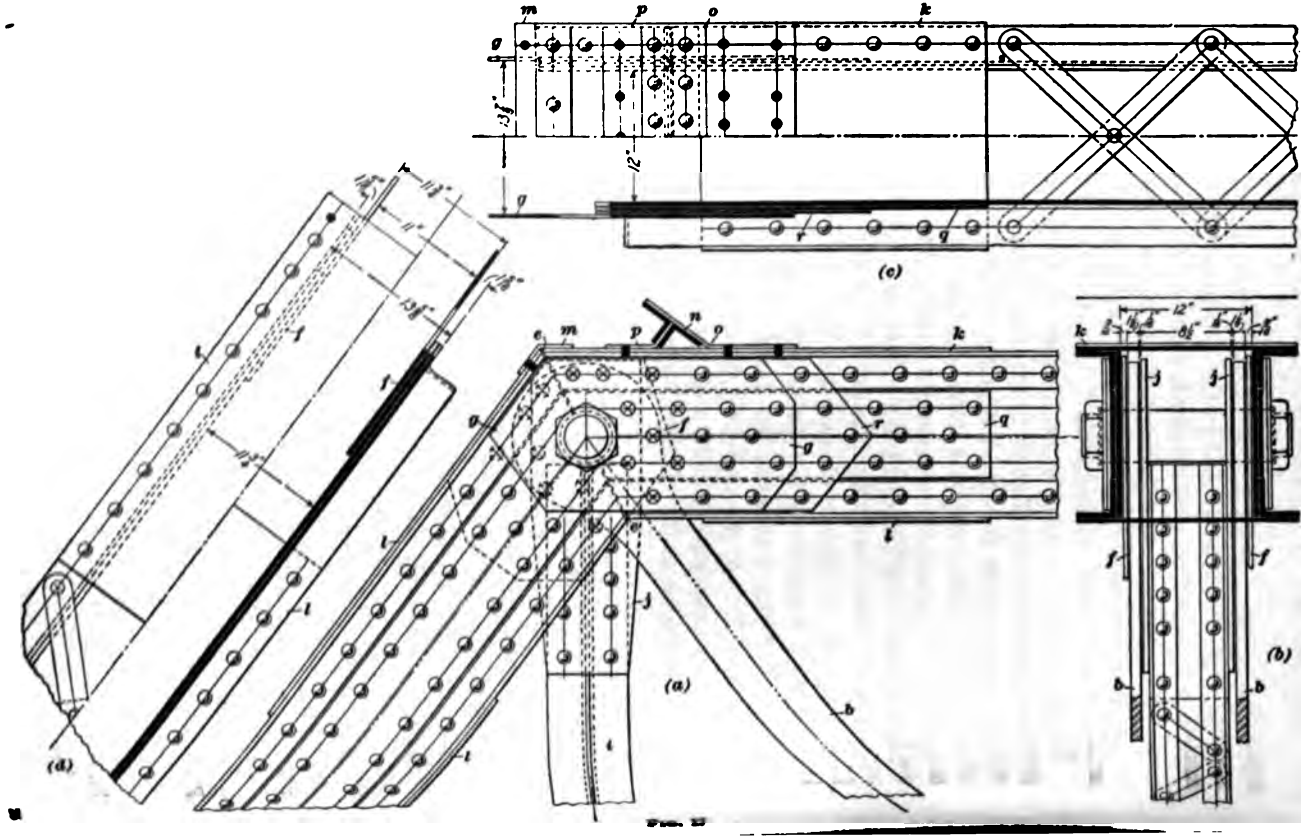


FIG. [illegible]

stress in aB and BC being 272,100 pounds, and the working stress in bearing 22,000 pounds per square inch, the required thickness of bearing for each of these members is

$$\frac{272{,}100}{4.75 \times 22{,}000} = 2.6 \text{ inches,}$$

or 1.3 inches on each side of the member.

The webs of the end posts are $\frac{7}{16}$ inch thick, and the side plates $\frac{1}{2}$ inch thick. Pin plates f, f, Fig. 17, $\frac{3}{8}$ inch thick, will be riveted to the inside, making a total thickness of $1\frac{5}{16}$ inches bearing on each side. As the clear distance between the webs is $11\frac{3}{4}$ inches, the clear distance between the plates f is 11 inches, and the distance between the centers of the bearings is $12\frac{5}{16}$ inches. Since the pin plates are $\frac{3}{8}$ inch thick, and the total width of bearing on each side is $1\frac{5}{16}$ inches, the amount of stress transmitted by one pin plate is

$$\frac{.375}{1.3125} \times 136{,}050 = 38{,}900 \text{ pounds,}$$

and the number of rivets required to connect the plate to the end post is $38{,}900 \div 4{,}860 = 8$.

The webs of the top chord are $\frac{5}{16}$ inch thick. Pin plates q, 8 in. $\times \frac{3}{8}$ in., will be placed on the outside of the member between the vertical legs of the flange angles; pin plates r, 14 in. $\times \frac{1}{4}$ in., will be placed outside of the vertical legs of the angles and the plates q; and pin plates g, 14 in. $\times \frac{3}{8}$ in., will be placed outside of the plates r and enclosing the end of the end post. This makes the total thickness of bearing at each side $1\frac{5}{16}$ inches, and the distance between the centers of bearings, $13\frac{5}{16}$ inches. The numbers of rivets required to connect these plates to the top chord are:

$$\text{Plate, 14 in.,} \times \tfrac{3}{8} \text{ in.,} \quad \frac{\frac{.375}{1.3125} \times 136{,}050}{4{,}860} = 8 \text{ rivets}$$

$$\text{Plate, 14 in.,} \times \tfrac{1}{4} \text{ in.,} \quad \frac{\frac{.25}{1.3125} \times 136{,}050}{4{,}860} = 5.4, \text{ or, say, 6 rivets}$$

$$\text{Plate, 8 in.} \times \tfrac{3}{8} \text{ in.,} \quad \frac{\frac{.375}{1.3125} \times 136{,}050}{4{,}860} = 8 \text{ rivets}$$

In Fig. 17 (*a*), there are ten rivets (not counting countersunk rivets) in the right-hand end of plate *g*, five rivets in the plate *r* beyond the end of *g*, and eight rivets in the plate *q* beyond the end of plate *r*. This is the customary way of arranging the rivets when more than one pin plate is used.

24. The clear distance between the pin plates *f* on the end post was found to be 11 inches; leaving $\frac{1}{16}$ inch clearance on each side makes the outside surfaces of the eyebars $10\frac{7}{8}$ inches apart, their centers being $10\frac{7}{8} - 1\frac{1}{8} = 9\frac{3}{4}$ inches apart, and their inside surfaces $10\frac{7}{8} - 1\frac{1}{8} - 1\frac{1}{8} = 8\frac{5}{8}$ inches apart. Leaving $\frac{1}{16}$ inch clearance on each side makes the outside surfaces of the hanger plates *j* of the hip vertical $8\frac{1}{2}$ inches apart, as shown at the top of Fig. 17 (*b*). It is customary to make the hanger plates not less than $\frac{3}{8}$ inch thick, although in many cases thinner plates give sufficient bearing on the pin. A plate $\frac{5}{8}$ inch thick will be used in the present case. No calculation will be made for the required thickness of bearing, as the actual thickness is greater than necessary; the required net section of the hanger plates, however, must be computed.

In *Design of a Highway Truss Bridge*, Part 1, the required net section of *B b* was found to be 4.7 square inches. In *B. S.*, Art. **143**, it is specified that riveted tension members shall have a net section through the pinhole 25 per cent greater than the net section of the member. Since the required net section of *B b* is 4.7 square inches, the required net section of the hanger plates through the pinholes is $1.25 \times 4.7 = 5.88$ square inches. As there are two plates, each plate must have a net section equal to $5.88 \div 2 = 2.94$ square inches. Hanger plates 10 in. $\times \frac{5}{8}$ in. will be used; the net width (deducting the diameter of the pin) is $10 - 4\frac{3}{4} = 5\frac{1}{4}$ inches, and the net area is $5\frac{1}{4} \times \frac{5}{8} = 3.28$ square inches, which is sufficient. The rivets that connect the hanger plates to *B b* are $\frac{3}{4}$ inch in diameter, shop driven, and in single shear; their value is 4,860 pounds. Then, since the stress in *B b* is 75,500 pounds, the number of rivets required to connect each plate is $37,750 \div 4,860 = 7.8$, or say, 8. In Fig. 17 (*b*), the distance between the outer surfaces of the hanger plates is shown as $8\frac{1}{2}$ inches; then, the distance between centers of bearings is $8\frac{1}{2} - \frac{5}{8} = 7\frac{7}{8}$ inches.

25. The horizontal forces acting on the pin are shown in Fig. 18 (*a*), and the vertical forces in Fig. 18 (*b*). The greatest moment due to the horizontal forces is between

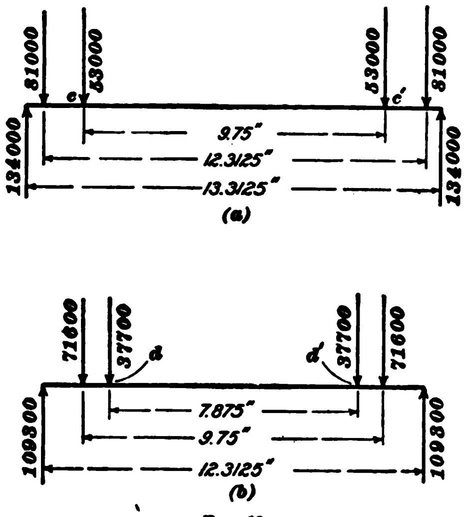


Fig. 18

c and *c'*, and is constant; it is equal to the moment, about *c*, of the forces on the left of *c*, or

$$134{,}000 \times 1.781 - 81{,}000 \times 1.281 = 134{,}900 \text{ inch-pounds}$$

The greatest moment due to the vertical forces is between *d* and *d'*; it is constant between these two points, and is equal to the moment, about *d*, of the forces on the left of *d*, or

$$109{,}300 \times 2.219 - 71{,}600 \times .9375 = 175{,}400 \text{ inch-pounds}$$

Then, the greatest bending moment on the pin is from *d* to *d'*, and its value is

$$\sqrt{134{,}900^2 + 175{,}400^2} = 221{,}300 \text{ inch-pounds}$$

Consulting Table XLI, it is seen that a $4\frac{3}{4}$-inch pin has sufficient resisting moment.

26. Packing of Bottom Chord.—The process of arranging the members on the bottom-chord pins is spoken of as **packing** the bottom chord, and the arrangement is called the **bottom-chord packing.** The packing for the truss under consideration is shown in Fig. 19. It is customary to draw first the cross-sections *f*, *f* of the verticals, and show the location of the webs *h*, *h* and side plates *g*, *g* of the end post. Next, the diagonals *i*, *i* are placed close to the verticals; in the present case, their inner surfaces will be placed $\frac{7}{16}$ inch from the backs of the channels, the same as at joint *C*, to allow $\frac{3}{8}$ inch for the height of rivet heads and $\frac{1}{16}$ inch for clearance. The counter *j* is connected to the center of the pin. Next, the eyebars that form the bottom chord are placed in position on the pins outside of the diagonals, and as close to them and to each other as possible. In calculating the location of the eyebars on the pins, it is assumed that there is $\frac{1}{16}$ inch space between the heads of eyebars. The eyebars of the bottom chord are always alternated on the pin; that is, there is first placed one that comes from the panel on one side, as *k*, Fig. 19 (*e*), then one from the panel on the other side, as *l*, then *k'* and then *l'*, as shown, until all the bars are on.

The rules given in *B. S.*, Art. **139**, regarding the packing of the bottom chord must always be carefully observed. For example, it is seen in Fig. 19 that no two eyebars in the same panel are in contact, and that no eyebar diverges from the center line by more than $\frac{1}{16}$ inch per foot. The distances from the center of the bottom chord to the various eyebars are shown in Fig. 19; the calculation of these distances is simply a matter of addition.

27. In packing the eyebars on the pins, it is seldom possible to have any of them exactly parallel to the center line of the chord; so that one end of each bar will usually be farther from the center line than the other end, the difference being spoken of as the **divergence.** For example, the eyebar *m*, panel *c d*, is $6\frac{5}{16}$ inches from the center line at *c*, and $6\frac{1}{16}$ inches at *d*, so that the divergence is $6\frac{5}{16} - 6\frac{1}{16} = \frac{1}{4}$ inch. In packing the eyebars, it is well to determine the allowable divergence in a panel. Since, in the present

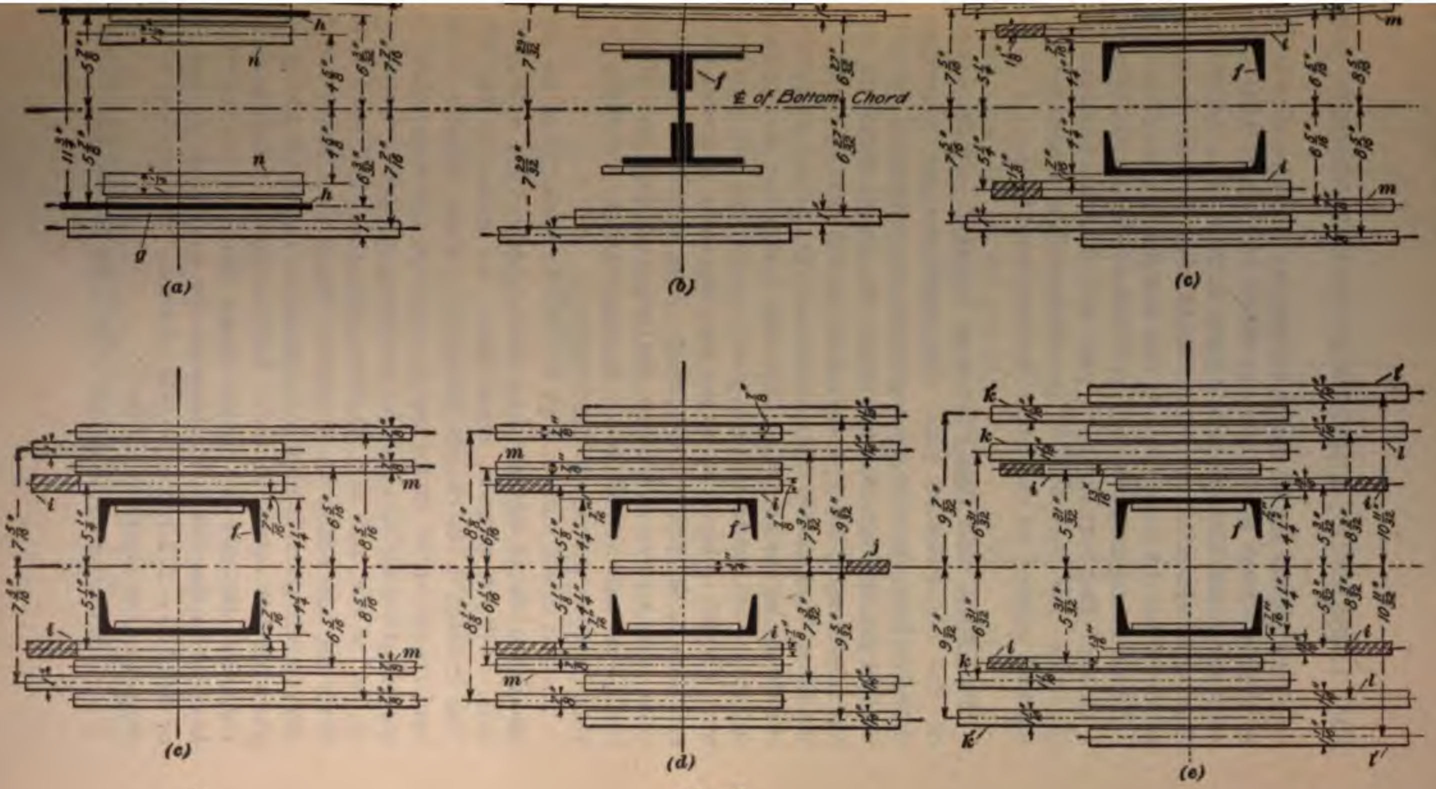


FIG. 19

case, the panels are 20 feet long and the bars are allowed to diverge as much as $\frac{1}{16}$ inch per foot (*B. S.*, Art. **139**), the total allowable divergence in a panel is $20 \times \frac{1}{16} = 1\frac{1}{4}$ inches; that is, the center of an eyebar cannot be more than $1\frac{1}{4}$ inches farther from the center line at one end than at the other.

The dimensions given in Fig. 19 (*a*) will be discussed later.

One of the bottom-chord pins will now be designed to illustrate the method of procedure. The same general method applies to the other bottom-chord pins.

28. Pins and Pin Plates at Joint *d*.—In the design of the bottom-chord pins, the conditions are somewhat different from those in the top chord, on account of the fact that in the former the stresses in the chord members are entirely transmitted from one panel to the next by means of the pins. In general, there are two conditions of loading that require to be considered. They are, first, when the stresses in the *chord members* that meet at the pin are maxima, and, second, when the stress in the *main diagonal* that connects to the pin is a maximum. One of these two conditions will always determine the required diameter of the pin. For each condition it is necessary to calculate the simultaneous stresses in the members.

The pin at the joint *d*, Fig. 9, will first be designed for the maximum chord stresses in cd and de, which are $-272{,}100$ and $-340{,}100$ pounds, respectively, as given in Fig. 11. The stresses in the chords are greatest when there is a full load on the truss, under which condition the counter dE is out of action, and the only forces acting at d are the stresses in Cd, cd, de, and Dd, together with the panel load at d. Since the floorbeams are riveted to the vertical posts above the pin, the panel load at d is applied to the pin at the same place as the stress in Dd. The horizontal component in Cd, when the stresses in the chord members are maxima, is equal to the difference between the stresses in de and cd, or $340{,}100 - 272{,}100 = 68{,}000$ pounds. The vertical component of the stress in Cd is

$$68{,}000 \tan H = 68{,}000 \times \tfrac{27}{20} = 91{,}800 \text{ pounds}$$

The stress that is transmitted to the pin at its contact with the vertical must be 91,800 pounds, equal to the vertical component in *Cd*. The forces that act on pin *d* for this condition are shown in Fig. 20. As only the components of the stress in *Cd* are required, it is not necessary to compute the actual stress in this member.

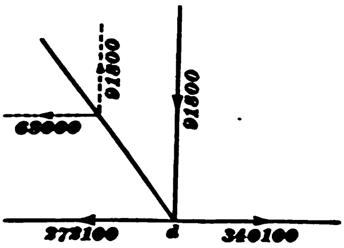


FIG. 20

For this condition, the required diameter of the pin is found to be 4½ inches. This value will now be verified. The only member for which the required thickness of bearing must be computed is the vertical *Dd*. The greatest stress transmitted to the pin by the vertical is greater than that shown in Fig. 20, since the stress shown in Fig. 20 is that in *Dd* when the stresses in the chord members are greatest; it is also greater than the stress in *Dd* shown in Fig. 11, on account of the panel load applied to *Dd* above *d*; it is equal to the vertical component of the maximum stress in *Cd*, that is,

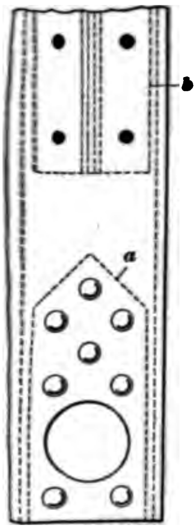


FIG. 21

$$139{,}000 \times \frac{27}{33.6} = 111{,}700 \text{ pounds}$$

Then, the required thickness of bearing is

$$\frac{111{,}700}{4.875 \times 22{,}000} = 1.04 \text{ inches}$$

or .52 inch on each side. Two pin plates $\frac{5}{16}$ inch thick will be riveted to the inside of the member. Since the web of a 9-inch 15-pound channel is .29 inch thick, the total width of bearing on each side of the member is $.29 + .31 = .60$ inch; since the channels are 8¼ inches apart, the distance between the centers of bearings is 7.9 inches. The number of rivets required in each pin plate is $\dfrac{\frac{.31}{.60} \times 111{,}700}{2 \times 4{,}860} = 6$. The lower end of this vertical is shown in Fig. 21. The pin plate is shown at *a*, and the lower end of the diaphragm for the floor connection is shown at *b*.

The horizontal forces acting on the pin at *d* are shown in Fig. 22 (*a*), and the vertical forces in Fig. 22 (*b*). The bending moment due to the vertical forces is constant from *g* to *g*′, and equal to 45,900 × 1.175 = 53,930 inch-pounds.

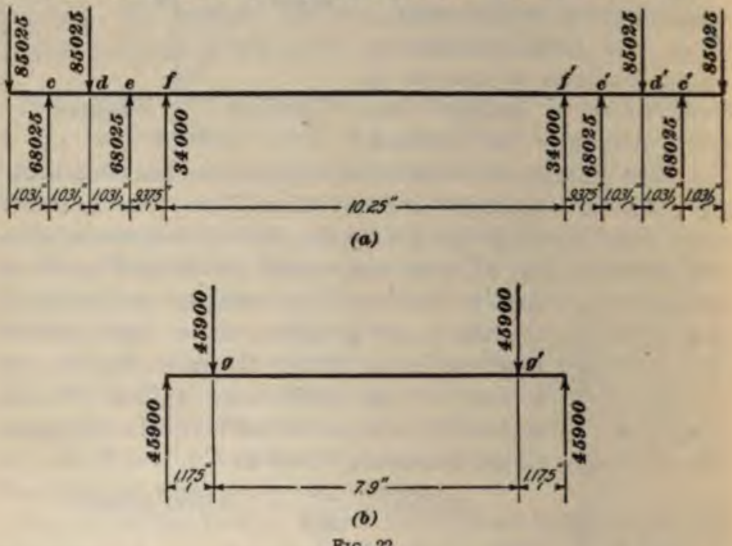


Fig. 22

The bending moments, in inch-pounds, caused at the different points by the horizontal forces are as follows:

at *c*, 85,025 × 1.031 . . . . = 87,660
at *d*, 85,025 × 2.062 − 68,025 × 1.031 . . . . . . = 105,190
at *e*, 85,025 × (1.031 + 3.093) − 68,025 × 2.062 . . = 210,380
at *f*, 85,025 × (1.969 + 4.031) − 68,025 × (.9375 + 3.0) = 242,300

It can be seen that the bending moment is greatest at *f*: it is constant from *f* to *f*′. Then, the greatest bending moment on the pin is from *g* to *g*′, and its value is $\sqrt{53,930^2 + 242,300^2}$ = 248,230 inch-pounds. Consulting Table XLI, since the working stress in bending is 22,000 pounds per square inch, it is seen that a pin 4⅞ inches in diameter has sufficient resisting moment.

It is necessary, in the calculation of bending moments due to the horizontal forces, to calculate the moment at each eyebar or other member; for, under some conditions of packing, the bending moment at some such point as *d* or *e*, due to the horizontal forces alone, is greater than that at any other point due to the combined horizontal and vertical forces.

29. This pin will now be tried for the stresses on it when the stress in the diagonal *Cd* is a maximum; this stress is given in Fig. 11 as $-139{,}000$ pounds. The horizontal component of this stress is

$$139{,}000 \times \frac{20}{33.6} = 82{,}740 \text{ pounds}$$

The stress transmitted to the pin by the vertical under this loading is equal to the vertical component of the stress in *Cd*, or, as found in Art. **28**, 111,700 pounds. The simultaneous stresses in the chords can best be found by considering the live- and dead-load stresses separately. The maximum live-load stress in *Cd* occurs when the live load extends from the right end up to joint *d* and there is no live load to the left of *d*; then, the only external force acting on the truss to the left of the panel *cd* is the left reaction, which, under these conditions, is evidently equal to the shear in panel *cd*. This was found in *Design of a Highway Truss Bridge*, Part 1, to be 70,425 pounds. Since there are no live external forces on the truss to the left of *d*, except a reaction of 70,425 pounds, the live-load stress in *cd* is

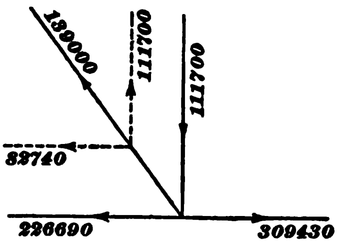


FIG. 23

$$\frac{70{,}425 \times 40}{27} = -104{,}330 \text{ pounds}$$

The dead-load stress is given in Part 1 as $-122{,}360$ pounds. Then, the total stress in *cd* for this loading is $104{,}330 + 122{,}360 = 226{,}690$ pounds.

The stress in *de* will be found by adding to the stress in *cd* the horizontal component of the stress in *Cd*, which gives $226{,}690 + 82{,}740 = 309{,}430$ pounds. The forces acting on the pin are shown in Fig. 23. The horizontal forces acting on the pin are shown in Fig. 24 (*a*), and the vertical forces in Fig. 24 (*b*). The bending moment due to the vertical forces is greatest from g to g', and its value is

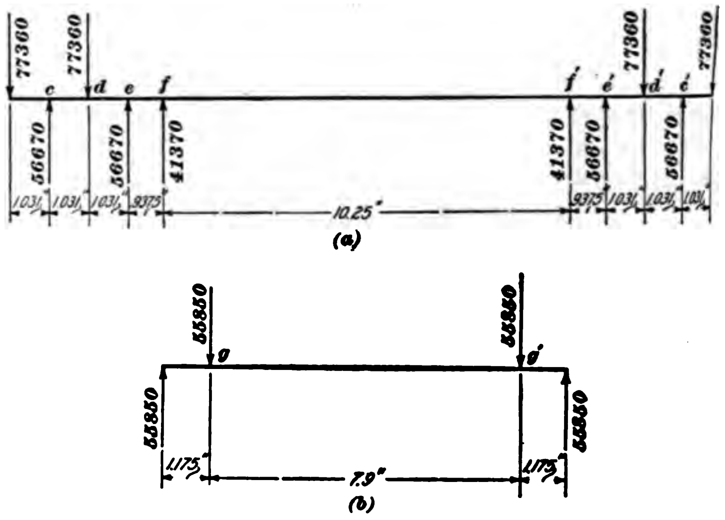


Fig. 24

55,850 × 1.175 = 65,600 inch-pounds. The bending moments, in inch-pounds, caused at different points by the horizontal forces are as follows:

at c, 77,360 × 1.031 . . . . = 79,760
at d, 77,360 × 2.062 − 56,670 × 1.031 . . . . = 101,090
at e, 77,360 × (1.031 + 3.093) − 56,670 × 2.062 . . = 202,180
at f, 77,360 × (1.969 + 4.031) − 56,670 × (.9375 + 3) = 241,000

The greatest bending moment is from g to g', and is

$$\sqrt{65{,}600^2 + 241{,}000^2} = 249{,}800 \text{ inch-pounds}$$

Consulting Table XLI, it is seen that a pin $4\frac{7}{8}$ inches in diameter has sufficient resisting moment; hence, it will be used for the joint d.

In the present case, the required diameters, as found for the two conditions, are the same. This is simply a coincidence; if the diameters had come out different, it would have been necessary to use the larger.

30. Pins at Joints *c* and *e*.—The method of finding the diameters of the bottom-chord pins at joints *c* and *e* is entirely similar to that given in the preceding articles, and should present no difficulty. In the present case, they both work out less than that required at *d*, but for uniformity they will both be made the same size as at *d*, that is, $4\frac{7}{8}$ inches in diameter. It is customary to make all the bottom-chord pins the same size, so that it is necessary in each case to find the pin on which the bending moment is the greatest. This pin can only be found by trial and computation.

31. Pin at Joint *b*.—At the joint *b* there is no stress transmitted to the chord from the vertical, but the pin is passed through plates in the bottom of the vertical, similar to the hanger plates at the top. The only stresses that need be considered at this joint are the stresses in the bottom chords *a b* and *b c*. The eyebars on this pin are not placed close to the vertical, but are so placed as to run as straight as possible from joint *c* to joint *a*; at the latter joint the eyebar is usually placed outside the end post and as close to it as practicable.

32. Pin at Joint *a*.—The forces acting on the pin at the joint *a* are the stresses in *a B* and *a b*, the load from the end floorbeam, and the reaction. The maximum stress in *a B* is +272,100 pounds. The load from the end floorbeam is given in *Design of a Highway Truss Bridge*, Part 1, and is 9,644 + 36,250 = 45,894 pounds. The reaction is equal to the sum of the vertical component in *a B* and the load at *a*, or 218,615 + 45,894 = 264,510 pounds. The stress in *a b* is given in Fig. 11; but, as the stresses in *a B* and *a b* were found for different conditions of loading, there is a slight inconsistency at this joint; that is, the maximum stress in *a b* is not exactly equal to the horizontal component of the maximum stress found in *a B*. Under such conditions, it is best to take for the stress in *a b*, in the design of the pin at joint *a*, the horizontal component of the stress in *a B*, or

$$272{,}100 \times \frac{20}{33.6} = 161{,}960 \text{ pounds,}$$

which in this case is a small amount in excess of the stress in $a\,b$. The external forces acting at the joint a are shown in Fig. 25. A $4\frac{7}{8}$-inch pin will be tried first, this being the size of the remainder of the bottom-chord pins.

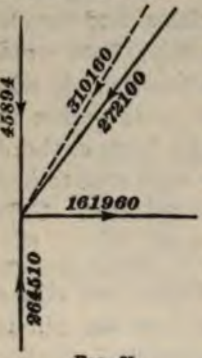


FIG. 25

The load from the end floorbeam is applied to the pin by means of a riveted chair or seat shown at f, Fig. 7 (b), which is riveted to the end post, so that this chair and the end post act as a single member in so far as the load on the pin is concerned. To find the required combined thickness of bearing of the chair and end post, it is necessary to find the resultant of the load from the floorbeam and the stress in $a\,B$, Fig. 25. This is evidently the same as the equilibrant of the reaction and the stress in $a\,b$, that is,

$$\sqrt{264{,}510^2 + 161{,}960^2} = 310{,}160 \text{ pounds}$$

The actual direction of this resultant is of no importance; it is shown in its approximate position by the dotted line in Fig. 25. The required thickness of bearing for the combined chair and end post is, then,

$$\frac{310{,}160}{4.875 \times 22{,}000} = 2.89 \text{ inches,}$$

or 1.44 inches, say $1\frac{7}{16}$ inches on each side. The side plates of the end post are $\frac{1}{2}$ inch thick; the webs are $\frac{7}{16}$ inch thick; the side plates f of the chair, Fig. 7 (b), will be made $\frac{1}{2}$ inch thick; the total thickness on each side is, therefore, $1\frac{7}{16}$ inches. It is not necessary to calculate the number of rivets required to transmit the stress in the plate f to the floorbeam and end post; as a rule, if the ordinary rules of rivet spacing are followed, there will be an excess. Since the webs of the end post are $11\frac{3}{4}$ inches apart, the chair plates will be $10\frac{3}{4}$ inches apart, and the centers of bearing $10\frac{3}{4} + 1\frac{7}{16} = 12\frac{3}{16}$ inches apart. The outside surfaces of the side plates of the end post are, then, $12\frac{3}{16} + 1\frac{7}{16} = 13\frac{5}{8}$ inches apart. Leaving $\frac{1}{8}$ inch clearance on each side makes the inside surfaces of the eyebars $13\frac{7}{8}$ inches apart, and the centers of the eyebars 14¾ inches apart, as shown in Fig. 19 (*a*).

The load is transmitted to the abutment by means of a built-up pedestal, the vertical webs *n*, *n*, Fig. 19 (*a*), of which are inside the end post. The required thickness of the vertical plates is

$$\frac{264{,}510}{4.875 \times 22{,}000} = 2.47 \text{ inches,}$$

or 1.23, say 1¼ inches, on each side. It is somewhat better to make this up with two plates ⅝ inch thick than to use one 1¼-inch plate. The inside surfaces of the chair plates are 10¾ inches apart. Leaving ⅛ inch clearance on each side makes the vertical webs of the pedestal 9¼ inches apart center to center.

The horizontal forces acting on the pin at *a* are shown in Fig. 26 (*a*); the vertical forces, in Fig. 26 (*b*). The bending moment due to the horizontal forces is 80,980 × 1.344 = 108,840 inch-pounds, and is constant from *c* to *c'*; that

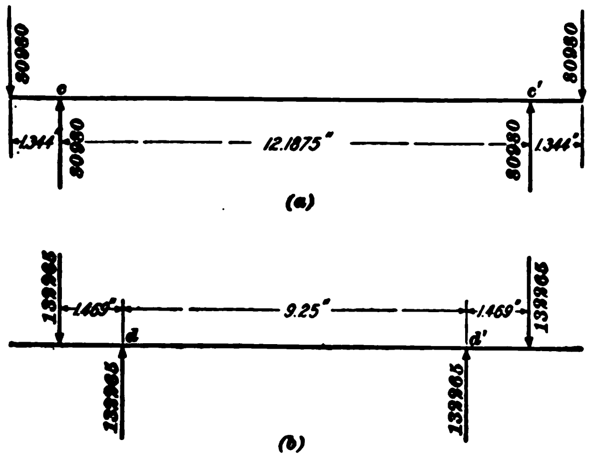


FIG. 26

due to the vertical forces is 132,265 × 1.469 = 194,300 inch-pounds, and is constant from *d* to *d'*. The greatest bending moment on the pin is then from *d* to *d'*, and is

$$\sqrt{108{,}840^2 + 194{,}300^2} = 222{,}700 \text{ inch-pounds}$$

Consulting Table XLI, it is seen that a pin $4\frac{3}{4}$ inches in diameter has a sufficient resisting moment; but, as the remaining bottom-chord pins are to be made $4\frac{7}{8}$ inches in diameter, the latter size will be used for joint *a* also. The detail of joint *a* will be discussed presently.

BEARINGS

33. Required Area.—The required area of bearing for each end of each truss is found by dividing the reaction by the working pressure on the masonry. Since the reaction is 264,510 pounds, and the masonry is cement concrete, on which the working pressure is 300 pounds per square inch (*B. S.*, Art. 103), the required area of bearing is $264{,}510 \div 300 = 881.7$ square inches. If the bedplate is made square, it should be about 30 inches square. The actual dimensions depend on other details, such as rollers, etc.

34. Pedestal.—Built-up pedestals like that shown in Fig. 27 are used for pin-connected trusses. The bottom of the pedestal is made the same length as the length of the bedplate; the width depends somewhat on the distance between the vertical webs, which are just inside the end post. The height of the pedestal depends on the allowable distance from the bottom of the truss to the top of the bridge seat; it is desirable to keep the top of the bridge seat above high water. In general, the height should be about one-half the length; hence, in this case it will be made 15 inches.

In Fig. 27, (*a*) is the elevation of joint *a*, and shows the pedestal, the rollers, and the chair *e* that supports the end floorbeam; (*b*) is an end view of the joint; (*c*) is a top view of the pedestal; and (*d*) is a top view of the rollers. The bottom plate *f* of the pedestal is generally made about $1\frac{1}{2}$ inches thick, and the angles *g*, *g* that connect the vertical webs *h* to the base plate *f* are made 6 in. $\times$ 6 in. $\times$ $\frac{3}{4}$ in. The rivets that connect the angles *g* to the plate *f* are countersunk on the under side. The diaphragms *i*, *i* are riveted to the vertical webs to stiffen the pedestal. In the chair *e*, a diaphragm *j* is inserted between the side plates, in order to

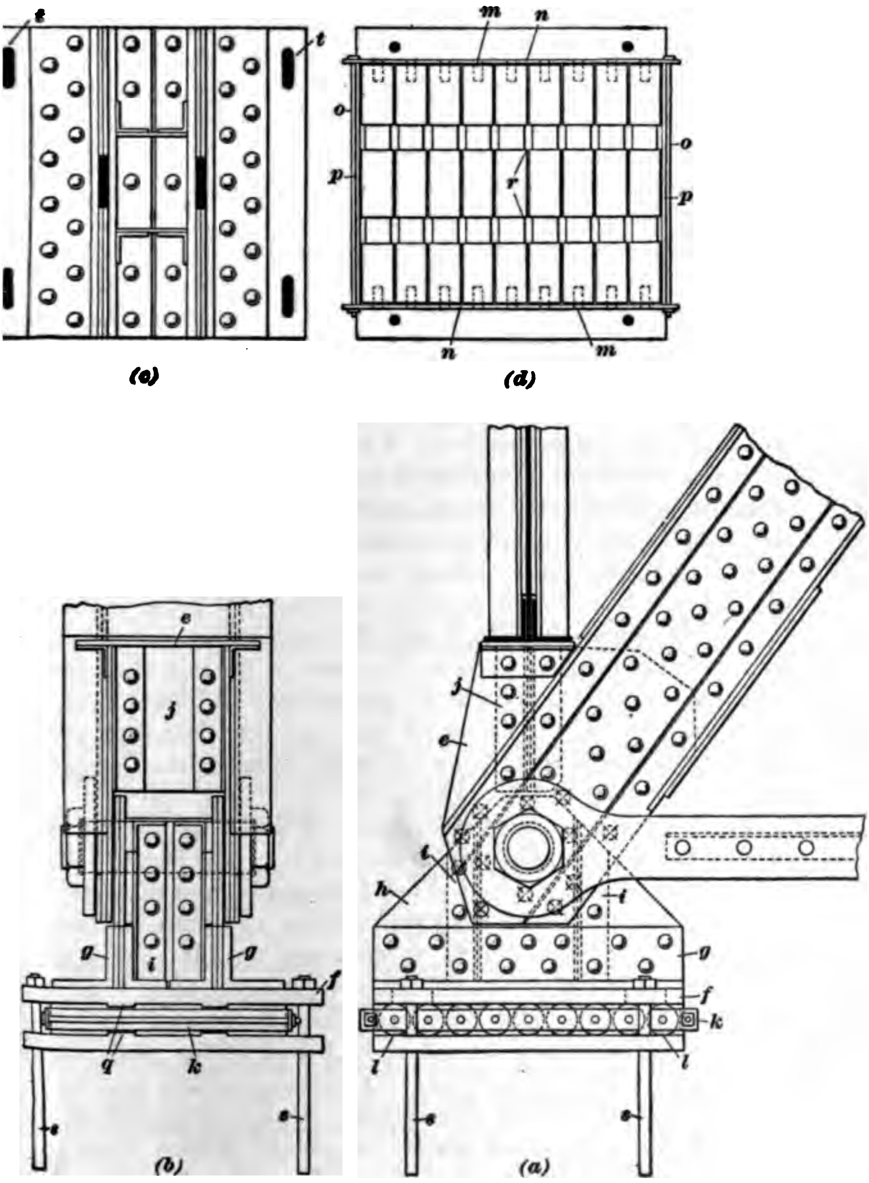


FIG. 27

distribute the load from the floorbeam more evenly between the two sides. At one end of the bridge, the fixed end, the base plate *f* rests directly on the masonry, and is anchored to it by means of anchor bolts. At the other end, rollers are placed under the plate *f*, as shown at *k*, in order to allow that end to move back and forth as the temperature changes.

35. Rollers.—The allowable pressure per linear inch on the rollers is given in *B. S.*, Art. **103**, as 600 *D*. In the present case, the smallest-sized roller (3 inches, *B. S.*, Art. **153**) gives sufficient resistance to transmit the reaction, as will be shown presently. It is customary to space the rollers about $\frac{1}{4}$ inch apart for the full length of the pedestal. In the present case, if the end rollers *l*, *l* are each placed $\frac{1}{2}$ inch from the end of the plate *f*, there is just room for nine rollers in the length of the pedestal. The rollers are usually made at least as long as the distance between the outside edges of the base angles *g*, in this case about 23 inches. Nine rollers at 23 inches gives 207 linear inches of rollers, which, at $600 \times 3 = 1{,}800$ pounds per linear inch, can support 372,600 pounds. As this is greater than the reaction, these rollers (that is, nine rollers 23 inches long) are sufficient. If the allowable load had been less than the reaction, it would have been necessary to use either longer rollers or rollers of larger diameter.

The rollers are fastened together so as to form what is known as a **roller nest**, in the manner shown in Fig. 27 (*d*). Short bolts *m*, *m*, known as **studs**, about $\frac{3}{4}$ inch in diameter, are set into the ends of each roller about $1\frac{1}{2}$ inches, and the ends left to project $\frac{1}{2}$ inch at each end. A flat bar *n* is placed on each end of the rollers, and the short bolts fit into the holes in this bar. The bars *n* are kept in position by the bolts *o* and additional bars *p* at each end of the nest; the bars *p* are $\frac{1}{8}$ inch longer than the rollers, and are inserted between the ends of the side bars *n*. A plate about 2 inches in thickness is inserted under the rollers to give them a smooth surface to roll on, and also to distribute the pressure more evenly over the masonry. The bearing surfaces above and below the rollers are provided with projections *q* that fit into grooves *r* planed in the rollers, and are for the purpose of preventing the pedestal and rollers from moving sidewise. The base plate of the pedestal and the bedplate are continued out beyond the ends of the rollers a sufficient distance (generally 3½ or 4 inches) to allow the insertion of the anchor bolts *s* that hold the pedestal and bedplate in position. At the expansion end, the holes in the base plate through which the anchor bolts pass are made just wide enough to allow the insertion of the bolts, and long enough, as shown at *t*, to allow the base plate and pedestal to move backwards and forwards.

LATERAL CONNECTIONS

36. Upper Lateral Truss.—Fig. 28 shows the connection of the diagonals of the upper lateral truss to the top of the top chord: *a* and *b* are the diagonals, and *c* and *d* are

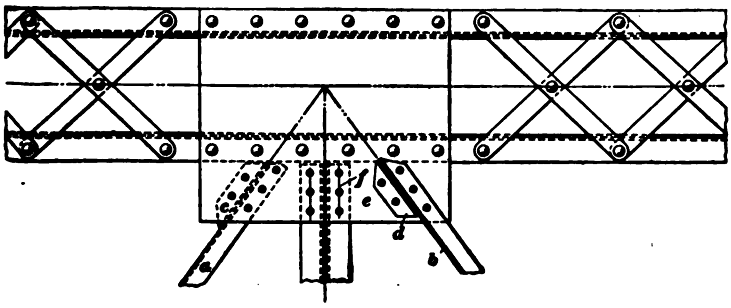


FIG. 28

ng angles that help to connect *a* and *b* to the connection late *e*. This plate also acts as a tie-plate for the top chord, eing extended out a sufficient distance to allow the connection of the laterals. The top flange of the transverse frame s also riveted to this connection plate; the holes for the onnection are shown at *f*.

37. Transverse Frame.—Fig. 29 shows the connection of the transverse frame to the truss. The upper flange *a* is riveted to the connection plate *e* on top of the top chord, as explained in the preceding article. These angles, together with the web members *b* and the bottom flange angles, are connected to the connection plate *c*, which is riveted to the vertical post of the truss by means of the connection

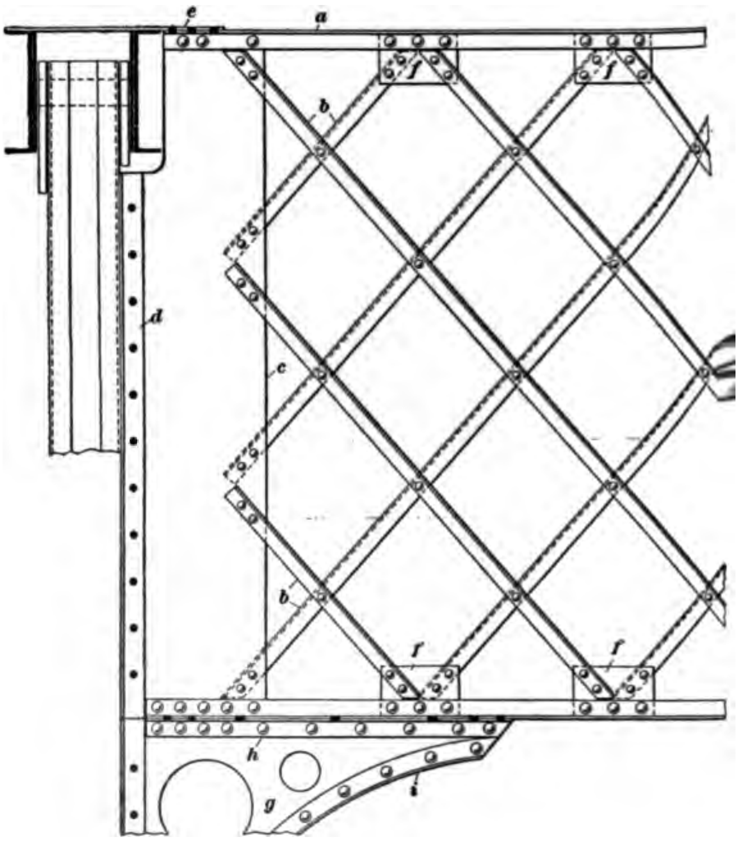


FIG. 29

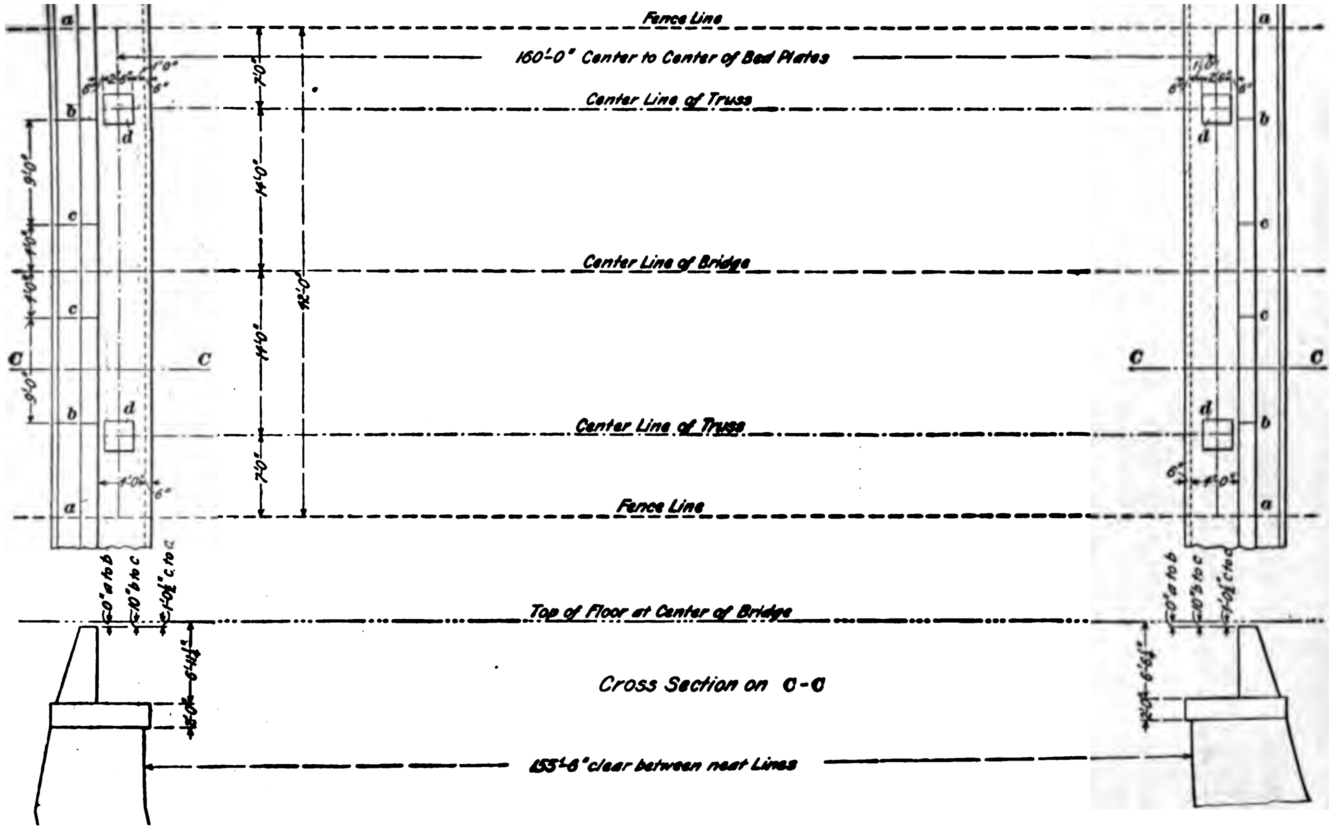
angles *d*; the plate *c* is usually made $\frac{5}{16}$ inch thick, and the angles *d* not less than 3 in. × 3 in. × $\frac{5}{16}$ in. The web members *b* of the frame are connected to the top and bottom flanges by means of the $\frac{5}{16}$-inch plates *f*. The bracket below the frame is composed of the $\frac{5}{16}$-inch web *g*; the angle *h*, the same size as the bottom flange angles; the curved angles *i*, generally $2\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × $\frac{5}{16}$ in.; and the connection angles—continuations of the angles *d* shown above.

38. Portal.—The connection of the portal to the end post corresponds to the connection of the transverse frame that was just described. On account of the portal being inclined, however, the top flange angles are connected to the top chord in the manner shown in Fig. 17 at *p* and *o*. The bent plate *o* is continued out beyond the edge of the top chord, and serves as the connection plate for the diagonal in the end panel of the upper lateral truss, in the same way that the plate *e* serves this purpose in Figs. 28 and 29.

BRIDGE-SEAT PLAN

39. Fig. 30 is the bridge-seat plan for the bridge that has just been designed. The distance between the centers of bedplates or pedestals *d, d* is shown equal to the span, in this case 160 feet. The fronts of the pedestals are placed 1 foot from the neat line of the abutment, making the neat lines 2 feet 3 inches from the center of the pedestal at each end, or 155 feet 6 inches apart. The face of the parapet is placed 6 inches from the back edge of the pedestal and 1 foot 9 inches from the center of the pedestal. This makes the bridge seat 4 feet wide from neat line to parapet. It is common practice to make the bridge-seat stone 2 feet thick for a span of this length, and to have it project about 6 inches beyond the neat line.

The distance from the top of the floor to the top of the bridge seat is found by adding together the vertical distances occupied by stringers, floorbeams, rollers, pedestals, etc. In Fig. 1 it can be seen that the distance from the top of the floor to the center of the pins in the bottom chords is 5 feet $3\frac{1}{4}$ inches. In Art. **34**, it was decided to make the pedestal 1 foot 3 inches from the center of the pin to the bottom. Then, the distance from the top of the floor to the top of the bridge seat at the fixed end (right-hand end) is 6 feet $6\frac{1}{4}$ inches. At the roller end, the distance is


Fence Line
160'-0" Center to Center of Bed Plates
Center Line of Truss
Center Line of Bridge
Center Line of Truss
Fence Line
Top of Floor at Center of Bridge
Cross Section on C-C
155'-6" clear between neat Lines

increased by the rollers 3 inches in diameter and the bed-plate 2 inches in thickness, making the distance 6 feet 11½ inches, as shown in Fig. 30.

The top of the parapet is sometimes finished even with the top surface of the sidewalk and roadway. This is objectionable because the parapet makes a rigid surface over which the traffic must pass. This surface does not wear down as fast as the approach leading to the bridge, with the result that it forms a lump or ridge at each end, and this is a great inconvenience to the traffic. A somewhat better method is to finish the parapets level with the tops of the stringers, so that the floor plank can be continued over them. In Fig. 30, the parapet from *a* to *b* is level with the tops of the sidewalk stringers, that from *b* to *c* is level with the tops of the roadway stringers, and that from *c* to *c* is level with the tops of the stringers under the railway track.

DESIGN OF A RAILROAD TRUSS BRIDGE

ote.—The tables referred to in this Section are the *Bridge Tables* ished with the Course. The abbreviation *B. S.* stands for *Bridge ifications*, which forms another Section of the Course.

order to shorten the work, the correct shape will invariably be en the first time in the following pages. The method of arriving e correct shape by trial has been illustrated sufficiently in preceding ons.

DATA

, **General Data.**—In this Section, a through railroad s bridge will be designed according to the data given the next page. In order to illustrate the method of gning such a bridge, each member will be designed in il. Many of the steps taken in the following pages are ormed mentally by experienced designers; it is necessary beginners, however, to perform almost all the operations iven here.

, **Kind of Bridge.**—As the maximum allowable dis- e from the base of rail to the underneath clearance line feet, it is evident that a through bridge must be used. he span is less than 150 feet, riveted trusses will be used S., Art. **14**). Trusses for railroad bridges are usually e slightly deeper than those for highway bridges, the h being generally from one-fifth to one-sixth of the th. In the present case, an approximate mean will be en between these ratios, and the trusses will be made eet 6 inches deep. In *B. S.*, Art. **229**, it is stated that, riveted truss bridges, panel lengths of from 15 to 20 feet best; in the present case, eight panels, 18 feet each, will

GENERAL DATA

For bridge over *Highway and river*

at *Oberlin, Ohio*

Length and general dimensions *One span 144 feet center to center of bearings, to span the river and the highway on one side of the river*

Angle of abutments with center line of bridge *90°*

Width of bridge and location of trusses *Clear distance between trusses 14 feet*

Floor system *Standard ties on steel stringers (B. S., Art. 48)*

Number and location of tracks *One steam-railroad track at center of bridge*

Loading *Cooper's E50*

Description of abutments *Granite abutments*

Distance from floor to clearance line *Not over 4 feet below base of rail*

" " " to high water *17 feet*

" " " to low water *22 feet*

" " " to river bottom *23 feet*

Character of river bottom *Solid rock*

Usual season for floods *April and May*

Name of nearest railroad station *Oberlin, Ohio*

Distance to nearest " " *2 miles*

Time limit *Six months*

Name of Engineer *International Correspondence Schools*

Address of " *Scranton, Pa.*

Remarks *Top of highway at side of river is 16 feet below base of rail, and 12 feet headroom is required, leaving 4 feet as the maximum depth of floor from the base of rail* be used, as shown in Fig. 1. This will make the angle between the diagonals and the horizontal about 56°, which is a good angle (*B. S.*, Art. **15**).

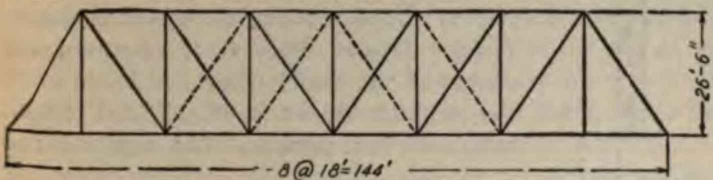


FIG. 1

3. **Width of Bridge.**—In the data, it is stated that there will be one steam-railroad track at the center of the bridge. The required distance from the center line of the track to the inside of the truss is given in *B. S.*, Art. **18**, as 7 feet, which makes the required clear width 14 feet. Assuming that each truss will occupy a width of 2 feet, the distance center to center of trusses is 16 feet. In case the width of the truss comes out 1 or 2 inches more or less than the assumed width, it is unnecessary to revise the design; the correction can be effected by making the floorbeams of such length that there will be 14 feet clear width.

DESIGN OF FLOOR SYSTEM

STRINGERS

4. **Spacing and Depth of Stringers.**—The stringers will be spaced 6 feet 6 inches center to center (*B. S.*, Art. **16**). In *B. S.*, Art. **49**, it is specified that the depth of the stringers shall be not less than one-eighth of the panel length; in this case, therefore, the least allowable depth is $\frac{18}{8} = 2.25$ feet. It is advisable, when possible, to use a somewhat greater depth in order to get greater stiffness; the stringers will be made 2.5 feet, or 30 inches, deep. As this depth is greater than that of the deepest I beam, a plate girder must be used.

5. **Live-Load Shears and Moments.**—The live load that goes to one line of stringers and to one truss is equal

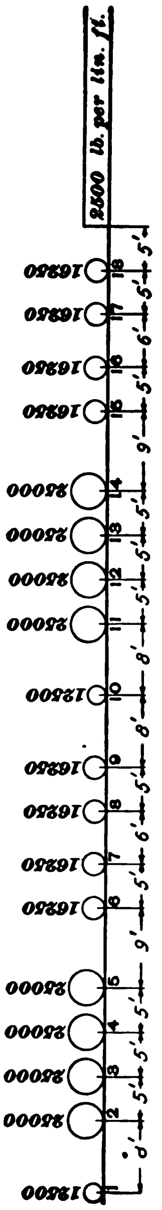


Fig. 2

to one-half of Cooper's E50 loading, ε shown in Fig. 2. The maximum bendin moment caused on a stringer by this loac ing is found as explained in *Stresses i Bridge Trusses*, Part 4: it occurs at th center of the panel when the loads are ir the position shown in Fig. 3, and is equal to 212,500 foot-pounds. The maximum end

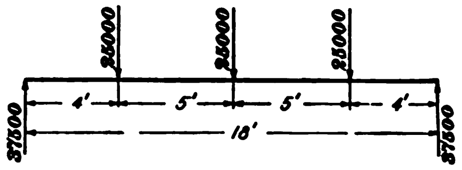


Fig. 3

shear occurs when the loads are in th position shown in Fig. 4, and is equal t 58,330 pounds. The maximum live-loa shear at a section 5 feet from the en which must be computed for the determ nation of the rivet pitch (see *Design of Pla Girders*, Part 1), is found to be 33,33 pounds. As will be seen presently, it :

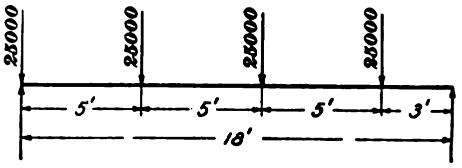


Fig. 4

unnecessary to compute the maximum shea at any other section.

6. Impact and Vibration.—Since th stringer is a floor member, the amounts t be added to the shears and moments, allow for impact and vibration, are as fo lows (*B. S.*, Art. **25**): to the mome 212,500 foot-pounds; to the shear at t end, 58,330 pounds; to the shear at 5 feet from the end, 33,330 pounds.

7. Wind Pressure.—The wind pressure on the train increases the amount of load that goes to the leeward stringer. The center of wind pressure is 7 feet above the top of the rail (*B. S.*, Art. 27) and about 8 feet above the top of the stringers. The center of moments will be taken at the top of the stringers. Then, since the wind pressure is 300 pounds per linear foot (*B. S.*, Art. 27), and the stringers are 6 feet 6 inches center to center (*B. S.*, Art. 48), the increase in load on the leeward stringer is $\frac{300 \times 8}{6.5} = 369.23$ pounds per linear foot. The bending moment due to this increase, at the section where the live-load bending moment is greatest, that is, at the center of the panel, is 14,950 foot-pounds; the shear at the end is 3,323 pounds, and at a section 5 feet from the end, 1,477 pounds.

8. Dead-Load Moments and Shears.—The weight of the track will be assumed as 400 pounds per linear foot, one-half of which, or 200 pounds per linear foot, is carried by each stringer. The weight of the stringer will be assumed to be 150 pounds per linear foot, which makes the total dead load 350 pounds per linear foot. The bending moment due to this dead load, at the section where the live-load moment is greatest, that is, at the center of the panel, is 14,180 foot-pounds; the shear at the end is 3,150 pounds, and at a section 5 feet from the end, 1,400 pounds.

9. Total Moments and Shears.—The total shears and moments are as follows:

Total maximum moment, $212{,}500 + 212{,}500 + 14{,}950 + 14{,}180 = 454{,}130$ foot-pounds.

Total shear at the end, $58{,}330 + 58{,}330 + 3{,}320 + 3{,}150 = 123{,}130$ pounds.

Total shear 5 feet from the end, $33{,}330 + 33{,}330 + 1{,}480 + 1{,}400 = 69{,}540$ pounds.

10. Design of Web.—It was decided in Art. 4 to have the web 30 inches deep. The thickness will be made $\frac{9}{16}$ inch.

Then, the intensity of shearing stress at the end is $\frac{123,130}{30 \times \frac{9}{16} (16.875)} = 7,297$ pounds per square inch. Consulting Table XXXVI, it is seen that the least unsupported distance for the web is 25 inches, and, as the vertical legs of the flange angles will probably be closer together than this, no stiffeners will be required. It is therefore unnecessary to find the intensity of shearing stress at any other section.

11. Design of Flanges.—The required area of flange will be found by the following formula, given in *Design of Plate Girders*, Part 1:

$$A = \frac{M}{s\,h_g} - \frac{t\,h}{8}$$

In the present case, everything is known except h_g. It has been seen in the preceding Sections that h_g is usually less than the width of web; in this case, it will be assumed to be 27 inches. Then,

$$A = \frac{454,100 \times 12}{16,000 \times 27} - \frac{30 \times \frac{9}{16}}{8} = 12.61 - 2.11$$
$$= 10.50 \text{ square inches}$$

Two angles 6 in. × 6 in. × ½ in. will be used for both the top and the bottom flanges. The gross area of the two angles (Table IX) is 11.5 square inches, and the net area, deducting one ⅞-inch rivet hole from each angle, is 10.5 square inches. The distance between the centers of gravity of the flanges is, then, 30.25 − 1.68 − 1.68 = 26.89 inches, and, therefore,

$$A = \frac{454,100 \times 12}{16,000 \times 26.89} - 2.11 = 12.67 - 2.11$$
$$= 10.56 \text{ square inches.}$$

Two angles 6 in. × 6 in. × ½ in. are, therefore, sufficient for the top flange and near enough the required size for the bottom flange.

12. Lateral Bracing Between Stringers.—Since the web is $\frac{9}{16}$ inch thick and the top flange angles are 6 inches wide, the width of the top flange is $12\frac{9}{16}$ inches. Since the panel length is 18 feet, it is greater than 12 times the width of the flange, and, according to *B. S.*, Art. 87, lateral bracing is required between the top flanges. This will be made of the same general form as used in the Section on *Design of Plate Girders*, Part 2.

13. **Flange Rivets.**—The flange rivets are ⅞ inch in diameter, shop driven, in double shear, and in bearing on the $\frac{7}{16}$-inch web; the latter value, 10,830 pounds, is the smaller. Since the center line between the two rivet lines of a 6-inch leg is 3⅜ inches from the back of the angle, and the flanges are 30¼ inches back to back, the distance h_r between the centers of the rivet lines is $30\frac{1}{4} - 3\frac{3}{8} - 3\frac{3}{8} = 23\frac{1}{2}$ inches. Then, the required pitch of the rivets at the end of the stringer, as found from the formula $p = \frac{K h_r}{V}$ (*Design of Plate Girders*, Part 1), is

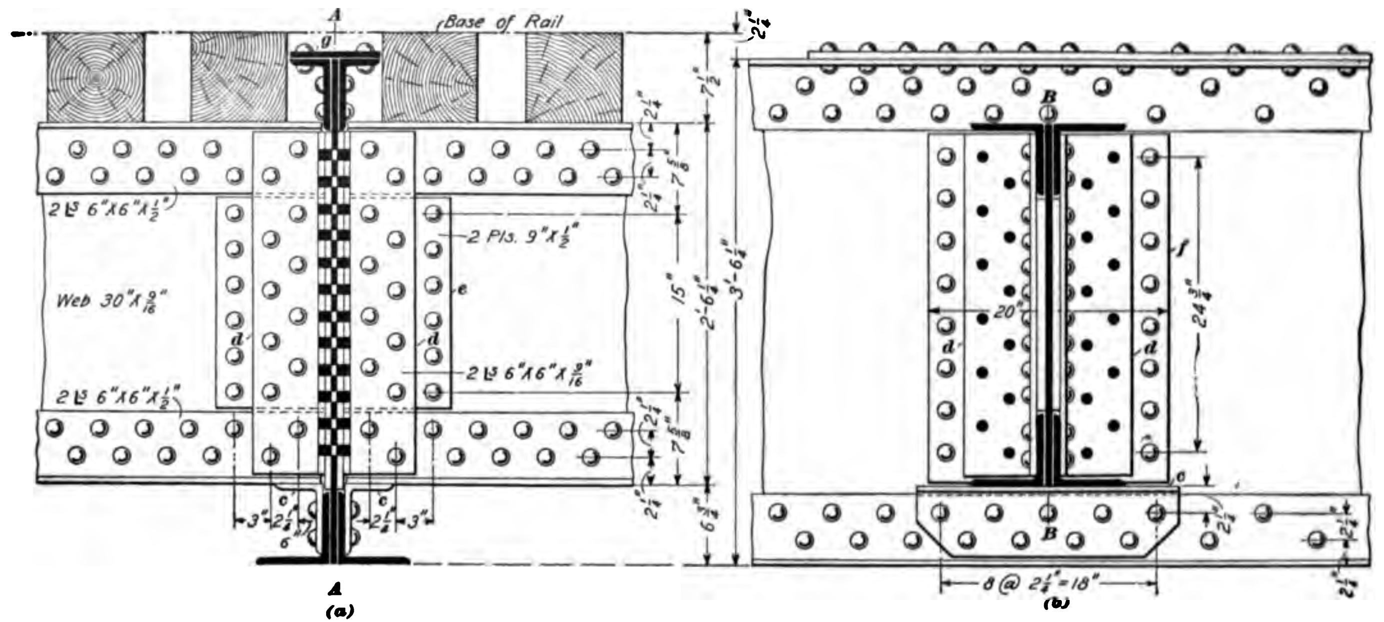
$$\frac{10{,}830 \times 23.5}{123{,}130} = 2.07 \text{ inches;}$$

and at 5 feet from the end,

$$\frac{10{,}830 \times 23.5}{69{,}540} = 3.66 \text{ inches}$$

As the ties will rest directly on top of the top flange, the required pitch of rivets in the top flange is nine-tenths of the above, that is, 1.86 inches at the end and 3.29 inches at 5 feet from the end.

For practical reasons, the rivet spacing in the top flanges of the stringers in railroad bridges should not exceed 3½ inches, the principal reason being the fact that these rivets transmit the load directly to the web. The pitch is usually taken to the next lower ¼ inch. Then, the rivets in the top flange will be spaced 1¾ inches apart at the end and 3¼ inches apart at 5 feet from the end. Between these sections, the pitch is gradually changed at the rate of about ½ inch each time, making the successive pitches 1¾, 2¼, 2¾, and 3¼ inches. It is customary to make each of these cover approximately the same distance as far as conditions will permit. For example, in the present case, the following spacing may be used:



Base of Rail
2 ⅃ 6"X6"X$\frac{1}{2}$"
2 Pls. 9"X$\frac{1}{2}$"
Web 30"X$\frac{9}{16}$"
2 ⅃ 6"X6"X$\frac{9}{16}$"
2 ⅃ 6"X6"X$\frac{1}{2}$"
15"
2'-6$\frac{1}{4}$"
3'-6$\frac{1}{4}$"
20"
24$\frac{3}{4}$"
8 @ 2$\frac{1}{4}$"=18"
(a)
(b)

FIG. 5

12 spaces at $1\frac{3}{4}$ inches = 1 foot 9 inches
10 spaces at $2\frac{1}{4}$ inches = 1 foot $10\frac{1}{2}$ inches
6 spaces at $2\frac{3}{4}$ inches = 1 foot $4\frac{1}{2}$ inches

Total distance = 5 feet

Beyond the section at 5 feet from the end, a few spaces at $\frac{1}{4}$ inches can be inserted, and then a pitch of $3\frac{1}{2}$ inches used ıroughout the remainder of the flange as far as the $3\frac{1}{4}$-inch itch at the other end.

14. Connection of Stringer to Floorbeam.—In the esign of the highway bridge in the preceding Sections, the esign of the floor connections was left until after the design f the trusses. In the present case, the connections of the :ringer to the floorbeam will be designed now, for in rail- ›ad bridges it is much less difficult to decide on the type f connection. Fig. 5 shows a typical connection: *AA*, ig. 5 (*a*), is the usual cross-section of the floorbeam, and *B*, Fig. 5 (*b*), is the cross-section of the stringer. The tter rests on a shelf angle *c* that is riveted to the bottom ınge angles of the floorbeam; the connection angles *d* are aced outside of the flange angles of the stringer, and tight lers *e* are placed between them and the web. Tight fillers : reinforcing plates *f* are also riveted to the web of the ɔorbeam. With this style of connection, the smallest value f the rivets that connect the angles to the stringer is 3,200 pounds ($\frac{7}{8}$-inch rivets, shop driven, in double shear), ıd that of the rivets that connect the angles to the floorbeam 5,410 pounds ($\frac{7}{8}$-inch rivets, field driven, in single shear). 'hen, the number of rivets required to connect the angles › the stringer and to the floorbeam is, respectively, since ıe end shear is 123,130 pounds, $\frac{123,130}{13,200} = 9.3$ rivets, and $\frac{23,130}{5,410} = 22.8$ rivets. It is customary to use the next arger even number, 24 in this case, for the latter, and half his number, 12 in this case, for the former, as shown in Fig. 5. As it is impossible to get this number of rivets in a single row without getting the rivets too close together, 6-inch angles are used, giving room for two rows in each leg. The least allowable thickness of connection angle is $\frac{9}{16}$ inch (*B. S.*, Art. **51**), so $6'' \times 6'' \times \frac{9}{16}''$ angles are used. It is well to make the thickness of the connection angles at least equal to that of the stringer web. Had the stringer web been $\frac{5}{8}$ inch thick, $6'' \times 6'' \times \frac{5}{8}''$ connection angles would have been used.

INTERMEDIATE FLOORBEAMS

15. Length and Depth.—The length of the floorbeam will be taken as 16 feet, the distance center to center of the trusses. When the connection represented in Fig. 5 is used,

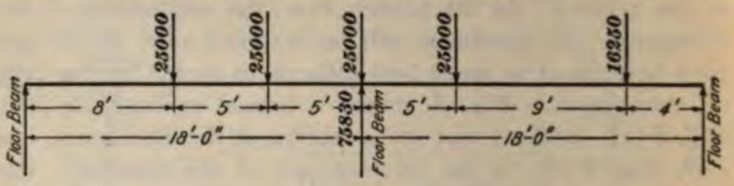


FIG. 6

the floorbeam is made 1 foot deeper than the stringer; if this does not give a depth equal to one-sixth the length (*B. S.*, Art. **49**), the shelf angles are placed farther from the bottom flange, and stiffeners are placed under them. The top flange of the floorbeam should never be more than about 6 inches above the top of the stringers, as it will interfere with the rail if it is higher than this. In the present case, a web 42 inches in width will be used for the floorbeam.

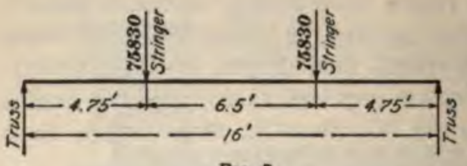


FIG. 7

16. Live-Load Shears and Moments.—The live load that comes on the floorbeam from one line of stringers is greatest when the loads are in the position shown in Fig. 6, and is equal to 75,830 pounds. As the stringers are 6 feet 6 inches apart, and the floorbeam is 16 feet long, the loads are applied to the floorbeam as shown in Fig. 7, the shear from the truss to the stringer being 75,830 pounds, and the bending moment on the floorbeam at the stringer connection being 75,830 × 4.75 = 360,200 foot-pounds. This is also the value of the bending moment at the center, as it is constant between the stringer connections.

17. Impact and Vibration.—Since the floorbeam is a floor member, the amounts that must be added to the shear and moment to provide for the effect of impact and vibration are as follows: to the shear, 75,830 pounds; and to the moment, 360,200 foot-pounds (*B. S.*, Art. 25).

18. Wind Pressure.—In Art. 7 it was found that the wind pressure on the train increased the load on the leeward stringer by 369.23 pounds per linear foot. Then, the increase in the load on the floorbeam at the leeward stringer connection is 18 × 369.23 = 6,650 pounds. There will be a corresponding decrease in the load on the windward stringer, as shown in Fig. 8, in which the increase and the decrease are shown as a downward and an upward force, respectively, instead of adding them to and subtracting them from the loads shown in Fig. 7.

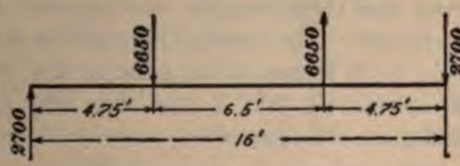


FIG. 8

Taking moments about the right end of the floorbeam, the reaction at the left end, which is also the shear from the truss to the stringer, is found to be 2,700 pounds, and the bending moment at the left-hand stringer, 2,700 × 4.75 = 12,800 foot-pounds. It is unnecessary to find the shear and moment at the other end, as they simply tend to decrease those due to live load.

19. Dead Load.—The dead load on each stringer (Art. 8) is 350 pounds per linear foot; the amount that goes to the floorbeam at each stringer connection is then 350 × 18 = 6,300 pounds. The weight of the floorbeam will be assumed to be 200 pounds per linear foot. The dead-load shear at the end is then 7,900 pounds, and the bending moment at the stringer connection is 36,300 foot-pounds.

20. Total Shear and Moment.—The total shear and moment are as follows:

Total shear at the end, 75,830 + 75,830 + 2,700 + 7,900 = 162,260 pounds.

Total bending moment at the stringer, 360,200 + 360,200 + 12,800 + 36,300 = 769,500 foot-pounds.

21. Design of Web.—In Art. 15 it was decided to make the web of the floorbeam 42 inches deep. A thickness of $\frac{5}{8}$ inch will be used. Then, the intensity of shearing stress at the end is $\frac{162{,}260}{42 \times \frac{5}{8}} = 6{,}180$ pounds per square inch; and, as the shear is nearly constant from the truss to the stringer, this will be taken as the intensity of shearing stress on that portion of the floorbeam. Consulting Table XXXVI, it is seen that the allowable unsupported distance on the web is 33 inches. The vertical legs of the flange angles will probably be 6 inches wide, leaving the distance between them 30 inches; as this is less than the allowable distance, no stiffeners are required.

22. Design of Flanges.—The required area of flange will be found by the formula

$$A = \frac{M}{s\,h_r} - \frac{t\,h}{8}$$

The value of h_r will be assumed to be 3 inches less than the distance back to back of the flange angles, that is, 42.25 − 3 = 39.25 inches. Then,

$$A = \frac{769{,}500 \times 12}{16{,}000 \times 39.25} - \frac{42 \times \frac{5}{8}}{8} = 14.70 - 3.28$$
$$= 11.42 \text{ square inches.}$$

For the bottom flange, two angles 6 in. × 6 in. × $\frac{9}{16}$ in. will be used. As there will be no flange plates in this case, there is no necessity for rivets in the outstanding legs of the bottom flange angles at the point where the moment is greatest, so that it will be sufficient to deduct the cross-section of one hole from each angle. The area of one angle is

6.43 square inches (Table IX); the area to be deducted for one $\frac{7}{8}$-inch rivet hole in material $\frac{9}{16}$ inch thick is .56 square inch. Then, the net area of the two angles is 2 (6.43 − .56) = 11.74 square inches, which is sufficient. The center of gravity is 1.71 inches from the back of the angles.

For the top flange, two angles 6 in. × $3\frac{1}{2}$ in. × $\frac{7}{16}$ in. and one flange plate 8 in. × $\frac{1}{2}$ in. will be used. They give a gross area of 11.92 square inches, which is sufficient. The center of gravity is 1.28 inches from the top of the angles. This gives h_r = 42.25 − 1.71 − 1.28 = 39.26 inches, which is very close to the assumed distance. It is customary to make the top flange of the floorbeam as narrow as practi-

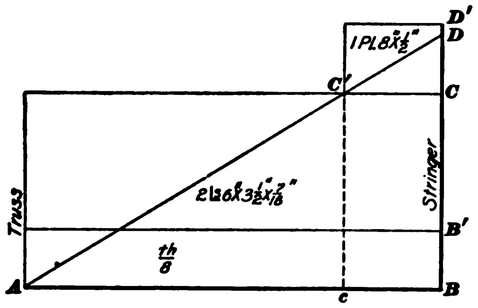


Fig. 9

cable, on account of the fact that it projects above the stringers and therefore lies between the ties (see *g*, Fig. 5), which must be spread to make room for it.

23. Length of Flange Plate.—The required length of flange plate can be found by the curve of the flange areas for the portion of the floorbeam between the truss and the stringer (see *Design of Plate Girders*, Part 1). This curve is shown in Fig. 9: *A B* represents the distance from the truss to the stringer, *B D* the required area $\left(\frac{M}{s\,h_r}\right)$, *B B'* one-eighth of the web $\left(\frac{t\,h}{8}\right)$, *B' C* the area of the two flange angles, and *C D'* the area of the flange plate. As the bending moment from *A* to *B*, due to all the loads except the weight of the floorbeam, varies uniformly from *A* to *B*, it is customary to

assume that the curve of the flange areas AD is a straight line. It is unnecessary to consider the required flange area between the stringers, as the entire flange section is carried between them.

The line CC' is drawn parallel to AB, to its intersection C' with AD, and $C'c$ is drawn at right angles to AB; then, c is the section at which the flange plate can be cut off. The distance Ac can be found very readily by the formula

$$Ac = \frac{C'c}{BD} \times AB$$

In the present case, $C'c = 3.28 + 7.92 = 11.20$, $BD = 14.70$, and $AB = 4.75$; therefore,

$$Ac = \frac{11.20}{14.70} \times 4.75 = 3.62 \text{ feet}$$

24. Flange Rivets.—The flange rivets are $\frac{7}{8}$ inch in diameter, shop driven, in double shear, and in bearing on the $\frac{5}{8}$-inch web; the bearing value, 12,000 pounds, is the smaller. Since the flange angles are $42\frac{1}{4}$ inches back to back, and the center line between the gauge lines is $3\frac{3}{8}$ inches from the back of the angles (Table XII), the rivet lines of the flanges are $42.25 - 3.375 - 3.375 = 35.5$ inches center to center. Then, (see formula in Art. **13**), the required pitch of the rivets at the end of the floorbeam is $\frac{12{,}000 \times 35.5}{162{,}260} = 2.63$ inches. As the shear is very nearly constant from the truss to the stringer connection, this pitch will be used for that entire distance. The shear between the two stringers is practically equal to zero, so that the smallest allowable pitch, or $4\frac{1}{2}$ inches, will be used in the floorbeam flanges between them (*B. S.*, Art. **57**).

As the width of the outstanding leg of the top flange angles is $3\frac{1}{2}$ inches, only one line of rivets can be driven in it, while in the vertical leg, 6 inches wide, two rows can be driven. The required pitch or number of rivets in the outstanding leg in this case can be found by considering the stress in the $8'' \times \frac{1}{2}''$ flange plate. As the area of the plate is 4 square inches, and the working stress in compression is 16,000 pounds per square inch (*B. S.*, Art. **29**), the total stress in the plate is $4 \times 16{,}000 = 64{,}000$ pounds. The value of one rivet ($\frac{7}{8}$ inch in diameter, shop driven, in single shear) is 6,610 pounds. Then, the number of rivets required to transmit the stress to the plate is $\frac{64{,}000}{6{,}610} = 9.7$, or, say, 10. That is, there must be 10 rivets in the top flange plate, between the stringer connection and the end of the plate. If it is impossible to get this number of rivets in the distance cB, Fig. 9, without violating the rules for minimum rivet pitch, the top flange plate will be made longer than required by the curve of flange areas. In any case, it is well to make it slightly longer than required.

25. Connection of Floorbeam to Truss.—The floorbeams of railroad bridges, especially in pin-connected truss bridges, are frequently connected to the vertical posts above the lower chord in the same way as in the highway bridge designed in the two preceding Sections of this Course. A much more usual connection, however, especially for riveted trusses, is shown in Fig. 10. In this figure, (*a*) is the elevation of one end of the floorbeam, showing the holes for the stringer connection and the cross-section of the bottom chord of the truss, together with the elevation of a vertical post; (*b*) is an end view of the connection angles of the floorbeam; (*c*) is a cross-section on the plane CC and a plan of the lower flange; and (*d*) is a top view of a portion of the top flange. The stringer connection was considered in Art. **14**, and will not be further discussed. The splice in the floorbeam web and the connection to the truss will now be considered.

26. The main object in this type of connection is to distribute the load that comes from the floorbeam over a greater length of the vertical member, thereby insuring a better distribution of the load between the two sides of the truss. For this purpose, the depth of the floorbeam web is increased at the end; this is accomplished by splicing the web between the truss and the stringer at ee, and allowing a portion f of the end of the web to project up through the top flange angles.

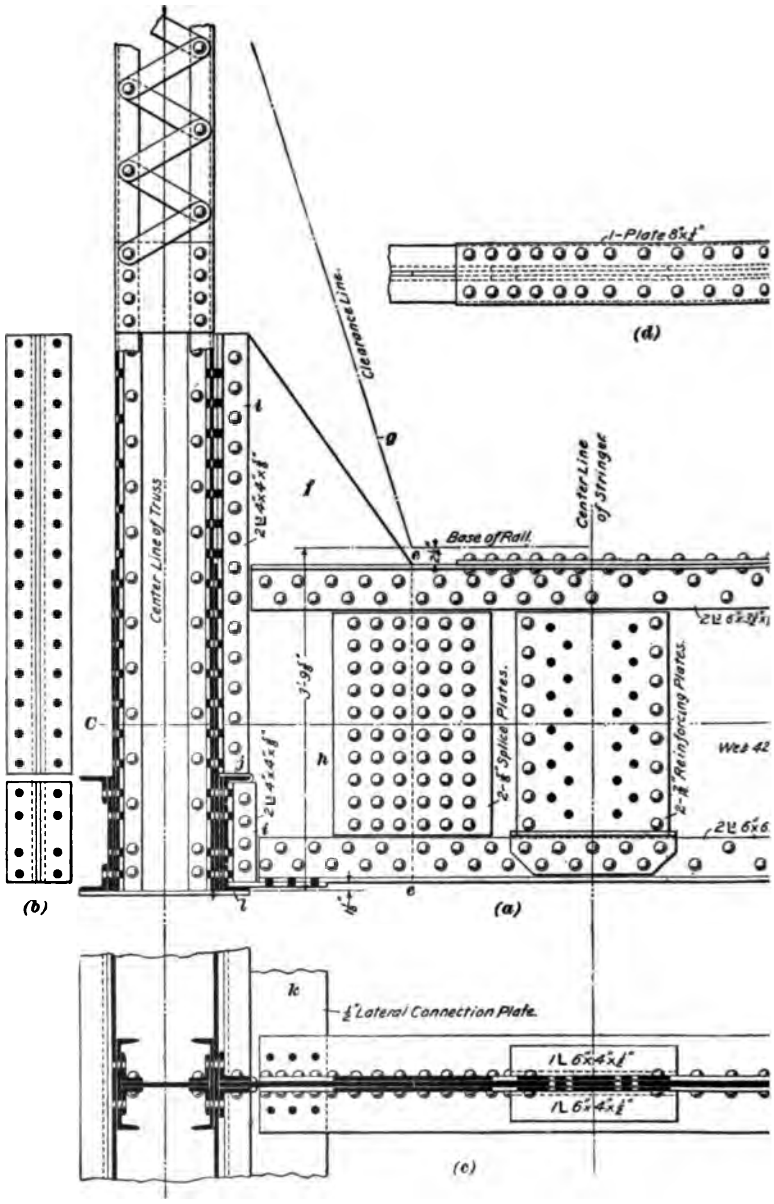

1-Plate 8"x½"
(d)
Clearance Line.
g
f
Center Line of Truss
Center Line of Stringer
Base of Rail
Splice Plates.
Reinforcing Plates.
Web 42
h
C
e
(b)
(a)
k
½" Lateral Connection Plate.
1L 6"x4"x½"
(c)

FIG. 10

The deep end portion of the web, frequently called the **floorbeam gusset**, should not come inside the clearance line, as given in *B. S.*, Art. **18**, and shown at *g* in Fig. 10.

It is not customary, in splicing a floorbeam web, to apply the formulas given in *Design of Plate Girders*, Part 1, but rather to design the splice so as to transmit the shear from one portion of the web to the other. As a rule, if the floorbeam is over 3 feet deep, and each splice plate *h* is made the same thickness as the web, and three rows of rivets spaced 3 inches apart, as shown in Fig. 10, are put in on each side of the splice, it will be unnecessary to make any computation. When the floorbeam is less than 3 feet deep, the splice plates are turned into reinforcing plates and continued from the end of the floorbeam out beyond the splice and stringer connection. In the splice shown in Fig. 10, the resisting moment of the rivets is not so great as that of the web, but at this section there is sufficient flange area even if the effect of the web in resisting the moment is not counted; so that it is not necessary to have the resisting moment of the rivets so great as that of the web at this section.

The rivets connecting the connection angles *i* to the gusset are $\frac{7}{8}$ inch in diameter, and shop driven. Their smallest value is that in bearing on the $\frac{5}{8}$-inch gusset, and is 12,000 pounds. Since the end shear on the floorbeam is 162,260 pounds, the number of rivets required is $\frac{162,260}{12,000} = 13.5$, or, say, 14. The rivets connecting the connection angles to the truss are field driven; their value in single shear, 5,410 pounds, is the smallest. The required number of rivets is $\frac{162,260}{5,410} = 30$.

In Fig. 10, there are more rivets than are necessary. This is a common practice, as it is considered advisable to have a few extra rivets.

27. No rule can be given as to the height of the floorbeam gusset at the end. In half-through plate-girder bridges, the gusset is continued up to the under side of the top flange;

in pony truss bridges, it is made as large and as high as the clearance line will allow; in through truss bridges, as in the present case, its height depends on the depth of the floorbeam and somewhat on the required number of rivets in the connection angle. In no case should the gusset encroach on the required clearance.

The gusset is cut near the bottom at *j* to allow for the outstanding leg of the top flange angle of the bottom chord, and the connection angle *i* on each side is put on in two pieces. The connection angles will be made 4 in. × 4 in. × $\frac{5}{8}$ in.

28. Between the bottom of the floorbeam and the top side of the outstanding leg of the lower flange angle of the bottom chord is placed a plate *k*, $\frac{1}{2}$ inch thick, which is riveted to the chord and to the bottom flange of the floorbeam, and serves the purpose of connecting the diagonals of the lateral truss to the floorbeam and bottom chord. The lateral connections will be discussed further on.

29. In Fig. 5, the distance from the base of the rail to the top of the stringer is $7\frac{1}{2}$ inches (*B. S.*, Art. **48**), and the top of the shelf angle is $6\frac{3}{4}$ inches above the bottom of the bottom flange of the floorbeam, making the bottom of the floorbeam 3 feet $8\frac{1}{2}$ inches below the base of the rail. Adding to this $\frac{1}{2}$ inch for the thickness of the lateral connection plate *k*, and $\frac{5}{8}$ inch for the thickness of the bottom chord angle *l* (to be found later), gives the distance from the base of the rail to the bottom of the bottom chord as 3 feet $9\frac{5}{8}$ inches, as shown in Fig. 10. This distance will be referred to again in the following articles.

END STRUTS OR FRAMES

30. For the sake of variety, no end floorbeams will be provided, although in the best practice they are always put in. In the present case, the end stringers will be allowed to rest directly on the masonry, and their ends will be connected to each other and to the ends of the trusses by frames, as

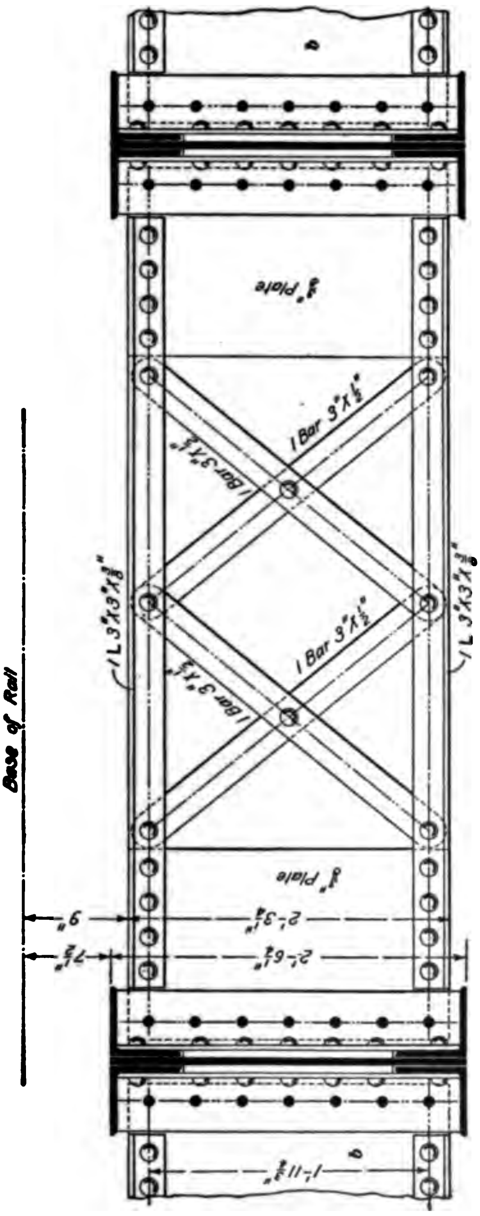


Fig. 11

shown in Fig. 11. Four stiffeners are placed at the end of each stringer where it rests on the masonry, and sole plates and bedplates 1 inch thick are provided, as shown in Fig. 12. In the present case, the bedplates will be made 20 inches square, which is a customary size. It is unnecessary to calculate the required thickness of the end stiffeners on stringers; if the smallest allowable angles are used—in the present case, 5 in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in. (*B. S.*, Art. **55**)—they will be large enough. End stringers are usually made somewhat longer than the intermediate stringers; the difference is equal to one-half the length of the bedplate, which makes the distance from the center of the bedplate to the next floorbeam equal

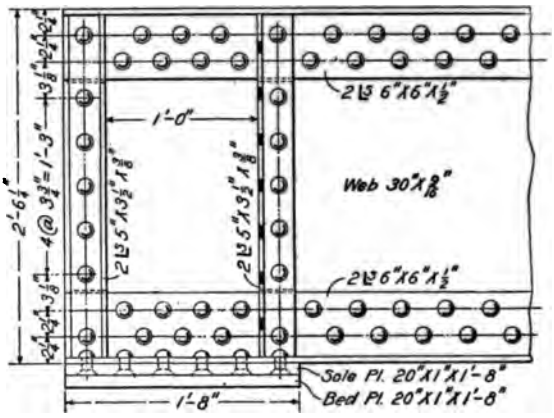


Fig. 12

to a panel length. Otherwise, the end stringers are the same as the intermediate stringers. As shown in Fig. 11, end frames are usually made shallower than the stringers. The tops of the frames are placed below the tops of the stringers that the frames may not interfere with the ties. The bottoms of the frames are placed above the bottoms of the stringers so as to have the former above the bridge seat.

The connection of the outside frames *b*, *b*, Fig. 11, to the trusses will be considered later.

DESIGN OF TRUSSES

STRESSES

31. Instead of finding the actual stresses caused in the ifferent members by the different loadings, as in *Stresses in ridge Trusses*, Part 2, the shears and moments will be found, ıd, after properly combining them, the stresses will be calılated.

32. Live-Load Shears and Moments.—The portion the live load that goes to each truss is one-half of Cooper's 50, as shown in Fig. 2. The portion that the load must cupy in order to cause the greatest shear in a panel or e greatest moment at a panel point is found by means of the inciples explained in *Stresses in Bridge Trusses*, Part 4. The aximum positive live-load shears in the different panels, d the corresponding positions of the loads, are as follows ee Fig. 13):

Panel	Position of Load	Maximum Positive Shear Pounds
a b	3 at *b*	207,850
b c	3 at *c*	156,910
c d	3 at *d*	111,400
d e	2 at *e*	72,220
e d'	2 at *d'*	42,570
d' c'	2 at *c'*	20,310
c' b'	1 at *b'*	4,170

The maximum live-load bending moments at the different nel points, and the corresponding positions of the loads, e as follows:

Panel Point	Position of Load	Maximum Bending Moment Foot-Pounds
b	3 at *b*	3,741,300
c	5 at *c*	6,180,000
d	8 at *d*	7,571,900
e	11 at *e*	8,165,600

33. Impact and Vibration.—The amount to be added to each shear and bending moment found above to allow for the effect of impact and vibration depends on the length L of the track that is loaded when the shear or moment is a maximum. When load 1 is off the left end, the entire bridge is loaded, and L is 144 feet; when load 1 is on the bridge, L is the distance from the right end to load 1. For example,

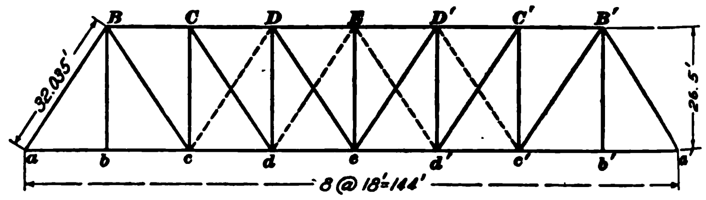


FIG. 13

when the shear in the panel cd is a maximum, load 3 is at d; in this case, since load 1 is 13 feet to the left of d, and d is 90 feet to the left of the right end of the truss, load 1 is 13 + 90 = 103 feet from the right end; that is, L = 103 feet. Applying the formula given in *B. S.*, Art. 25 $\left(I = \frac{300}{L + 300}\right)S$ gives the amounts to be added to the shears and bending moment as follows:

PANEL SHEAR, IN POUNDS

$ab \qquad I = \frac{300}{139 + 300} \times 207{,}900 = 142{,}000$

$bc \qquad I = \frac{300}{121 + 300} \times 156{,}900 = 111{,}800$

$cd \qquad I = \frac{300}{103 + 300} \times 111{,}400 = 82{,}900$

PANEL	SHEAR, IN POUNDS
de	$I = \dfrac{300}{80 + 300} \times 72{,}200 = 57{,}000$
ed'	$I = \dfrac{300}{62 + 300} \times 42{,}600 = 35{,}300$
$d'c'$	$I = \dfrac{300}{44 + 300} \times 20{,}300 = 17{,}700$
$c'b'$	$I = \dfrac{300}{18 + 300} \times 4{,}170 = 3{,}930$

PANEL POINT	BENDING MOMENT, IN FOOT-POUNDS
b	$I = \dfrac{300}{139 + 300} \times 3{,}741{,}300 = 2{,}556{,}700$
c	$I = \dfrac{300}{131 + 300} \times 6{,}180{,}000 = 4{,}301{,}700$
d	$I = \dfrac{300}{133 + 300} \times 7{,}571{,}900 = 5{,}246{,}100$
e	$I = \dfrac{300}{136 + 300} \times 8{,}165{,}600 = 5{,}618{,}500$

34. Dead-Load Shears and Moments.—The dead load consists of the weight of the track and the weight of the bridge. In the general data, it is stated that the floor shall consist of standard ties, and in *B. S.*, Art. **23**, it is specified that this type of floor shall be assumed to weigh 400 pounds per linear foot. The approximate weight w_1 per linear foot of bridge can be found by means of the formula

$$w_1 = 1{,}500\left[1 + \left(\frac{l - 90}{100}\right)^2\right]$$

given in *B. S.*, Art. **242.** In the present case, $l = 144$; therefore,

$$w_1 = 1{,}500\left[1 + \left(\frac{144 - 90}{100}\right)^2\right] = 1{,}937.4 \text{ lb. per linear foot}$$

The total dead load per linear foot is, therefore, 400 + 1,937.4 = 2,337.4 pounds, of which one-half, or 1,168.7 pounds per linear foot, goes to each truss. The dead panel load is equal to 1,168.7 × 18 = 21,036.6 pounds, or, practically, 21,000 pounds; of this load, one-third, or 7,000 pounds, will be treated as applied at each top joint, and the remainder, 14,000 pounds, as applied at each lower joint of the truss. (*B. S.*, Art. **23.**)

The dead-load shears in the different panels are then as follows (see Fig. 13):

Panel	Shear, in Pounds
a b	73,500 (positive)
b c	52,500 (positive)
c d	31,500 (positive)
d e	10,500 (positive)
e d'	10,500 (negative)
d' c'	31,500 (negative)
c' b'	52,500 (negative)
b' a'	73,500 (negative)

The dead-load bending moments at the different panel points are as follows:

Panel Point	Bending Moment, in Foot-Pounds
b	1,323,000
c	2,268,000
d	2,835,000
e	3,024,000

35. Increase in Shears and Moments on Account of Wind Pressure on the Train.—The load on the leeward truss is increased by the overturning effect of the wind. The amount of the increase can be found by the following formula, given in *Stresses in Bridge Trusses*, Part 5:

$$w = \frac{P h}{b}$$

In this formula, P is the intensity of wind pressure, 300 pounds per linear foot (*B. S.*, Art. 27); h is the distance from the center of wind pressure to the lower lateral system, and will be taken as 11.25 feet (see Fig. 10); b is the distance center to center of trusses, in this case 16 feet. Then,

$$w = \frac{300 \times 11.25}{16} = 210.94 \text{ pounds per linear foot}$$

The increase in the panel loads is then $210.94 \times 18 = 3{,}797$, or, practically, 3,800 pounds. The shears due to this increase are as follows:

Panel	Positive Shear, in Pounds
a b	13,300
b c	9,975
c d	7,125
d e	4,750
e d'	2,850
d' c'	1,425
c' b'	475

The bending moments due to this increase are as follows:

Panel Point	Bending Moment, in Foot-Pounds
b	239,400
c	410,400
d	513,000
e	547,200

Those stresses to which the members are subjected as members of the lateral trusses will be considered later, and treated in the same way as for highway bridges.

36. Longitudinal Force.—The longitudinal force due to suddenly stopping trains is taken care of by connecting the stringers to the diagonals of the lower lateral truss wherever they intersect. No calculations are needed for this connection; it is made in the most convenient way. The laterals transmit the force to the trusses, which in turn transmit it to the abutments.

37. Combined Maximum Shears and Moments.—The combined maximum shears are as follows:

Panel	Combined Maximum Shear, in Pounds
a b	207,800 + 142,000 + 73,500 + 13,300 = 436,600
b c	156,900 + 111,800 + 52,500 + 10,000 = 331,200
c d	111,400 + 82,900 + 31,500 + 7,100 = 232,900
d e	72,200 + 57,000 + 10,500 + 4,700 = 144,400
e d'	42,600 + 35,300 − 10,500 + 2,800 = 70,200
d' c'	20,300 + 17,700 − 31,500 + 1,400 = 7,900
c' b'	4,200 + 3,900 − 52,500 + 500 = − 43,900

Since the combined shears in the panels *e d'* and *d' c'* are positive, counters are required in these two panels. Since the combined shear in the panel $c' b'$ is negative, no counter is required in this panel.

The combined maximum bending moments are as follows:

Panel Point	Combined Maximum Bending Moment, in Foot-Pounds
b	3,741,300+2,556,700+1,323,000+239,400 = 7,860,400
c	6,180,000+4,301,700+2,268,000+410,400 = 13,160,100
d	7,571,900+5,246,100+2,835,000+513,000 = 16,166,000
e	8,165,600+5,618,500+3,024,000+547,200 = 17,355,300

38. Stresses in the Members.—Since the depth of the truss is 26.5 feet, the maximum chord stresses, to the nearest 100 pounds, are as follows:

Member	Stress, in Pounds
$a b$, $b c$	$\frac{7,860,400}{26.5} = 296,600$ (tension)
$c d$	$\frac{13,160,100}{26.5} = 496,600$ (tension)
$d e$	$\frac{16,166,000}{26.5} = 610,000$ (tension)
$B C$	496,600 (compression)
$C D$	610,000 (compression)
$D E$	$\frac{17,355,300}{26.5} = 654,900$ (compression)

The length of the diagonal is $\sqrt{26.5^2 + 18^2} = 32.035$ feet, and $\csc H = \frac{32.035}{26.5} = 1.209$. Then, the stresses in the diagonals are practically as follows:

Diagonal	Stress, in Pounds
$a B$	436,600 × 1.209 = 527,800 (compression)
$B c$	331,200 × 1.209 = 400,400 (tension)
$C d$	232,900 × 1.209 = 281,600 (tension)
$D e$	144,400 × 1.209 = 174,600 (tension)
$E d'$ or $d E$	70,200 × 1.209 = 84,900 (tension)
$D' c'$ or $c D$	7,900 × 1.209 = 9,600 (tension)

The stress in the hip vertical is equal to the end shear on the floorbeam, which is 162,300 pounds. Then, the stress in $B b$ is 162,300 pounds, tension.

The stresses in the other verticals are as follows:

Member	Compressive Stress, in Pounds
Cc	232,900 + 7,000 = 239,900
Dd	144,400 + 7,000 = 151,400
Ee	70,200 + 7,000 = 77,200

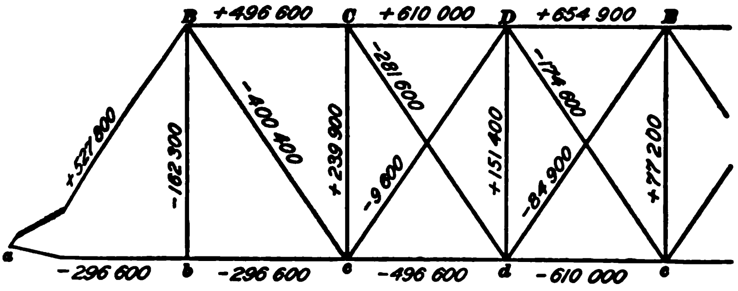


Fig. 14

The above combined stresses are shown on the members in Fig. 14.

DESIGN OF MEMBERS

39. Bottom Chord.—Since the working stress in tension is 16,000 pounds per square inch, the required net areas of the bottom chord members are as follows:

Member	Required Net Area, in Square Inches
ab, *bc*	$\frac{296{,}600}{16{,}000} = 18.54$
cd	$\frac{496{,}600}{16{,}000} = 31.04$
de	$\frac{610{,}000}{16{,}000} = 38.13$

Fig. 15 shows the usual type of member used for the bottom chords of riveted trusses. It consists of two vertical webs *c*, four flange angles *d*, and, when necessary, two side plates *e*. The two sides of the section are connected by double latticing *f* at the top and bottom. The customary method of riveting the parts of the section together is shown in the figure. In computing the net section, allowance must be made for the greatest number of rivet holes in each part of the section. The rivets are $\frac{7}{8}$ inch in diameter.

The usual depth of this type of member varies from about 12 to 24 inches; in the present case, it will be made 15 inches.

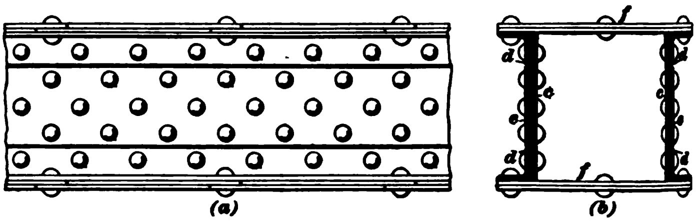


FIG. 15

The clear width between the webs depends on the width of the web members and other details, and is generally from 12 to 24 inches.

40. Design of Bottom Chord Members.—1. *Members a b and b c* (Fig. 14).—The bottom chord will be spliced in panels $c\,d$ and $d'\,c'$ close to the joints d and d', so that the parts that form $a\,b$ and $b\,c$ will also form part of $c\,d$. The following shapes will be used for $a\,b$ and $b\,c$:

2 web plates 15 in. × $\frac{3}{8}$ in.
4 angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{5}{8}$ in.

Three holes must be deducted from each web, and one hole from each angle; then, the net area of the section is as follows:

2 plates 15 in. × $\frac{3}{8}$ in., net section
= 2 (5.625 − 3 × .375) . . . = 9.0 square inches
4 angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{5}{8}$ in., net
section = 4 (3.98 − .625) . . = 13.42 square inches

Total net section . . = 22.42 square inches

This is somewhat more than necessary, but it is desirable to use these sizes in the present case, because, by the addition of two side plates of about the same thickness as the angles, the next member $c\,d$ can be formed.

2. *Member c d.*—As the web is 15 inches deep and the flange angles are $3\frac{1}{2}$ inches in width, the side plates will be made 8 inches in width, and two rivet holes will be deducted

from each plate. The net area of two plates 8 in. × ¾ in. is 2 (6.0 − 2 × .75) = 9.0 square inches; this, added to the net area of *a b* and *b c*, gives 9.0 + 22.42 = 31.42 square inches, which is sufficient.

3. *Member d e*.—The same general form will be used as for *c d*, except that the webs will be made ¾ inch thick instead of ⅝ inch. Then, the net area is as follows:

2 plates 15 in. × ¾ in., net area = 2 (11.25 − 3 × .75) . .	= 18.0 square inches
4 angles 3½ in. × 3½ in. × ⅝ in., net area = 4 (3.98 − .625)	= 13.42 square inches
2 plates 8 in. × ⅝ in., net area = 2 (5.0 − 2 × .625) . .	= 7.50 square inches
Total net area . .	= 38.92 square inches

41. Location of Center Line of Bottom Chord. The center line of the bottom chord will be placed a short distance above the center of gravity of the section, so that the bending moment caused in the chord members by the eccentricity of the stress will neutralize that due to the weight of each member considered as a beam having a span equal to a panel length. The eccentricity will be found for the heaviest member (*d e*), and will be made the same for all the bottom chord members. The gross area of *d e* is 48.4 square inches, and this member weighs 161 pounds per linear foot. To allow for the weight of latticing, the weight will be taken about 10 per cent. greater, or, say, 175 pounds per linear foot. The required eccentricity e, in inches, is computed by the following formula, given in *Bridge Members and Details*, Part 1:

$$e = \frac{w\,l^2}{8\,S} \times 12$$

In the present case, $w = 175$, $l = 18$, and $S = 610{,}000$ (Art. 38); therefore,

$$e = \frac{175 \times 18^2}{8 \times 610{,}000} \times 12 = .14 \text{ inch, or, say, } \tfrac{1}{8} \text{ inch}$$

Then, as the center of gravity is 7⅝ inches from the bottom of the angles, the center line will be 7⅝ + ⅛ = 7¾ inches above the bottom of the bottom angles.

42. Main Diagonals and Counters.—In this article, only the tension diagonals will be designed; the design of the end post will be considered in connection with the design of the top chord. The required net areas are as follows:

MEMBER	REQUIRED NET AREA, IN SQUARE INCHES
Bc	$\frac{400,400}{16,000} = 25.02$
cd	$\frac{281,600}{16,000} = 17.60$
De	$\frac{174,600}{16,000} = 10.91$
dE	$\frac{84,900}{16,000} = 5.31$
cD	$\frac{9,600}{16,000} = .60$

For members requiring more than about 15 square inches net area, the form of cross-section most frequently used is

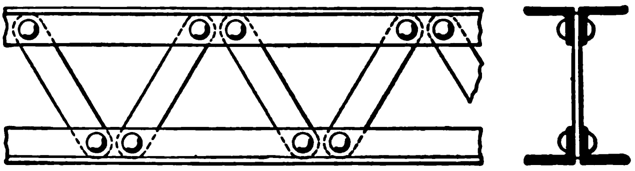


FIG. 16

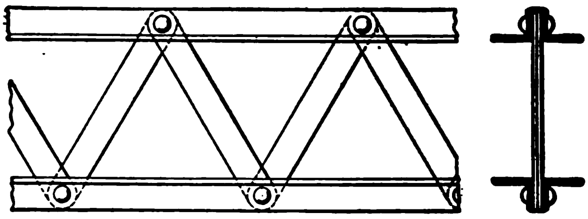


FIG. 17

the same as that of the bottom chord members that have just been considered. For the others, either two or four angles latticed together as shown in Figs. 16 and 17 are used.

1. *Member Bc.*—Where the diagonals are riveted to the connection plates, the web will be reduced by three $\frac{7}{8}$-inch rivet holes, and each angle by one rivet hole. The following section will be used for this member:

	Square Inches
2 plates 15 in. × $\frac{1}{2}$ in., net area = 2(7.5 − 3 × .5)	= 12.00
4 angles 3$\frac{1}{2}$ in. × 3$\frac{1}{2}$ in. × $\frac{5}{8}$ in., net area = 4(3.98 − .625)	= 13.42
Total net area	= 25.42

2. *Member Cd.*—The following section will be used for this member:

	Square Inches
2 plates 14 in. × $\frac{1}{2}$ in., net area = 2(7.0 − 3 × .5)	= 11.00
4 angles 3 in. × 3 in. × $\frac{3}{8}$ in., net area = 4(2.11 − .375)	= 6.94
Total net area	= 17.94

3. *Member De.*—As the required net area is less than 15 square inches, this member will be composed of four angles. The section of each angle will be decreased by one $\frac{7}{8}$-inch rivet hole. The following section will be used for this member:

4 angles 4 in. × 3 in. × $\frac{1}{2}$ in., net area = 4(3.25 − .5)
= 11 square inches

4. *Member dE.*—This member is a counter, and the same type of member will be used as for *De*, except that only two angles are needed, as follows:

2 angles 4 in. × 3 in. × $\frac{1}{2}$ in., net area = 2(3.25 − .5)
= 5.50 square inches

5. *Member cD.*—This member is also a counter, the required net section of which is .60 square inch. According to *B. S.*, Art. **69**, the net area cannot be less than 3 square inches, and if two of the smallest allowable angles are used (3 in. × 3 in. × $\frac{3}{8}$ in., *B. S.*, Art. **39**), the net area will be 2(2.11 − .375) = 3.47 square inches. This is the smallest section that can be used, and will be adopted.

43. Compression Verticals.—The most desirable section for the verticals consists of two channels with their flanges turned toward each other and connected by latticing *f*,

Fig. 18. The easiest way to find the proper sections is to begin at the center of the truss and work toward the end, trying the smallest allowable sections at the center, and increasing them toward the end. The length of a vertical from center to center of chords is 26.5 feet, or 318 inches.

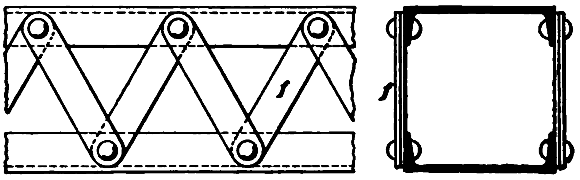


Fig. 18

In *B. S.*, Art. **32**, it is specified that the ratio of the length to the least radius of gyration shall not exceed 100, from which it may be seen that in the present case the least allowable value of the radius of gyration is $\frac{318}{100} = 3.18$ inches. If the channels are placed far enough apart, only the radius of gyration about an axis at right angles to the web need be considered.

1. *Member Ee* (Fig. 14).—Consulting Table XIII, it is seen that the smallest channel having a radius of gyration as great as 3.18 inches is a 9-inch 13.25-pound channel; but this cannot be used, as its web is less than $\frac{3}{8}$ inch in thickness (*B. S.*, Art. **38**). The lightest channel that has the required thickness is a 9-inch 20-pound channel, and this might be used. A 10-inch 20-pound channel, however, also has the required value of radius of gyration and the required thickness, and as it is not heavier and is somewhat stiffer than the 9-inch channel, the 10-inch 20-pound channel will be tried. Its radius of gyration is 3.66 inches, and, therefore, $\frac{l}{r} = \frac{318}{3.66} = 86.9$. The working stress (Table XXXV) is 11,270 pounds per square inch. Since the area of one channel is 5.88 square inches, the total allowable stress on the member is $2 \times 5.88 \times 11,270 = 132,500$ pounds, compression. As this is greater than the stress in *Ee*, two 10-inch 20-pound channels will be used.

2. *Member* Dd.—As the allowable stress in two 10-inch 20-pound channels is less than the actual total stress in Dd, it is necessary to use a heavier section for this member. The next size of channel is a 10-inch 25-pound channel, but it is better to use a 12-inch 25-pound channel, which is somewhat stiffer and no heavier. The radius of gyration of the latter channel is 4.43 inches, and $\frac{l}{r} = \frac{318}{4.43} = 71.8$. The working stress (Table XXXV) is 12,440 pounds per square inch. Since the area of one channel is 7.35 square inches, the total allowable stress on the member is $2 \times 7.35 \times 12{,}440 = 182{,}900$ pounds, compression. As this is greater than the stress in Dd, two 12-inch 25-pound channels will be used for this member.

3. *Member* Cc.—As the allowable stress in two 12-inch 25-pound channels is less than the actual total stress in Cc, it is necessary to use a heavier section for this member. The next size of channel is a 12-inch 30-pound channel; but it is found by trial as above that this is not large enough. Two 15-inch 33-pound channels will be tried. The radius of gyration is 5.62 inches, and $\frac{l}{r} = \frac{318}{5.62} = 56.6$. The working stress (Table XXXV) is 13,580 pounds per square inch. Since the area of one channel is 9.90 square inches, the total allowable stress on the member is $2 \times 9.90 \times 13{,}580 = 268{,}900$ pounds, compression. As this is greater than the stress in Cc, two 15-inch 33-pound channels will be used.

In order that the radius of gyration with reference to an axis parallel to the webs shall be at least equal to that about the axis at right angles to the web, the channels in all these verticals must be $D + 4x$ back to back (see *Design of a Highway Truss Bridge*, Part 1), or, in this case, for member Cc, $9.50 + 4 \times .79 = 12.66$ inches. The actual distance depends on other details, but it will be well in the present case to make it from 12 to 13 inches.

44. Hip Vertical.—The same form of cross-section will be used for this member as for the other verticals.

Since the stress is tension, the required net area is $\frac{162.300}{16,000}$ = 10.14 square inches. Two $\frac{7}{8}$-inch rivet holes will be deducted from the web of each channel. Two 10-inch 20-pound channels will be used. Since the thickness of the web is .38 inch, the area to be deducted for each hole is .38 square inch. Since the gross area of one channel is 5.88 square inches, the net area of two channels is $2(5.88 - 2 \times .38) = 10.24$ square inches, which is sufficient.

45. Top Chord.—The cross-section usually employed for top chord members of riveted trusses is shown in Fig. 19. It consists of a top cover-plate *a*, two web-plates *b*, four flange angles *c*, and, when necessary, two side plates *d*. The angles are connected at the bottom by latticing *f*. The top chord will be spliced in the panels *CD* and *D′ C′* (Figs. 13 and 14), close to points *D* and *D′*, respectively, so that the parts that make up *B C* will continue beyond *C* and form part of *CD*. As the width of the web members is about 13 inches, the clear distance between the webs of the top chord will be assumed as 14 inches, leaving about ½ inch on each side for gussets. The same depth of web will be used as for the bottom chord, that is, 15 inches. As explained in *Bridge Members and Details*, when the clear distance between the webs of this form of section is greater than three-quarters of the width of the web, as in this case, the radius of gyration with respect to an axis parallel to the webs need not be considered, as that referred to an axis at right angles to the web will always be less.

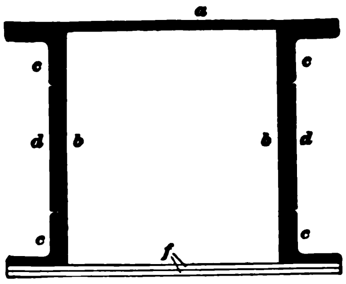


FIG. 19

1. *Member B C.*—The general method of arriving at the section of a top chord member by trial was treated at length in *Design of a Highway Truss Bridge*, and need not be repeated here.

The following section will be used for this member:

	Gross Area, in Square Inches
1 cover-plate 22 in. × $\frac{1}{2}$ in.	11.00
2 top flange angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{7}{16}$ in.	5.74
2 web-plates 15 in. × $\frac{3}{8}$ in.	11.25
2 bottom flange angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{7}{16}$ in.	5.74
Total gross area =	33.73

The center of gravity of this section is 5.06 inches from the top of the top flange angles, and 10.19 inches from the bottom of the bottom flange angles, as shown in Fig. 20. The radius of gyration with reference to an axis at right angles to the webs and passing through the center of gravity is 5.92 inches; $\frac{l}{r} = \frac{216}{5.92} = 36.5$, and the working stress (Table XXXV) is 14,900 pounds per square inch. Then,

Fig. 20

since the total stress in this member is 496,600 pounds, the required area of cross-section is $\frac{496,600}{14,900} = 33.33$ square inches. As the gross area of the section tried above is greater than the required gross area, that section is sufficient, and will be used.

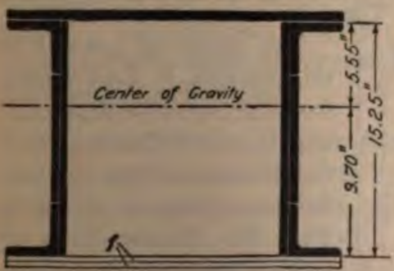


Fig. 21

2. *Member CD*.—The same simple shapes that were used for BC will be used for CD, and, in addition, two side plates 8 in. × $\frac{1}{2}$ in. The gross area of these two plates is 8 square inches, which, added to the area of BC, gives $8 + 33.73 = 41.73$ square inches, gross area of CD. The center of gravity of this section is 5.55 inches from the top of

the top flange angles and 9.70 inches from the bottom of the bottom flange angles, as shown in Fig. 21. The radius of gyration with reference to an axis at right angles to the webs and passing through the center of gravity is 5.51 inches; $\frac{l}{r} = \frac{216}{5.51} = 39.2$, and the working stress (Table XXXV) is 14,740 pounds per square inch. Then, since the total stress is 610,000 pounds, the required area of cross-section is $\frac{610{,}000}{14{,}740} = 41.38$ square inches. As the section tried has a gross area greater than the required gross area, it will be used.

3. *Member DE*.—As the top chord will be spliced near the joint *D*, different shapes can be used from those used in *CD*, but it is advisable to adopt the same width of top plate, depth of web, and width of legs of angles, although different thicknesses may be used. The following sections will be tried for this member:

	Gross Area, in Square Inches
1 cover-plate 22 in. × $\frac{9}{16}$ in.	12.38
2 top flange angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{1}{2}$ in.	6.50
2 web-plates 15 in. × $\frac{3}{8}$ in.	11.25
2 side plates 8 in. × $\frac{1}{2}$ in.	8.00
2 bottom flange angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{1}{2}$ in.	6.50
Total gross area =	44.63

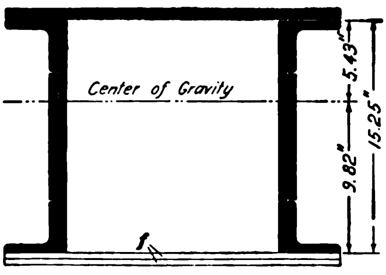


Fig. 22

The center of gravity of this section is 5.43 inches from the top of the top flange angles, and 9.82 inches from the bottom of the bottom flange angles, as shown in Fig. 22. The radius of gyration with reference to an axis at right angles to the webs and passing through the center of gravity is 5.58 inches; $\frac{l}{r} = \frac{216}{5.58}$

= 38.7; and the allowable working stress (Table XXXV) is 14,770 pounds per square inch. Then, since the total stress is 654,900 pounds, the required area of cross-section is $\frac{654,900}{14,770} = 44.34$ square inches. As the section tried has a gross area greater than the required area, it will be used.

46. Location of Center Line of Top Chord.—The center line of the top chord will be placed a short distance below the center of gravity of the chord section, so that the bending moment due to the eccentricity of the stress shall counteract that due to the weight of the member. The eccentricity will be calculated for the heaviest member (DE). The gross area of DE is 44.63 square inches, and the weight of this member per foot is 151.7 pounds. To provide for latticing, the weight will be taken as 160 pounds per linear foot. Then, proceeding as in Art. **41**,

$$e = \frac{160 \times 18^2}{8 \times 654,900} \times 12 = \tfrac{1}{8} \text{ inch, practically}$$

For convenience, it is well to have the center line the same distance from the back of the angles for each member, so that the average location of the center of gravity of BC, CD, and DE will be found. It is $\frac{5.06 + 5.55 + 5.43}{3} = 5.35$ inches, or $5\frac{3}{8}$ inches, below the top of the angles; then, the center line is $5\frac{3}{8} + \frac{1}{8} = 5\frac{1}{2}$ inches below the top of the top flange angles. The different positions of the center of gravity of the top chord members create an objection to the use of unsymmetrical sections.

47. End Post.—In *Design of a Highway Truss Bridge*, it was found necessary to revise the design of the end post after the wind stresses on it had been computed; so, in the present case, the end post will not be designed before the wind stresses have been found. For the purpose of computing the exposed area of the web members, the width of the end post will be taken equal to that of the top chord.

DESIGN OF LATERAL SYSTEM

WIND STRESSES

48. Wind Pressure.—The force to be resisted by the lateral system consists of the wind pressure on the train and that on the trusses, as specified in *B. S.*, Art. **27**. The wind pressure on the train is given as 300 pounds per linear foot of track; that on the trusses depends on the exposed area of the members. To find the exposed area, the length of each member, center to center of joints, will be multiplied by the width as seen in elevation; the latter width will be taken 1 inch greater than the width of web, channel, or angle that forms the member. On this basis, the exposed area of the web members of one truss is 548 square feet; of the top chord, 144 square feet; and of the bottom chord, 192 square feet. The exposed height of the floor will be taken as the distance from the top of the rail to the top of the bottom chord, since the portion of the floor system lower than this is sheltered by the bottom chord. In Fig. 10 (*a*), it can be seen that the exposed height of the floor is about 3 feet; then, the exposed area will be $3 \times 144 = 432$ square feet.

49. Upper Lateral Truss.—The upper lateral truss will be designed to resist the wind pressure on the top chord and one-half that on the web members. The exposed area of the top chord is 144 square feet, and one-half that of the web members is $\frac{548}{2} = 274$ square feet; the combined exposed area is, therefore, 418 square feet. In *B. S.*, Art. **27**, it is specified that the wind pressure shall be taken as 50 pounds per square foot on twice the exposed area of one truss, when, as in this case, it produces greater stresses than any other specified wind loading. Then, the total pressure to be resisted by the top lateral truss is $2 \times 418 \times 50 = 41,800$ pounds. As the top chord is 108 feet long, this corresponds to $\frac{41,800}{108}$, or 387, pounds per linear foot; and, since the panels are 18 feet in length, the panel load is $387 \times 18 = 6,970$ pounds.

The upper lateral truss, with the wind panel loads, is shown

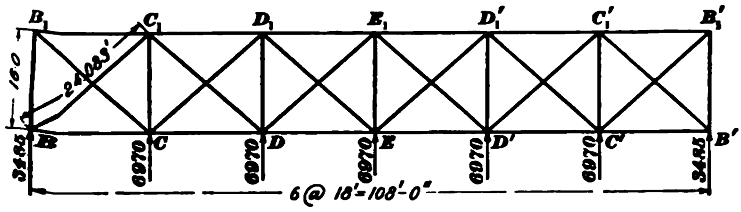


FIG. 23

in Fig. 23. There are five full panel loads and two half panel loads; the latter do not affect the stresses in the truss, but will be considered later in connection with the portal. The stresses in the members, determined in the usual way, are shown in Fig. 24.

50. Lower Lateral Truss.—The lower lateral truss will be designed to resist the wind pressure on the train, and

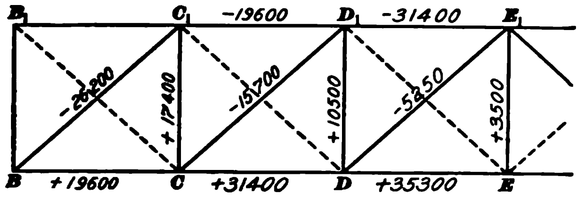


FIG. 24

that on the floor, bottom chord, and one-half the web members. It is necessary in every case to ascertain which of the two alternative loadings given in *B. S.*, Art. 27, causes the greater stresses.

The wind pressure of 300 pounds per linear foot on the train and 30 pounds per square foot on one truss and the floor will first be considered. The panel load due to the pressure on the train is $300 \times 18 = 5,400$ pounds, and will be taken as a live panel load. The exposed area of the floor is 432 square feet; that of the bottom chord, 192 square feet, and that of one-half the web members, 274 square feet; the total exposed area is, then, 898 square feet, the pressure on which, at 30 pounds per square foot, is $30 \times 898 = 26{,}940$ pounds. Since the bottom chord is 144 feet long, this pressure corresponds to $\frac{26{,}940}{144}$, or 187, pounds per linear foot; and, since the panels are 18 feet in length, the panel load is $187 \times 18 = 3{,}370$ pounds. The total panel load, including the wind pressure on the train, is, therefore, $5{,}400 + 3{,}370 = 8{,}770$ pounds.

The alternative wind pressure of 50 pounds per square foot on the exposed area of the floor plus twice the exposed area of one truss will now be considered. The exposed area of the bottom chord is 192 square feet; one-half that of the web members, 274 square feet; and that of the floor, 432 square feet. Then, the total wind pressure on the lower lateral truss is $50 \times [2(192 + 274) + 432] = 68{,}200$ pounds. Since the bottom chord is 144 feet long, this pressure corresponds to $\frac{68{,}200}{144}$, or 473.6, pounds per linear foot, and, since the panels are 18 feet in length, the

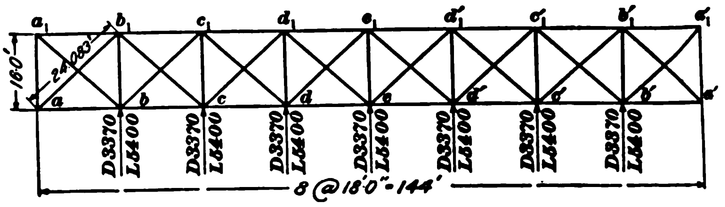


FIG. 25

panel load is $18 \times 473.6 = 8{,}525$ pounds. Since this panel load is less than that found in the preceding paragraph, the latter will be used, as it will cause greater stresses in the members of the lateral system.

The lower lateral truss and wind panel loads are shown in Fig. 25. There are seven full panel loads of 3,370 pounds each, dead wind pressure, and seven panel loads of 5,400 pounds each, live wind pressure. In finding the stresses in the members of the lateral truss, the live panel loads are assumed to be placed so as to cause the greatest stresses in the members, in the same way that vertical live load is con-

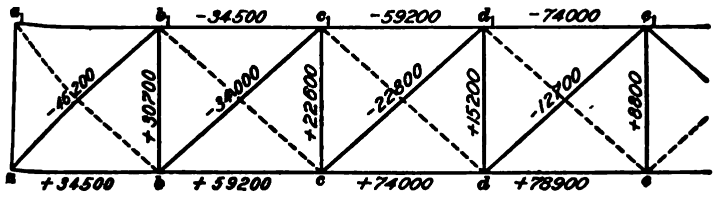


FIG. 26

sidered in finding the stresses in vertical trusses. The stresses caused in the members of the bottom lateral truss by the combined dead and live wind loads are shown in Fig. 26. The half panel loads at *a* and *a'* have not been considered, because they are transmitted directly to the masonry and do not affect the stresses.

51. Transverse Frames. The depth of portal and transverse frames depends on the amount of room above the overhead clearance line. The distance between the center lines of the chords is 26 feet 6 inches; the bottom of the bottom chord is 7¾ inches below the center line, and the top of the cover-plate of the top chord is about 6 inches above the center line. The vertical distance from the bottom of the bottom chord to the top of the top chord is, therefore, 27 feet 7¾ inches, as shown in Fig. 27. Consulting Fig. 10 (*a*), it will be seen that the base of the rail is 3 feet 9⅝ inches above

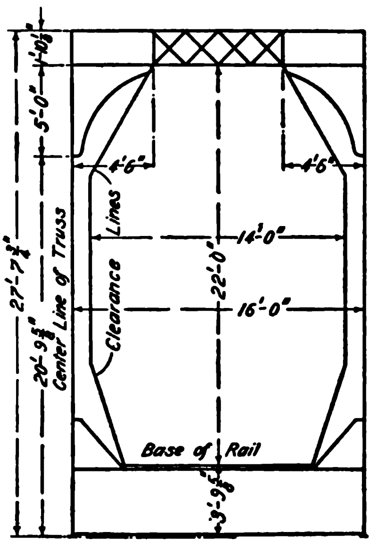


FIG. 27

the bottom of the bottom chord. The overhead clearance line is 22 feet above the base of the rail (*B. S.*, Art. **18**) and 25 feet 9$\frac{5}{8}$ inches above the bottom of the bottom chord. This leaves 27 feet 7$\frac{3}{4}$ inches − 25 feet 9$\frac{5}{8}$ inches = 1 foot 10$\frac{1}{8}$ inches from the top of the top chord to the overhead clearance line. This will be taken as the depth of the transverse frame.

Curved brackets will be placed at each end of each frame. They will be made to extend out as far as the clearance diagram (*B. S.*, Art. 18) will allow; in the present case, 4 feet 6 inches from the center of the truss. They can be made considerably deeper than this, but it is customary to make the depth but little greater than the width. In this case, they will be made to extend 5 feet below the bottom of the frame. The general type of frame and bracket is shown in Fig. 27.

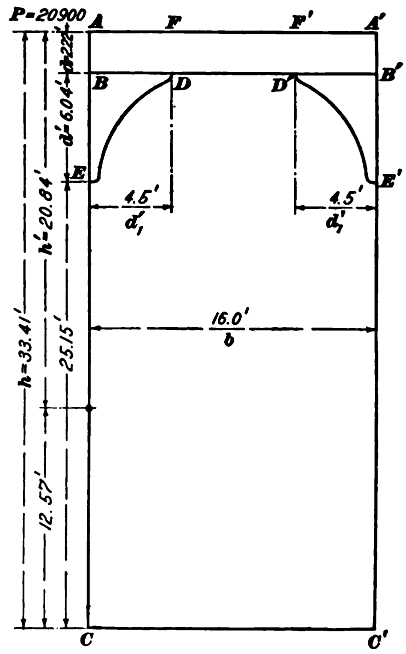


FIG. 28

52. Portal.—Fig. 28 is a plan of the portal, portal brackets, and end posts. The same general type of frame and bracket will be used as for the transverse frames, except that, as the portal is only about 2 feet deep, a plate-girder portal will be used. The portal brackets will extend 4.5 feet out from the center of the end post; the other dimensions shown in Fig. 28 are found from the corresponding dimensions in Fig. 27, by multiplying the latter by csc H (1.209).

The force P that acts at the top of the portal is one-half the total wind pressure on the upper chord, or 20,900 pounds. Making use of the formulas given in *Stresses in Bridge Trusses*, Part 5, assuming the point of inflection of the end post to be half way between the bottom of the bracket and the bottom of the post, the stresses are found to be practically as follows:

Direct stress in end posts,

$$\frac{Ph'}{b} = \frac{20{,}900 \times 20.84}{16} = 27{,}200 \text{ pounds}$$

Bending moment on end posts,

$$\frac{P}{2}(h' - d - d') = 10{,}450 \times 12.58 \times 12 = 1{,}577{,}500 \text{ inch-pounds}$$

Shear on portal,

$$\frac{Ph'}{b} = \frac{20{,}900 \times 20.84}{16} = 27{,}200 \text{ pounds}$$

In the plate-girder portal, the flanges are assumed to resist the entire bending moment, and the required area at any section is found by means of the formula $A = \frac{M}{s\,h_g}$, in which M is the moment about a point in the opposite flange directly opposite the section under consideration. The formula for moments about points in the top flange is

$$M = \frac{Ph'}{2} - \frac{Ph'x}{b}$$

At F, $x = 4.5$, and at F', $x = 11.5$; then,

$$\text{moment at } F = \frac{20{,}900 \times 20.84}{2} - \frac{20{,}900 \times 20.84 \times 4.5}{16}$$

$$= 95{,}300 \text{ foot-pounds;}$$

$$\text{moment at } F' = \frac{20{,}900 \times 20.84}{2} - \frac{20{,}900 \times 20.84 \times 11.5}{16}$$

$$= -\,95{,}300 \text{ foot-pounds}$$

The formula for moments about points in the bottom flange is

$$M = \frac{Ph'}{2} + \frac{Pd}{2} - \frac{Ph'x}{b}$$

At D, $x = 4.5$, and at D', $x = 11.5$; then,

$$\text{moment at } D = \frac{20{,}900 \times 20.84}{2} + \frac{20{,}900 \times 2.22}{2}$$

$$- \frac{20{,}900 \times 20.84 \times 4.5}{16} = 118{,}500 \text{ foot-pounds;}$$

$$\text{moment at } D' = \frac{20,900 \times 20.84}{2} + \frac{20,900 \times 2.22}{2}$$
$$- \frac{20,900 \times 20.84 \times 11.5}{16} = -72,100 \text{ foot-pounds}$$

The value of h_t is not known, but may be assumed as a little less than d; in this case, it will be assumed as equal to 24 inches. Since the above moments are of one kind when the wind blows in the direction shown, and of the other kind when it blows in the opposite direction, the stresses in the flanges reverse, and in finding the required area of flange, the greatest moment in each flange must be increased by .8 of the moment at the other point in the same flange (*B. S.*, Art. **34**). The greatest moment in the lower flange is at D. Then, the required area of the upper flange is $\frac{(118,500 + .8 \times 72,100)12}{16,000 \times 24} = 5.51$ square inches. The moments at the two ends of the upper flange are the same. Then, the required area of the lower flange is

$$\frac{1.8 \times 95,300 \times 12}{16,000 \times 24} = 5.36 \text{ square inches}$$

There is also a direct stress caused in the bottom chords of the trusses by the direct stress in the end post; it is found by multiplying $\frac{Ph'}{b}$ by cos H, which gives 27,200 $\times$ .5619 = 15,300 pounds, compression on the windward, and tension on the leeward, side; only the latter stress need be considered, since the former simply decreases the tension in the bottom chord.

DESIGN OF MEMBERS

53. Upper Lateral Truss.—Comparing the wind stresses in the top chord members as given in Fig. 24 with the combined dead- and live-load stresses as given in Fig. 14, it is seen that the former are in no case as great as 25 per cent. of the latter, so that they need not be considered.

For the diagonals, it is well first to find the allowable stress in the smallest member that can be used. In *B. S.*, Art. **86**, it is specified that no member of a lateral truss shall be less than $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in. Allowing for one $\frac{7}{8}$-inch rivet hole, the net area of one angle of these dimensions is found to be 2.48 − .38 = 2.10 square inches. Since the working stress in tension is 16,000 pounds per square inch, the allowable stress in one angle is 2.1 × 16,000 = 33,600 pounds. As this is greater than the stress in any diagonal of the upper lateral truss, one $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle will be used for each member.

The transverse strut will be made as shown in Fig. 29, which is a customary form. Each flange will be composed of two $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles connected by double latticing of $3'' \times \frac{3}{8}''$ bars. In the present case, as the greatest stress

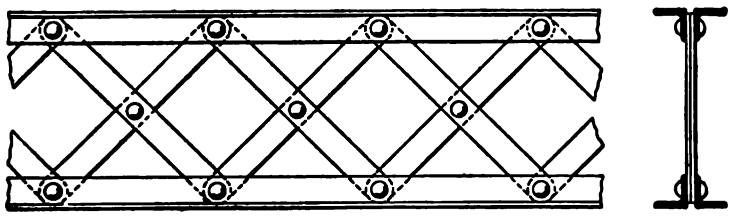


Fig. 29

in a transverse strut is 17,400 pounds (member CC_1, Fig. 24), these angles are sufficiently large.

54. Lower Lateral Truss.—In the design of the upper lateral truss, it was found that the allowable stress in one $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle is 33,600 pounds. As this is greater than the stresses in the diagonals in the panels *cd* and *de*, Fig. 26, one $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle will be used for each of the diagonals in these panels.

In the panel *bc*, the required net area of each diagonal is $\frac{34,000}{16,000} = 2.13$ square inches. One $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$ angle will be used (net area = 2.87 − .44 = 2.43 square inches).

In the panel *ab*, the required net area of each diagonal is $\frac{46,200}{16,000} = 2.89$ square inches. One $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{9}{16}''$ angle will be used (net area = 3.62 − .56 = 3.06 square inches).

The wind stresses in the lower chord members are less than 25 per cent. of the combined dead- and live-load stresses, and so need not be considered.

55. Portal.—The shear on the portal was found in Art. **52** to be 27,200 pounds. This is treated in exactly the same way as the shear on any other plate girder. A web $\frac{3}{8}$ inch thick will be used; the depth 2.22 feet will, for convenience, be called 26 inches. Then, the intensity of shearing stress is $\frac{27,200}{26 \times \frac{3}{8}} = 2,790$ pounds per square inch. Consulting Table XXXVI, it is seen that the allowable unsupported distance is 37 inches; so that no stiffeners are required.

The required net area of the top flange is 5.51 square inches, and that of the lower flange is 5.36 square inches (Art. **52**). It is customary to make the section of the top and bottom flanges the same. Two $5'' \times 3'' \times \frac{7}{16}''$ angles will be used for each [net area = $2(3.31 - .44) = 5.74$ square inches].

The required pitch of flange rivets, computed by the formula used in Art. **13**, comes out greater than 6 inches, so that a pitch of 6 inches will be used (*B. S.*, Art. **41**).

56. Design of End Post.—The combined dead- and live-load stress in the end post is shown in Fig. 14 to be 527,800 pounds. The direct stress due to the wind is 27,200 pounds; the total stress is, therefore,

$$527,800 + 27,200 = 555,000 \text{ pounds}$$

In addition, there is a bending moment due to the wind equal to 1,577,500 inch-pounds (Art. **52**). The following section will be used:

	Gross Area, in Square Inches
1 cover-plate 22 in. $\times \frac{5}{8}$ in.	13.75
2 top flange angles $3\frac{1}{2}$ in. $\times 3\frac{1}{2}$ in. $\times \frac{5}{8}$ in.	7.96
2 web-plates 15 in. $\times \frac{1}{2}$ in.	15.00
2 side plates 8 in. $\times \frac{5}{8}$ in.	10.00
2 bottom flange angles $3\frac{1}{2}$ in. $\times 3\frac{1}{2}$ in. $\times \frac{5}{8}$ in.	7.96
Total gross area =	54.67

b-plate 15 in. $\times \frac{7}{16}$ in., $\dfrac{6.56 \times 14{,}740}{5{,}410} = 17.9$, say 18, rivets

le plate 14 in. $\times \frac{1}{2}$ in., $\dfrac{7.00 \times 14{,}740}{5{,}410} = 19.1$, say 20, rivets

ttom plate $3\frac{1}{2}$ in. $\times \frac{3}{8}$ in., $\dfrac{1.31 \times 14{,}740}{5{,}410} = 3.6$, say 4, rivets

ıe splice is shown in Fig. 31, in which (*a*) is the top view,

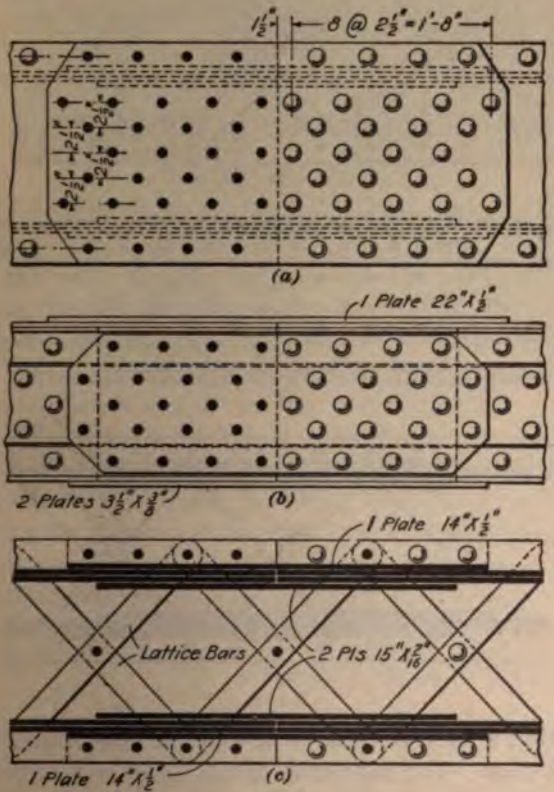
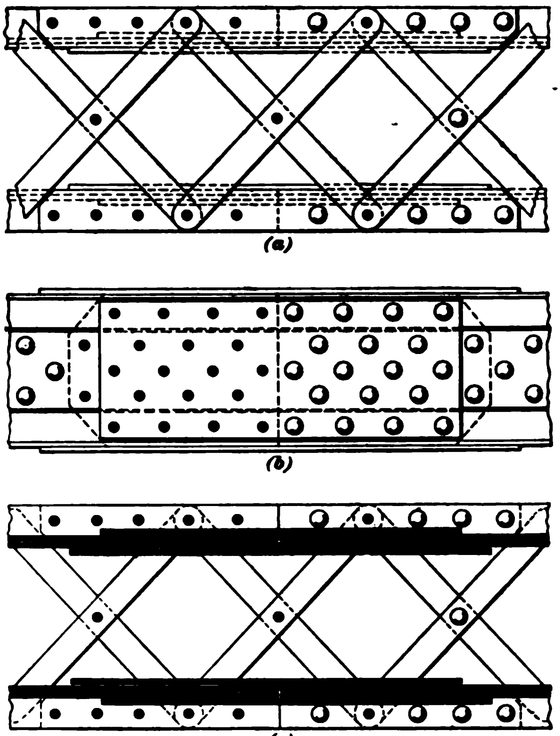


Fig. 31

he elevation, and (*c*) the section and bottom view. The ct number of rivets is shown in each splice plate; it is mary to space the rivets in splices from 2 to 3 inches when staggered, as shown; in the figure, the pitch is about $2\frac{1}{2}$ inches The latticing is continued right along the splice, in the same way as for the rest of the member.

58. Splice in Bottom Chord.—The bottom chord will be spliced in panel *c d* (Fig. 14) just to the left of *d*. The method of splicing is precisely the same as for the top chord



(a)

(b)

(c)

FIG. 32

except that in this case net area must be considered instead of gross area. The following splice plates will be used:

			Net Area, in Square Inches
4 plates 4 in. × $\frac{9}{16}$ in.,	4(2.25 − .56)	=	6.76
2 inside plates 15 in. × $\frac{9}{16}$ in.,	2(8.44 − 3 × .56)	=	13.52
2 outside plates 14 in. × $\frac{1}{2}$ in.,	2(7 − 3 × .50)	=	11.00
	Total area	=	31.28

The number of rivets required to connect each of the splice plates on each side of the joint is as follows:

$$1 \text{ plate } 4 \text{ in.} \times \tfrac{9}{16} \text{ in., } \frac{1.69 \times 16{,}000}{5{,}410} = 5.0 \text{ rivets}$$

$$1 \text{ plate } 15 \text{ in.} \times \tfrac{9}{16} \text{ in., } \frac{6.76 \times 16{,}000}{5{,}410} = 20 \text{ rivets}$$

$$1 \text{ plate } 14 \text{ in.} \times \tfrac{1}{2} \text{ in., } \frac{5.5 \times 16{,}000}{5{,}410} = 16.3 \text{ rivets}$$

The splice is shown in Fig. 32, in which (*a*) is the top view, (*b*) the elevation, and (*c*) a section and bottom view. The required number of rivets is shown in each splice plate.

TRUSS JOINTS

59. The general method of connecting the members of riveted trusses to each other by means of connection plates or gussets has been explained in *Bridge Members and Details*. The connection of the members to each other at the different joints of the truss under consideration will now be discussed. The rivets that connect the members to the gussets are $\frac{7}{8}$ inch in diameter, and are part shop driven and part field driven. The value of each rivet in single shear is the smallest and the only one that need be considered. As most of the rivets are field driven, the value for field-driven rivets, which is 5,410 pounds, will be used, and the number of rivets required to transmit the stress to and from each member will be first calculated. These numbers are as follows:

Member	Number of Rivets
BC	$\frac{496{,}600}{5{,}410} = 92$, or 46 on each side
aB	$\frac{527{,}800}{5{,}410} = 98$, or 49 on each side
Bc	$\frac{400{,}400}{5{,}410} = 74$, or 37 on each side
Cd	$\frac{281{,}600}{5{,}410} = 52$, or 26 on each side
De	$\frac{174{,}600}{5{,}410} = 33$, or 17 on each side

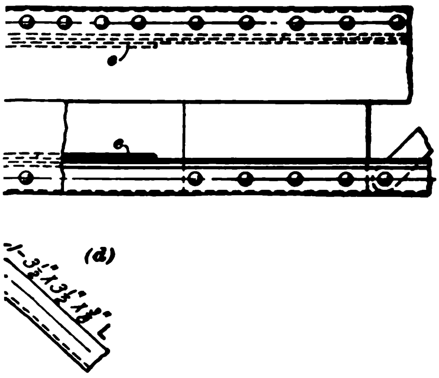
Member	Number of Rivets
dE	$\frac{84,900}{5,410} = 16$, or 8 on each side
ab	$\frac{296,600}{5,410} = 55$, or 28 on each side
Bb	$\frac{162,300}{5,410} = 30$, or 15 on each side
Cc	$\frac{239,900}{5,410} = 45$, or 23 on each side
Dd	$\frac{151,400}{5,410} = 28$, or 14 on each side
Ee	$\frac{77,200}{5,410} = 14$, or 7 on each side

As cD was made larger than necessary, to conform to the rule that no counter shall have an area less than 3 square inches, the number of rivets will be found for the stress that the section chosen can transmit at 16,000 pounds per square inch. Since the net area of cD is 3.47 square inches (Art. 42), the required number of rivets is

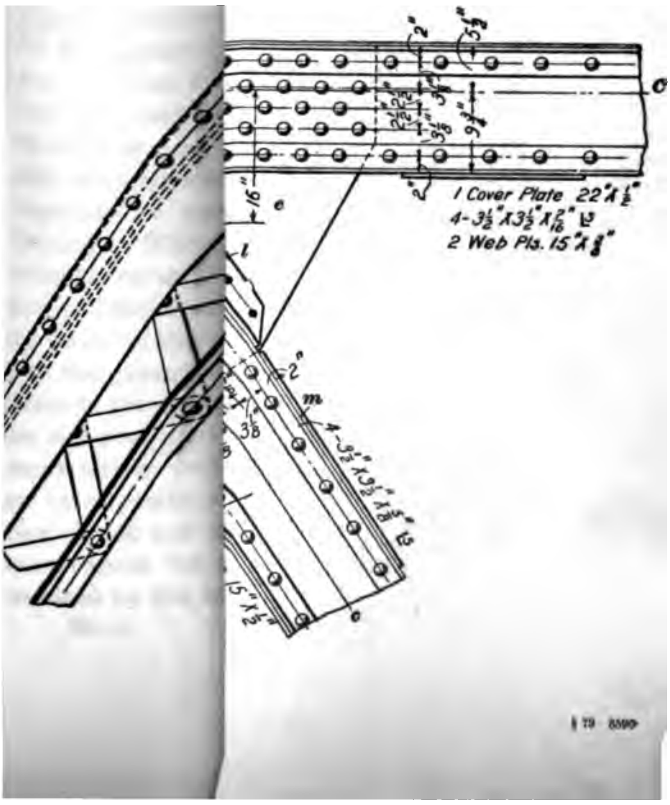
$$\frac{3.47 \times 16,000}{5,410} = 10.3, \text{ or, say, 6 on each side}$$

60. Joint *B*.—Fig. 33 shows joint *B*: (*a*) is the elevation of the joint, and shows one of the gussets *e* that are riveted to the inside of the top chord and end post, and to the ends of the members aB, Bb, Bc, and BC that meet at the joint; (*b*) is the elevation of the end of the portal *g* and portal bracket *h* where they connect to the end post; (*c*) is the top view of the end of the top angles of the portal, and shows the bent plate *i* that connects the angles to the top chord of the truss; (*d*) is the plan of the top chord, and shows in section at the lower right-hand side a portion of the bottom flange of the top chord. The lines aB, Bb, Bc, and BC are the center lines of the different members, and meet at the point *B*; the end post and top chord are cut off on the line ff that bisects the angle between them.

In drawing a riveted joint, the center lines of the members are first drawn, and then the lines that represent the members



(d)



1 Cover Plate 22"×½"
4-3½"×3½"×$\frac{7}{16}$" Ls
2 Web Pls. 15"×⅜"
4-3½"×3½"×⅜" Ls
15"×½"
16"
2"
9¼"

themselves are drawn parallel to the center lines. The gauge lines of the rivets are next drawn, and the rivets are spaced on these lines according to the rules for rivet spacing. In locating the gauge lines of angles, the distances to be used are the standard distances given in Table XII; in locating gauge lines on plates, two lines are first drawn parallel to the edges of the plate and 1½ or 1¾ inches from them, as *j*, *j* in the end post. If they are more than about 4½ inches apart, one or more lines from 2¼ to 4 inches apart are drawn between them, as in the case of the top chord. The rivets are located on these lines and staggered in such manner that they will be not nearer to each other than 3 inches; they are in general spaced closer in compression members than in tension members.

The pitch of rivets in the end post, Fig. 33 (*a*), is about 1⅜ inches; in the top chord, 2¼ inches; and in the main diagonal, about 2½ inches. When convenient, lug angles *l* are riveted to the outstanding legs of the angles of the main members, so as to spread or distribute the stress over a greater width of gusset. Plates ½ inch thick will be used for the gussets *e*, as they are found to furnish sufficient section. One-half of *Bc* is placed outside of the gusset on each side; the two halves are connected below the gusset by tie-plates *m* and lattice bars. The two channels that compose the vertical *Bb* are inserted between the gussets and connected to each other below them by tie-plates *n* and latticing. There are more rivets than necessary in this member, on account of the fact that the rivets in the end post and main diagonal control the size of the gusset, and the rivets in the vertical are spaced about 3½ inches apart to hold the gussets and channels together more tightly. The rivets in the upper end of the vertical are also counted with the end post and top chord. Some designers prefer not to count a rivet twice; but when, as in this case, the members are on opposite sides of the gusset, there is no reason why they should not be counted; in such a case, however, allowance should be made by providing more rivets than are required by the computation.

In order to save extra work in handling and gussets are usually riveted in the shop to one of t In Fig. 33, they are shown riveted to the top cl account less rivets are required, since the valı driven rivet is greater than that of a field-driven

61. The cross-section of the portal is sl Fig. 33 (*a*). The lowest point *p* of the lower portal is made level with the bottom of the tran of the intermediate panel points. The top flang placed high enough to continue right across th as shown at *q*, and are connected to it by mean plates *i* and *r*. The web of the portal is s Fig. 33 (*b*), and the portion toward the truss down to form the web of the bracket. This pc nected to the end post by means of the connect which are usually $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{3}{8}$ in. The ri connection angles are spaced about 6 inches apa the upper end, where those which also transm from the gusset to the end post are spaced 3 in

The diagonal in the end panel of the top la connected to the bent plate *i*, as shown at *u* in a lug angle being riveted to the main angle.

62. Joint *C*.—Fig. 34 shows the connectic (*a*) is the elevation of the joint, showing one of and portions of the members *BC*, *CD*, *Cc*, and *Cd*; a cross-section *h h* of the strut and the connectio to the vertical; (*b*) is an elevation of an interm strut and bracket; (*c*) is a top view of the to shows the connection of the diagonals *i*, *i* of the truss to the chord by means of the plate *f*. At t vertical, which lies between the gussets, a diaph eted between the channels, in order to distribu between the sides of the truss. The same r regard to spacing of rivets, etc. apply here as i joint *B*. There is a small excess of rivets in but this is a frequent occurrence, and is done to ; cing in the gussets. In the remaining top chor

cross-strut and bracket connection will not be shown, but the holes for the connection of the strut to the vertical will be

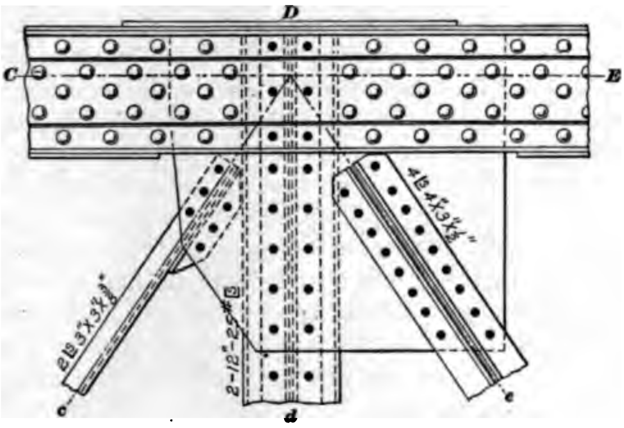


FIG. 35

shown. In Fig. 34 (*c*), a short portion of the top chord at the lower right-hand side is given in section, so as to show the tie-plate *j* and part of the lattice bar *k*.

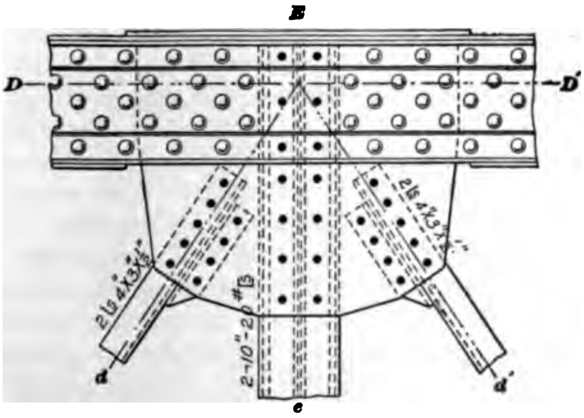
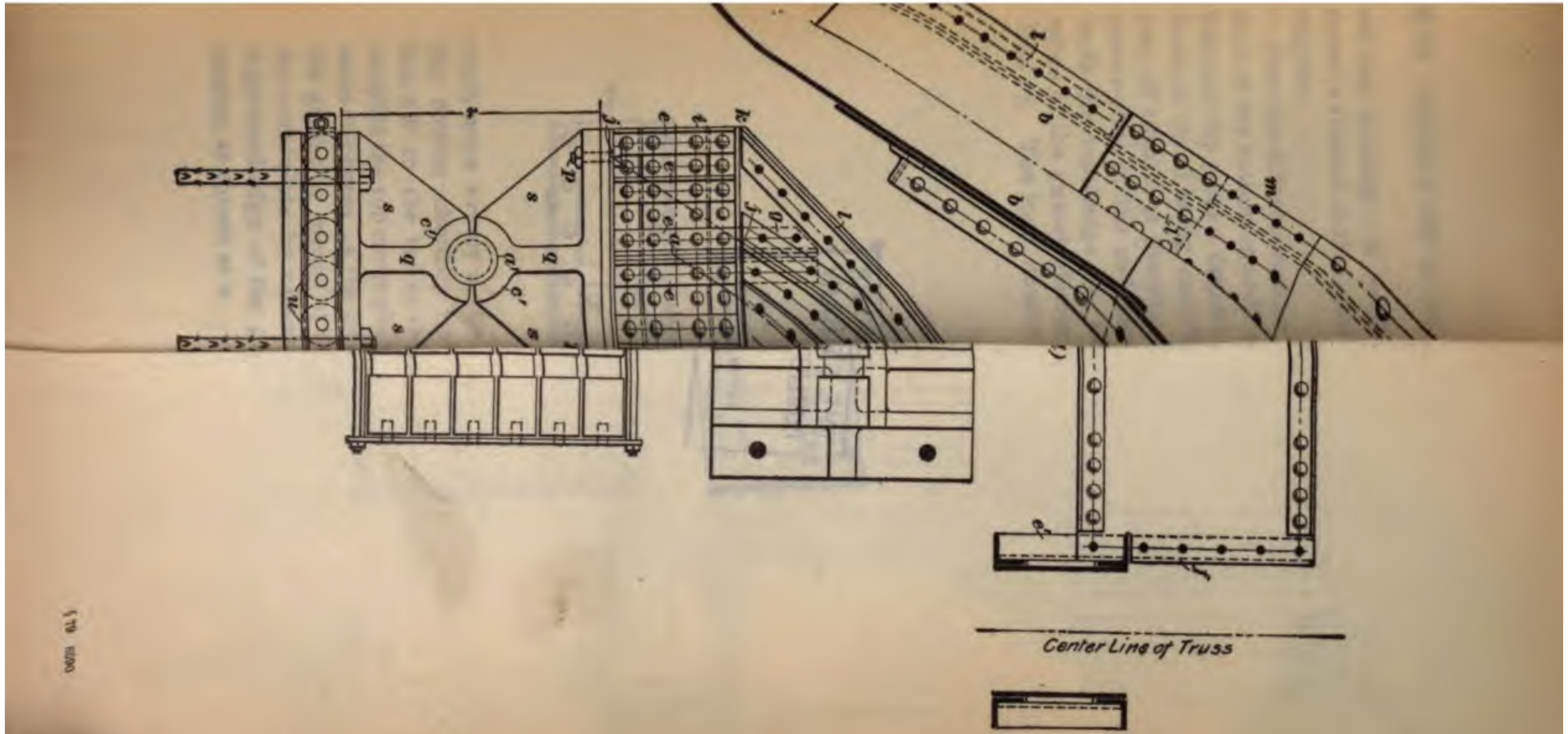


FIG. 36

63. Joints *D* and *E*.—Figs. 35 and 36 represent the joints at *D* and *E*, respectively. The connection of the laterals and of the transverse frame and bracket is the same here as at joint *C*. In Fig. 35, the counter *cD* at the left is placed inside the gussets, so as to clear the main diagonal *Cd* where they cross each other at the center of the panel. For the same reason, counter *dE*, Fig. 36, is placed inside the gusset so it will not interfere with the main diagonal *De*. The counters consist of two angles each, one on each side, and it is customary to place the angles so that the center of gravity shall coincide with the center line of the member. This is frequently done in the case of laterals that consist of single angles; but in the latter case it is customary to connect them as shown in Fig. 34 (*c*), making the back of the angle coincide with the center line. Diaphragms are placed inside the verticals *Dd* and *Ee* at the ends, and the same spacing of rivets is used as for *Cc*, so that the connection of the transverse struts will all be the same.

64. Joint *a*.—Fig. 37 represents the joint *a*. In this figure, (*a*) is the elevation, and shows one of the gussets *b* that connect the end post and the bottom chord, both these members being placed outside the gussets. The center lines and gauge lines are first laid off, and then the rivets are spaced along these lines. Those in the end post are spaced about 2¼ inches apart and staggered as shown, until a sufficient number of rivets is obtained, when the gusset is cut off square with the end post, as shown at *c*. The right-hand edge of the gusset is then carried vertically downwards into the bottom chord, as shown at *d*, and rivets are spaced about 3 inches apart from *d* to the end of this member. It is seldom necessary to count the numbers of rivets in the bottom chord connection, for the above spacing invariably gives sufficient rivets. Stiffeners *e* are placed both outside and inside of the connection over the pedestal to transmit the load to the bearing. The stiffener *f*, together with *e'*, serves for the connection of the end frame, as shown at (*d*). The other end of this frame connects to the end stringer. At the center of the shoe, a diaphragm *g*, shown at (*a*) and (*c*), consisting of a web and four angles, is riveted to the inside


Center Line of Truss

f the gussets. A sole plate *h* about 1 inch in thickness is riveted to the outstanding legs of the bottom flange gles.

Practice differs considerably as to the method of finding the d of the bottom chord and the gusset. The length of truss yond the point *a* depends on the required size of shoe, the ttom chord being carried out to the end of the shoe and t off square, as shown at *i*. Some designers continue the ver-plate of the end post straight down to its intersection th the top flange of the bottom chord, and continue the ge of the gusset along that line, as shown by the dotted e *j j*. The load, however, is distributed over the bearing

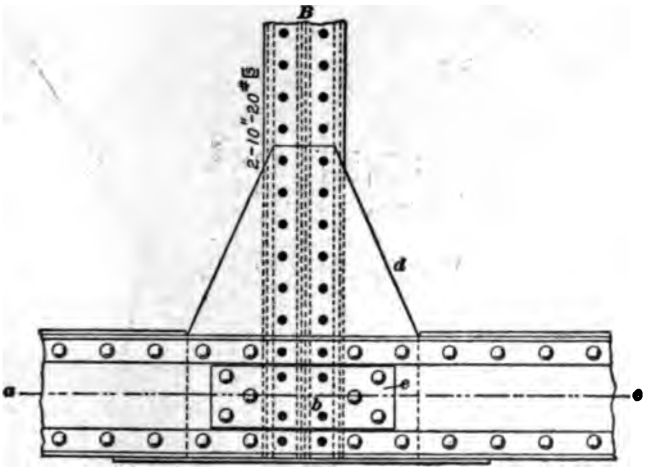


FIG. 38

uch more evenly if the gusset is carried up to the top of e bottom chord at its end and then straight up on the ne *k k'* to the top of the gusset. This makes it necessary splice the top cover-plate of the end post at *k'*; a plate of the ame thickness as the cover-plate is fastened to the edge of e gusset by means of the angles *l* and *l'*, and is spliced to e cover-plate by means of the bent splice plate *m*. The ght-hand edge of the gusset is stiffened by 3″ × 3″ × ⅜″ ngles, as shown at *n*.

Fig. 37 (*b*) is a plan of the end post; the upper left-hand half is a top view of the cover-plate, and the lower right-hand half is a section through the gusset, and a plan of the bottom flange of the end post. The pedestal and rollers will be discussed later.

65. Joint *b*.—Fig. 38 shows joint *b*. No stress is transmitted from one member to another at this joint, except the load from the floorbeam to the vertical, so that no gusset is

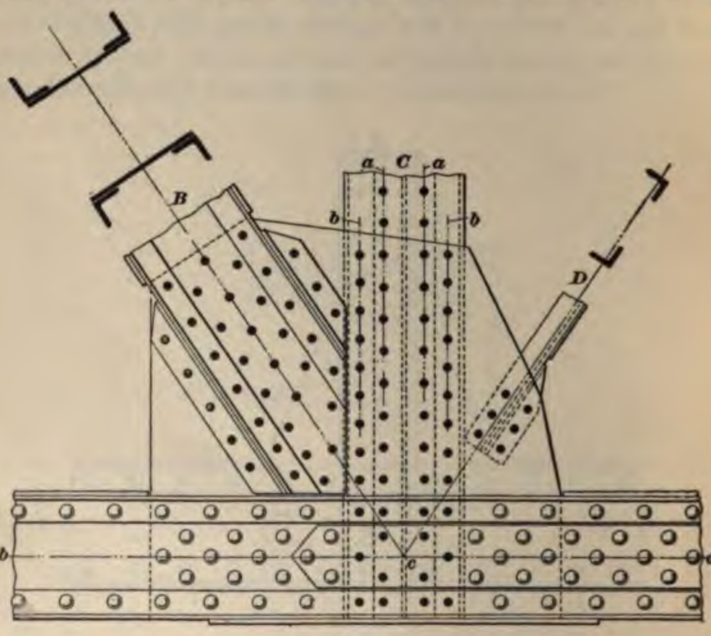


FIG. 39

required to connect the vertical and the chord. It is customary, however, to provide a small gusset *d*, a little wider on each side than the vertical, in order to fill up the space between the vertical and the chord, and to make the joint a little stiffer. The plate *e* is for the purpose of filling up the space between the flange angles of the bottom chord.

66. Joints *c*, *d*, and *e*.—Figs. 39, 40, and 41 show, respectively, joints *c*, *d*, and *e*. These joints are somewhat simpler

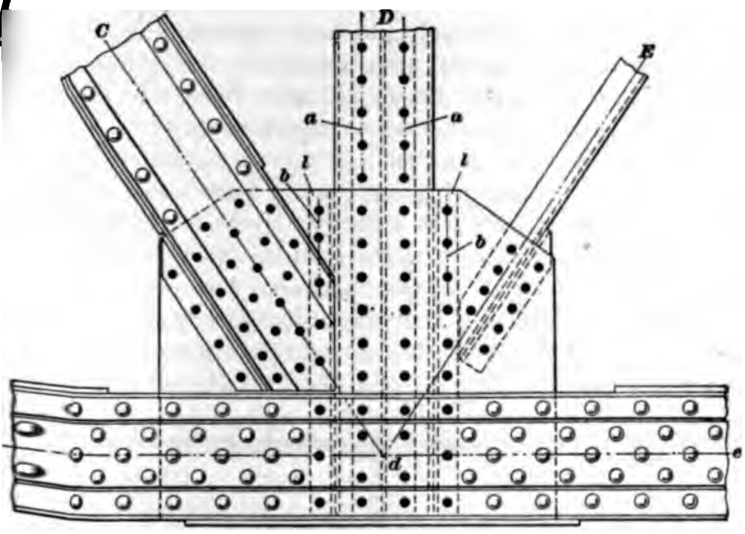


FIG. 40

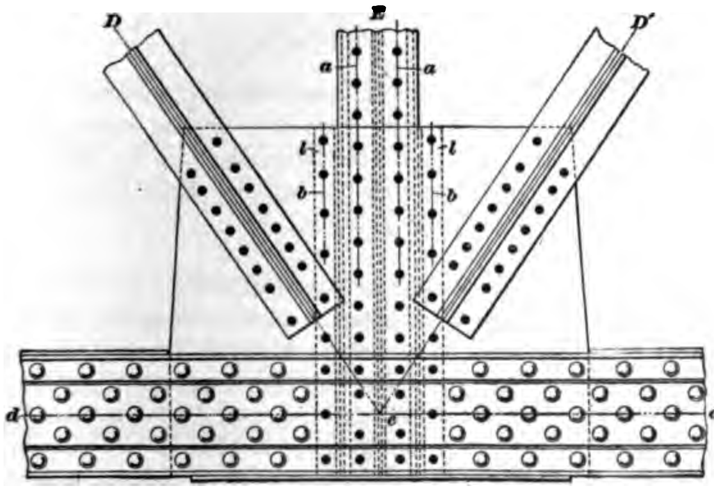


FIG. 41

than joint *a*, but the same general rules are observed in laying them out. The only point in connection with these joints that requires special attention is the connection of the verticals to the gussets. It should be remembered that the rivets in the two rows *a*, *a* near the center of the vertical transmit the load from the floorbeam to the truss, and should not be counted as transmitting any stress from the vertical to the gusset; there should be sufficient rivets in the lines *b*, *b* outside of these two lines to transmit this stress. When, as in the case of *Cc*, Fig. 39, the channel is 15 inches in width, additional rows can be driven through the web of the channel on each side of the floorbeam connection angles. When, as in the case of *Dd*, Fig. 40, and *Ee*, Fig. 41, the channel is

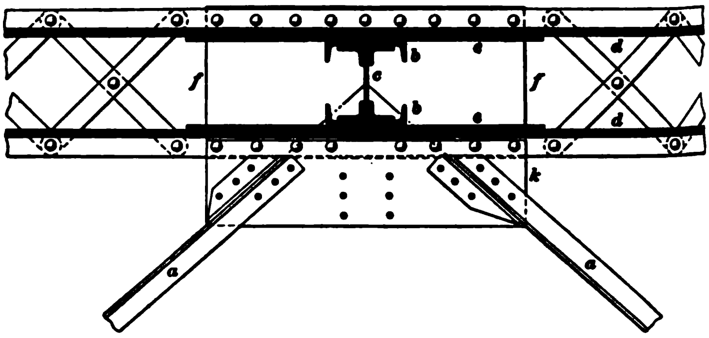


FIG. 42

not wide enough to allow extra rows to be driven, lug angles *l* are riveted to the flanges of the channels, and the required number of rivets is driven in the legs of these lug angles that are in contact with the gussets.

67. Bottom Lateral Connection.—Fig. 42 shows a connection of the lower laterals *a*, *a* to the lateral connection plate *k* shown in Fig. 10, and the connection of the latter to the truss. The vertical *b b* and the diaphragm *c* are shown in cross-section, as well as the sides *d*, *d* of the bottom chord, and the gussets *e*, *e*. The figure also shows the tie-plate *f f* and lattice bars on the bottom of the bottom chord. The connections at the joints *a*, *b*, *c*, and *d*, Fig. 25, are similar to this connection, except that, where necessary, more rivets are driven and the lateral plate is made somewhat larger. The rivets in the lateral and lug angles are $\frac{7}{8}$ inch in diameter. Both diagonals in each panel are placed on top of the connection plate, in order that they may be in better position to connect to the stringers where they intersect them.

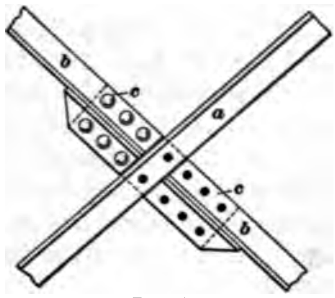


FIG. 43

The method of crossing the laterals at the center of the panel is shown in Fig. 43: one diagonal *a* continues unbroken, and the other *b b* leads up to it on each side, and is spliced by a plate *c c*, which is also riveted to the angle *a*.

The connection to the bottom flange of the stringer is shown in Fig. 44: the lug angle *k* connects the lateral angle *l* with the bottom flange of the stringer *m*.

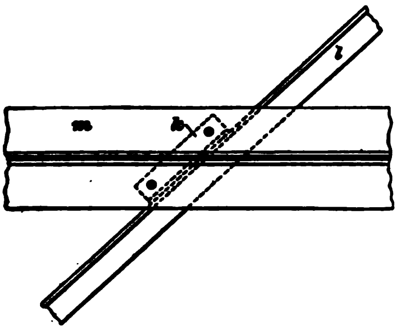


FIG. 44

BEARINGS

68. Required Area. The required area of bearing for each end of each truss is found by dividing the reaction by the allowable or working pressure on the masonry. As there is no end floorbeam, the reaction is equal to the vertical component of the stress in the end post, that is, 436,600 pounds (Art. 37). The allowable pressure on the masonry is given in *B. S.*, Art. 29, as 500 pounds per square inch. Then, the required area is $\frac{436{,}600}{500} = 873$ square inches. If the bed-plate is square, it must be about 30 inches on each side. The actual dimensions depend on other details.

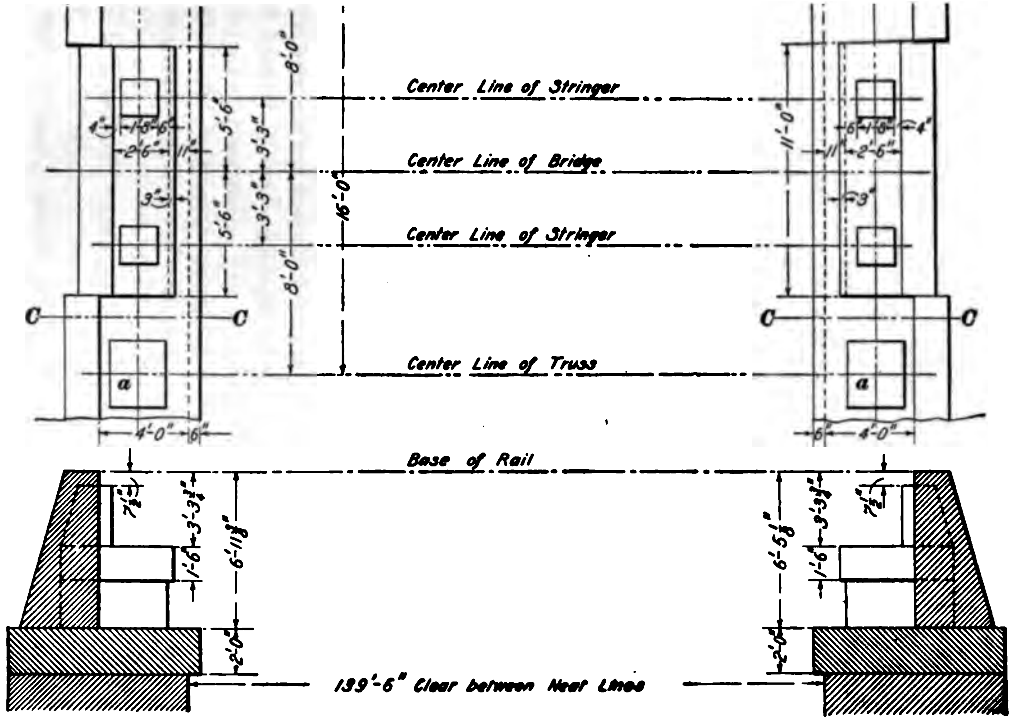
69. Pedestal.—Cast-steel pedestals, as shown in Fig. 37 (*a*), are well adapted to riveted trusses. The top half is made 30 inches long and a little wider than the bottom chord; in this case, it will be made 22 inches wide. It is bolted to the outstanding legs of the lower flange angle by means of 1-inch bolts *p*.

The height of the pedestal depends on the allowable distance from the base of the rail to the top of the bridge seat; it is desirable to keep the top of the bridge seat above high water. When, as in the present case, high water is considerably below the base of the rail (17 feet, Art. **1**), the total height *r* of the pedestal should be made about the same as its length, and the pin *a'* placed half way between the top and the bottom. For a truss of this length, a pin 6 inches in diameter is used.

In Fig. 37 (*a*), the part *b'* of the pedestal in contact with the sole plate *h*, and the corresponding part *b'* of the lower half are made 2 inches thick; the part *c'* in contact with the pin is also made 2 inches thick. The two bearing surfaces of each half of the pedestal are connected by a vertical web *q*, 3 inches thick, along the axis of the pin from one side of the pedestal to the other, and by webs *s*, $1\frac{3}{4}$ inches thick and 4 to 6 inches apart. Projections *t*, Fig. 37 (*c*), are cast on the inside of the circular part of the pedestals; they fit into circular grooves of the same size planed out of the pin, and are for the purpose of holding the pin in place and also preventing halves of the pedestal from moving sidewise. At one end of the bridge the bottom of the pedestal is planed smooth, and a steel plate $\frac{1}{2}$ inch thick is placed between it and the masonry. It is then anchored to the masonry by bolts. Fig. 37 (*e*) is a top view of the lower half of the pedestal.

At the expansion end of the bridge, rollers are placed under the pedestal, as shown at *u*, Fig. 37 (*a*), in order to allow that end to move back and forth as the temperature changes.

70. Rollers.—The allowable pressure per linear inch on rollers is given in *B. S.*, Art. **29**, as 600 *D*. Rollers are



Cross Section at CC

FIG. 45

usually made from 3 to 6 inches in diameter, and about $\frac{1}{4}$ inch is left between each two rollers. In the present case, the diameter of the roller will be made $4\frac{3}{4}$ inches; 6 rollers can be placed under the pedestal, which is 30 inches long. The allowable pressure on the rollers is $600 \times 4.75 = 2{,}850$ pounds per linear inch; therefore, the number of inches required is $\frac{436{,}650}{2{,}850} = 153$. As there are 6 rollers, the length of each must be $\frac{153}{6} = 25\frac{1}{2}$ inches. They are fastened together so as to form a roller nest, in the manner shown in Fig. 37 (*f*).

BRIDGE-SEAT PLAN

71. Bridge Seat for Trusses.—Fig. 45 is a bridge-seat plan of the bridge that has just been designed. The distance between the centers of the pedestals or bedplates *a, a* is shown equal to the span, in this case 144 feet. The front of each pedestal is placed 1 foot from the neat line of the abutment, which makes the neat line 2 feet 3 inches from the center of the pedestal at each end, or 144 feet − 2 feet 3 inches − 2 feet 3 inches = 139 feet 6 inches, between neat lines.

The face of the parapet is placed 6 inches from the back edge of the pedestal, and 1 foot 9 inches from the center of the pedestal; this makes the bridge seat for the truss 4 feet wide from neat line to parapet. It is customary to make the bridge-seat stone 2 feet thick for a span of this length, and to have it project about 6 inches beyond the neat line.

The distance from the base of the rail to the top of the bridge seat is found by adding together the vertical distances occupied by rollers, pedestals, stringers, etc. For the fixed end shown at the right, these distances are as follows:

3 feet $9\frac{5}{8}$ inches, distance from the base of the rail to the bottom of the bottom chord, Fig. 10;

1 inch, thickness of the sole plate on top of the pedestal, Fig. 37 (*b*);

2 feet 6 inches, height of pedestal;

$\frac{1}{8}$ inch, thickness of bedplate under pedestal;

6 feet $5\frac{1}{8}$ inches, total distance from the base of the rail to the top of the bridge seat.

At the expansion end, all the vertical distances are the same as for the fixed end, except that the rollers and bedplates occupy $6\frac{3}{8}$ inches, or $6\frac{1}{4}$ inches more than the $\frac{1}{8}$-inch bedplate under the pedestal at the fixed end. This makes the distance from the base of the rail to the top of the bridge seat at the expansion end 6 feet – $5\frac{1}{8}$ inches + $6\frac{1}{4}$ inches = 6 feet $11\frac{3}{8}$ inches, as shown at the left end.

72. Bridge Seats for Stringers.—As explained in Art. 30, the bedplates under the stringers are made 20 inches square. It is customary to place them so that their centers are directly opposite the centers of the truss shoes. A separate bridge seat and parapet are built up on top of the truss seat to accommodate the stringers. The parapet is usually placed about 4 inches from the end of the stringer, and the neat line 6 inches ahead of the front end of the bedplate, making the stringer bridge seat 2 feet 6 inches wide from face of parapet to neat line; the latter is 11 inches from the neat line of the main abutment.

The distances from base of rail to top of stringer bridge seat are made up as follows:

$7\frac{1}{2}$ inches from base of rail to top of stringer;

2 feet $6\frac{1}{4}$ inches, depth of stringer over flange angles;

2 inches, thickness of sole plates and bedplates;

3 feet $3\frac{3}{4}$ inches, total distance from base of rail to top of stringer bridge.

WOODEN BRIDGES

INTRODUCTION

USES AND TYPES OF WOODEN BRIDGES

1. Disadvantages of Wood for Bridge Construction.—For permanent bridge work of much importance, wood has gone out of use. The principal reasons for this are the necessity of frequent renewal, the increased cost of timber and decreased cost of steel, and the difficulty and delay in securing the proper quality, sizes, and lengths of timber. In addition to this, the danger from fire has played an important part in the substitution of steel for wood in bridge building.

2. Conditions to Which Wooden Trusses Are Best Adapted.—Notwithstanding the above-mentioned disadvantages, there are several conditions under which the use of timber is very desirable. Notable among these are the circumstances that render advisable the use of pile and frame trestles, a trestle being practically a wooden bridge. The subject of trestles is fully treated in another Section of this Course, and need not be further considered. Perhaps the main advantage of a wooden over a steel bridge is its comparatively low first cost. The percentage of saving in first cost is greater in short than in long spans, and, of course, depends on the relative prices of wood and steel, these prices being to a great extent governed by the facilities for procuring the two materials. The difference in the

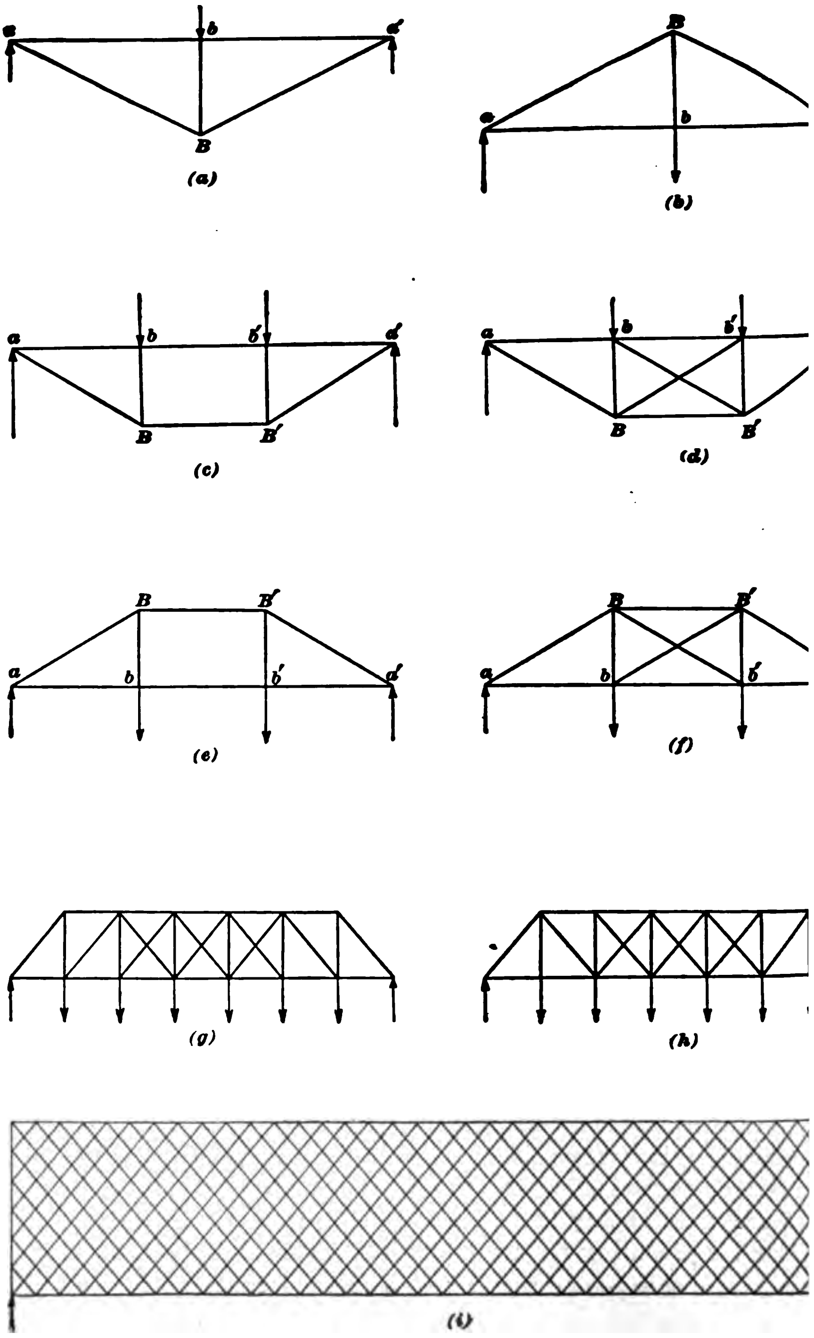


FIG. 1

percentage of saving between short and long spans is due principally to the increased cost of the large-sized timbers required in long spans.

3. Wooden trusses are also used for work of a temporary character, such as a temporary road around a site on which a permanent steel structure is being erected, or the false work necessary for the support of traffic, workmen, and materials during the erection of a metal bridge. They are also frequently used in irrigation work to carry flumes and conduits across openings. In places where timber is easily accessible, it has been found economical in many cases for highway and railroad bridges having spans from 100 to 250 feet.

4. Combination Bridges.—Bridges are occasionally constructed entirely of wood, except the bolts used to connect the members. In the great majority of wooden bridges, however, only the compression members are wood, the tension members being composed of eyebars or built-up shapes, the same as in pin-connected steel trusses. Such bridges are often called **combination** bridges, although the name *wooden bridge* is usually applied both to all-wood and to combination bridges.

5. Types of Wooden Trusses.—The principal types of trusses in which timber is used are shown in Fig. 1. In this figure, (*a*) is a **kingpost** truss, and (*b*) a **kingrod** truss. When the horizontal members ab and ba' serve both as chord members and as stringers to support the load on the floor, these trusses are called **trussed stringers**. The trusses shown at (*c*) and (*d*) are **queenpost** trusses, and those at (*e*) and (*f*) are **queenrod** trusses. Diagonals are sometimes inserted in the center panel, as at (*d*) and (*f*), and are sometimes omitted, as at (*c*) and (*e*). When the horizontal members ab, bb', and $b'a'$ serve both as chord members and as stringers to support the load on the floor, these trusses [(*c*), (*d*), (*e*), and (*f*)] also are called *trussed stringers*. The other types, (*g*), (*h*), and (*i*) are, respectively, **Howe, Pratt,** and **lattice** trusses.

6. Methods of Calculation.—As a rule, the methods employed to calculate the stresses in the members of the trusses shown in Fig. 1 are the same as those explained in the various parts of *Stresses in Bridge Trusses*. Special assumptions are made, however, for some of the types, as will be explained further on.

MATERIALS AND WORKING STRESSES

TIMBER

7. Kinds of Timber.—The kind of timber used for a wooden bridge depends to a great extent on the locality. In the eastern part of the United States, **yellow pine** is employed more than any other kind of timber; **white oak** and **white pine** are also largely used. **Spruce** is used to some extent, but is less desirable for bridge work than any of the other kinds mentioned. In the western part of the United States, **Douglas, or Oregon, fir,** which is very homogeneous and durable, is used to a great extent.

WORKING STRESSES OF TIMBER

(*Pounds per Square Inch*)

Kind of Timber	Kind of Stress					
	Transverse Stress S_b	Compression on Gross Section S_c	Tension on Net Section S_t	Shearing Along the Grain S_s	Bearing Across the Grain S_a	Bearing Along the Grain S_g
Yellow pine	1,200	800	1,000	80	400	1,250
White pine .	750	500	650	50	200	800
Spruce . . .	750	600	800	60	200	800
White oak .	1,000	700	900	100	600	1,250
Douglas fir .	1,200	800	1,000	80	400	1,250

8. Allowable or Working Stresses.—The allowable or working stress in timber depends largely on the seasoning. For the ordinary timber in the market, fairly well seasoned, the working stresses given in the table on page 4 may be used for bridge work.

1. *Transverse Stress.*—The allowable transverse stresses given in the table are the greatest allowable values of s_b, in pounds per square inch, as found by the formula

$$s_b = \frac{Mc}{I}$$

explained in *Strength of Materials.*

2. *Compression.*—The allowable compressive stresses given in the table are the greatest allowable values of s_c, in pounds per square inch, as found by the formula

$$s_c = \frac{S}{A},$$

and are for members whose greatest length is not greater than eight times the least side or diameter. For members whose length is greater than eight times the least width, the working compressive stress s_c', in pounds per square inch, is obtained by means of the following formula:

$$s_c' = \frac{s_c}{1 + .004\left(\frac{l}{d}\right)^2}$$

in which s_c = value given in the table;
l = length of the member;
d = diameter or least side.

For example, if the length of a yellow-pine column is 15 feet, and the cross-section is 10 in. × 12 in.; then, since the least side is 10 inches and the value of s_c given in the table is 800, the allowable working stress is

$$s_c' = \frac{800}{1 + .004\left(\frac{15 \times 12}{10}\right)^2} = 348 \text{ lb. per sq. in.}$$

3. *Tension.*—The allowable tensile stresses given in the table are the greatest allowable stresses s_t as found by the formula

$$s_t = \frac{S}{A}$$

in which A is the net area of the member after deducting bolt holes, cuts at connections, etc.

4. *Shearing Along the Grain.*—The working stresses s_s given in the table for shearing along the grain are the greatest allowable longitudinal shearing stresses at the neutral axis of a beam. For a rectangular beam, the intensity of shear, as found by the formula

$$s_s = \frac{3V}{2bh}$$

must not exceed the working stress given in the table. In this formula, V is the greatest external shear on the beam, and b and h are the width and depth of the beam at the section of greatest shear. In addition, the tabular values are the maximum allowable stresses at the ends of bridge members where they connect to others, as will be explained presently.

5. *Bearing Across the Grain.*—The allowable working stresses s_a given in the table for bearing across the grain are the greatest allowable intensities of bearing on the sides of members where other members at right angles to the first connect to them.

6. *Bearing Along the Grain.*—The allowable working stresses s_r given in the table for bearing along the grain are the greatest allowable intensities of bearing at the ends of compression members, where they rest on other members.

METALS

9. Steel.—Steel rods and shapes are commonly used for the metal portions of combination bridges, and when so used are designed in the same way as for steel bridges. The working stresses are the same as those given in *Bridge Specifications*.

10. Wrought Iron.—Although wrought iron has been almost entirely superseded by steel, it is still used to a slight extent for the metal portions of combination bridges. When so used, the members are designed in the same way as for steel bridges, and the allowable working stresses may be taken as 75 per cent. of those given for steel in *Bridge Specifications*.

KINGPOST AND KINGROD TRUSSES

11. Trusses of the kingpost and kingrod types are especially useful for short spans that are too long for the use of simple beams. They may be made as long as 25 or 30 feet. On account of the lighter loads, they can be used for longer spans for highway than for railroad bridges.

12. Description of Kingpost Truss.—Fig. 2 represents the simplest form of kingpost truss. It consists of a

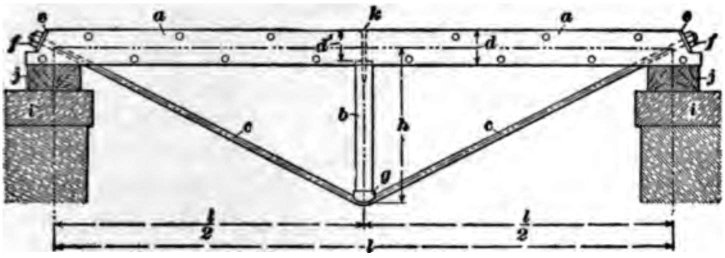


FIG. 2

top chord *a a*, a vertical **kingpost** *b*, which supports the top chord at the center, and a bent rod *c c* connecting the bottom of the kingpost with the ends of the top chord. The kingpost is held in position at the top by means of a wooden pin or iron drift bolt *k*, and also by being set or framed into the top chord. At the bottom of the kingpost, a casting *g* distributes the pressure of the rod *c c* over the end of the post; the bottom of the casting is grooved in order to afford the rod a better bearing. At the ends of the truss, the rod *c c* passes through plates *e*, *e* that bear on the end of the top chord; the ends of the rod are held in place by nuts *f*, *f*. The ends of the chord *a a* usually rest on timber blocking *j*, *j* that is supported by the bridge seat *i* or bent on which the truss rests. The distance *l* between the centers of the blocking on which the truss rests is the span. The post *b* is at the center of the span, making each panel equal to $\frac{l}{2}$. The distance h from the center of the horizontal chord to the intersection of the inclined members with the kingpost is called the **height** of the truss.

13. In some cases, the top chord simply consists of one timber, and the rod passes through it. In other cases, there are two or more timbers, and one less or one more rod than there are timbers, the rods passing between the timbers and in some cases outside of them at each side. Fig. 3 shows the arrangement at the end when there are two timbers and one rod. Large cast-iron washers *c* hold the timbers *f*, *f*, Fig. 3 (*b*), far enough apart to allow the rod *g* to pass between them, and $\frac{3}{4}$-inch bolts *e*, *e*, spaced 2 or 3 feet apart, hold the timbers together and make them act as one piece. The holes for these bolts are usually bored $\frac{1}{8}$ inch less in diameter than the diameter of the bolt, and the bolts are driven in; this makes them fit tight in the wood. Cast-iron washers *d*, *d* are placed on the ends of the bolts to prevent the heads and nuts from sinking into the wood.

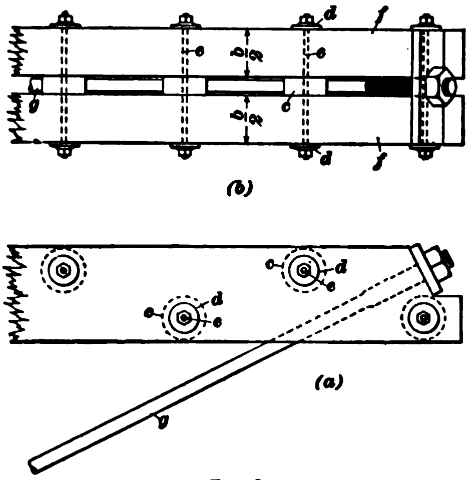


FIG. 3

14. Description of Kingrod Truss.—When there is not sufficient room for the kingpost and bent rod to project below the floor, the kingrod truss shown in Fig. 4 is used. In this truss, the bottom chord *a a* is composed of wood, and is horizontal; the kingrod *b* is made of iron or steel, and the inclined members *c*, *c*, commonly called **struts**, are made of wood. The kingrod passes through the bottom chord and supports its center, the plate *e* and the nut *f* helping to hold the chord in place. The top of the rod passes through and between the ends of the struts *c*, *c*. The nut *f'* and plate *e'* hold this end of the rod in place and transmit the stress in the rod to the struts. At the ends of the truss,

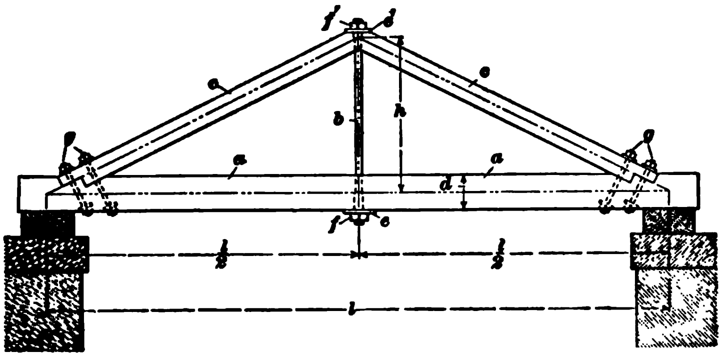


FIG. 4

the top of the bottom chord is notched, and the lower ends of the struts are framed so as to fit into these notches. To assist in transmitting the stresses from the struts to the bottom chord, each strut is connected to the chord by inclined bolts *g*, *g*.

15. Method of Calculating the Stresses.—The method of calculating the stresses in the kingpost or kingrod truss depends on the manner in which the load is applied. The two usual conditions of loading will be discussed; they are: (1) when the load is applied to the truss at the center by a floorbeam; and (2) when the load is applied along the horizontal chord, the latter acting both as a stringer and as a chord. Since the kingrod truss is simply a kingpost truss inverted, the same methods apply to both, and only the kingpost truss will be discussed here.

16. Case I.—When the load is applied to the truss as a single load at the center joint by means of a floorbeam, as shown in outline in Fig. 5, the stresses in the members are all direct stresses, and can be found by the methods explained in *Stresses in Bridge Trusses*. In Fig. 5, h is the vertical distance between the center of the horizontal chord and the center of the bent rod at the center of the span, and l is the distance between the centers of the blocking, which should be directly under the intersections of the inclined members with the horizontal chord.

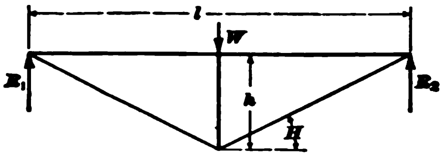


FIG. 5

17. Case II.—When the load is applied at every point along the truss, as shown in outline in Fig. 6, the stresses in the vertical and inclined members are direct stresses; but in the horizontal chord there is, in addition to the direct stress, a bending moment due to the loads. This moment is positive at sections from a and a' to sections near b, and is negative at b and for a short distance at each side of b. The formulas for the stresses in the members for this condition of loading are given here without their derivation, which requires the use of advanced mathematics.

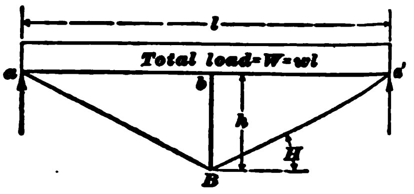


FIG. 6

In these formulas,

W = total load, in pounds, uniformly distributed on the truss;
l = span, in inches;
h = distance, in inches, from the center of the bent rod to the center of the top chord at the center of the span;
H = angle that the inclined member makes with the horizontal;
b' = net width, in inches, of the horizontal chord;
d' = net depth, in inches, of the horizontal chord.

In computing the net width and net depth, it is necessary to deduct from the gross width and depth, respectively, the amount of section removed for the drift bolt k, Fig. 2, usually 1 inch in width, and for the material removed at the top of the post b for its connection, usually about $\frac{1}{2}$ inch deep. Referring to Fig. 6, the stresses, in pounds, are as follows:

Total direct stress in b B:

$$S_1 = \frac{5W}{8}, \text{ compression} \qquad (1)$$

Total direct stress in a B and B a':

$$S_2 = \frac{5W}{16} \csc H, \text{ tension} \qquad (2)$$

Total direct stress in a b and b a':

$$S_3 = \frac{5Wl}{32h}, \text{ compression} \qquad (3)$$

Maximum bending moment on a a' (at b):

$$M = \frac{Wl}{32} \text{ inch-pounds} \qquad (4)$$

Maximum intensity of stress on a a' (at b):

$$s = \frac{Wl}{32hb'd'^2} \times [5d' + 6h] \text{ lb. per sq. in., compression} \qquad (5)$$

In designing the horizontal chord, it is necessary first to assume a cross-section, and then calculate the intensity of stress by means of formula **5**. If the actual stress is greater than the allowable stress s_c' as found by the formula in Art. **8**, or much less, the section is revised, and the intensity of stress is again computed by formula **5**. This process is repeated until the proper section is found.

EXAMPLE 1.—In the kingpost truss shown in outline in Fig. 6, let the span be 20 feet; the height, 5 feet, and the weight w per linear foot, 2,000 pounds. To find: (*a*) the total direct stress in bB; (*b*) the total direct stress in Ba'; (*c*) the total direct stress in ab; (*d*) the greatest bending moment on aa'.

SOLUTION.—Since the weight per linear foot is 2,000 lb. and the length is 20 ft., the total load W on the truss is 2,000 × 20 = 40,000 lb. Also, l = 20 × 12 = 240 in., and h = 5 × 12 = 60 in.

(*a*) The stress in bB is found by formula **1**. Substituting the proper value for W gives

$$S_1 = \frac{5 \times 40{,}000}{8} = 25{,}000 \text{ lb., compression. Ans.}$$

(*b*) The total direct stress in $B a'$ is found by formula 2. In the present case,

$$\csc H = \frac{\sqrt{\left(\frac{l}{2}\right)^2 + h^2}}{h} = \frac{\sqrt{\left(\frac{240}{2}\right)^2 + 60^2}}{60} = 2.236$$

Therefore,

$$S_2 = \frac{5 \times 40{,}000}{16} \times 2.236 = 27{,}950 \text{ lb., tension. Ans.}$$

(*c*) The total direct stress in $a b$ is found by formula 3:

$$S_3 = \frac{5 \times 40{,}000 \times 240}{32 \times 60} = 25{,}000 \text{ lb., compression. Ans.}$$

(*d*) The bending moment on $a a'$ is greatest at b, and its value is found by formula 4:

$$M = \frac{40{,}000 \times 240}{32} = 300{,}000 \text{ in.-lb. Ans.}$$

EXAMPLE 2.—If the member $a a'$ is composed of two timbers 8 inches wide and 16 inches deep, what is the greatest intensity of stress on the section?

SOLUTION.—To get the net width, it is necessary to deduct 1 in. from the width of each stick to allow for the decrease in section by the holes for the drift bolts; this leaves 7 in. for the net width of each, and 14 in. for the net width b' of the two. Deducting $\frac{1}{2}$ in. from the depth to allow for the decrease for the connection of the kingpost leaves the net depth $d' = 15\frac{1}{2}$ in. The maximum intensity of stress is found by formula 5:

$$s = \frac{40{,}000 \times 240}{32 \times 60 \times 14 \times 15.5^2} \times (5 \times 15.5 + 6 \times 60)$$
$$= 650 \text{ lb. per sq. in., compression. Ans.}$$

18. Example of Kingrod Bridge.—Fig. 7 shows a small highway bridge composed of one layer of floor plank 2 inches thick, the floor joists *f*, the floorbeam *g*, and the kingrod trusses. In the figure, (*a*) is the plan, (*b*) is the elevation, and (*c*) shows the connection of the floorbeam *g* to the kingrod *e* of the truss. The floorbeams are continued 4 feet 3 inches beyond the truss on each side, and inclined struts *i*, usually 4 in. × 6 in., are placed between the end of the floorbeam and the top joint of the truss to hold it in place. This is a very economical and serviceable bridge for localities to which it is adapted.

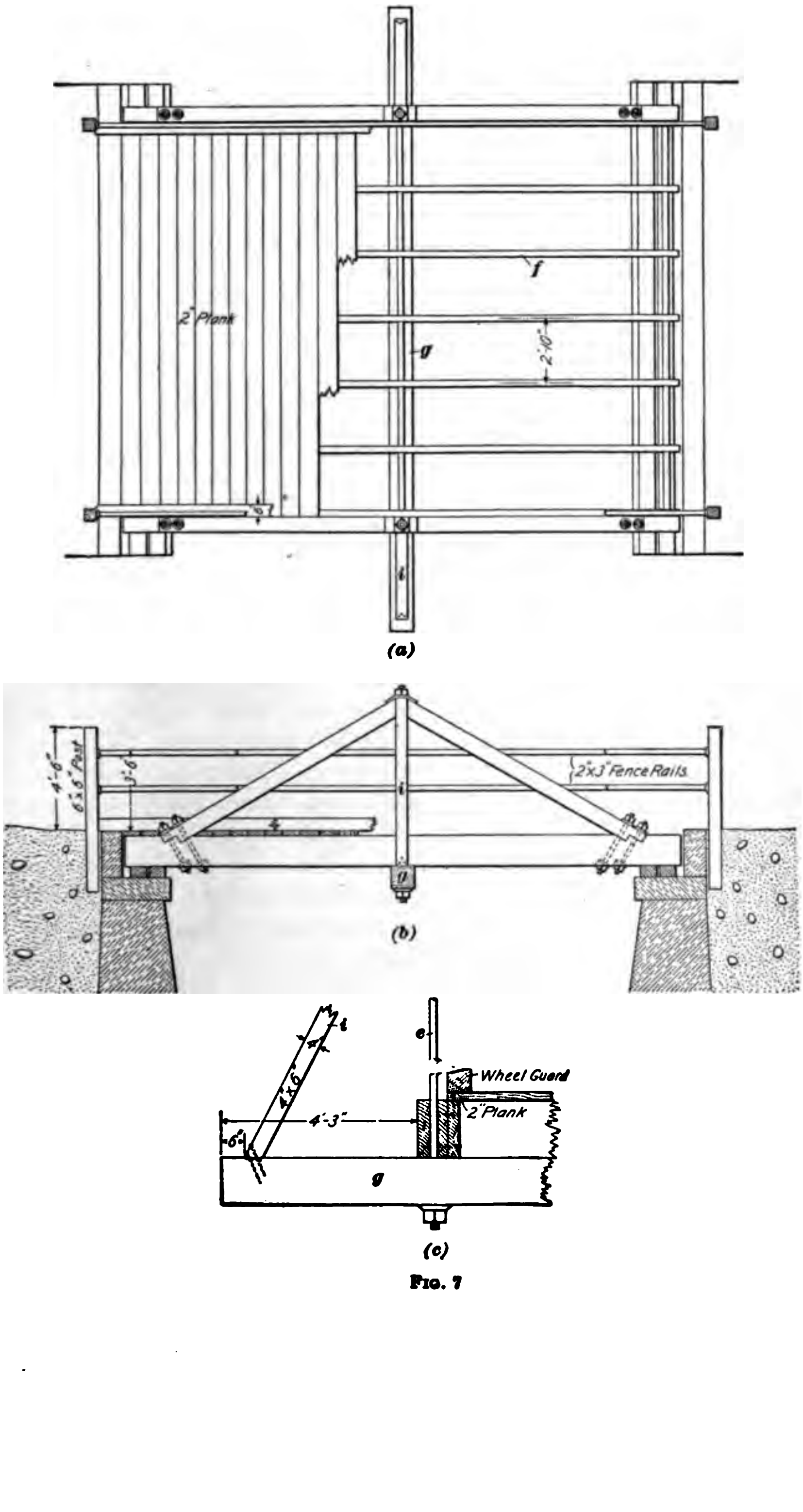

2" Plank
f
g
2'-10"
6"
i
(a)
4'-6"
6"x6" Post
3'-6"
2"x3" Fence Rails
4
i
g
(b)
i
4"x6"
e
Wheel Guard
2" Plank
4'-3"
6"
g
(c)

FIG. 7

EXAMPLES FOR PRACTICE

1. The panel load W of the truss shown in outline in Fig. 5 is 20,000 pounds. If the span is 20 feet, and the height is 4 feet, what is the direct stress in the top chord? Ans. 25,000 lb., compression

2. The load w per linear foot on the truss shown in outline in Fig. 6 is 2,400 pounds. If the span is 25 feet, and the height is 5 feet, what is the direct stress in the member $b\,B$? Ans. 37,500 lb., compression

3. What is the direct stress in the member $B\,a'$ of the truss referred to in example 2? Ans. 50,500 lb., tension

4. What is the direct stress in the member $b\,a'$ of the truss referred to in example 2? Ans. 46,900 lb., compression

5. What is the greatest bending moment on the top chord of the truss referred to in example 2? Ans. 562,500 in.-lb.

QUEENPOST AND QUEENROD TRUSSES

19. Description of the Queenpost Truss.—Fig. 8 represents a queenpost truss, which is similar to the kingpost truss, except that the former has three panels instead of two. The inclined struts e, e are placed in the center panel to provide for the shear in that panel; the top chord $a\,a$ is made continuous from end to end. The rod $c\,c$ is sometimes made in one piece, and sometimes in two pieces; in the latter case, a turnbuckle is inserted in the center panel. The castings f, f at the bottoms of the posts provide a bearing area for the posts and also for the ends of the inclined struts. The bottoms of the castings are grooved in order to afford the rods a better bearing.

20. Description of the Queenrod Truss.—When there is not sufficient room for the posts and rods to project below the floor, the queenrod truss shown in Fig. 9 is used. This truss is similar in every way to the kingrod truss, except that there are three panels in the former instead of two. Castings g are bolted to the upper ends of the inclined struts, to provide a bearing for the upper ends of the inclined web members e, e in the center panel. Instead of

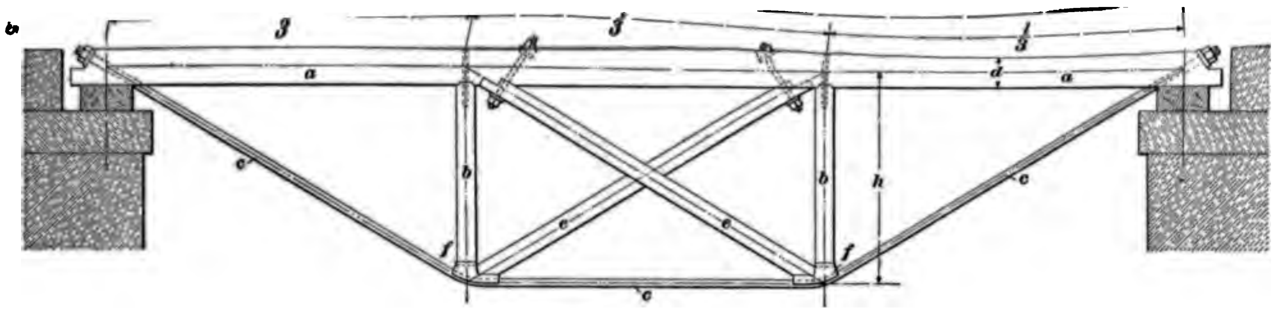


Fig. 8

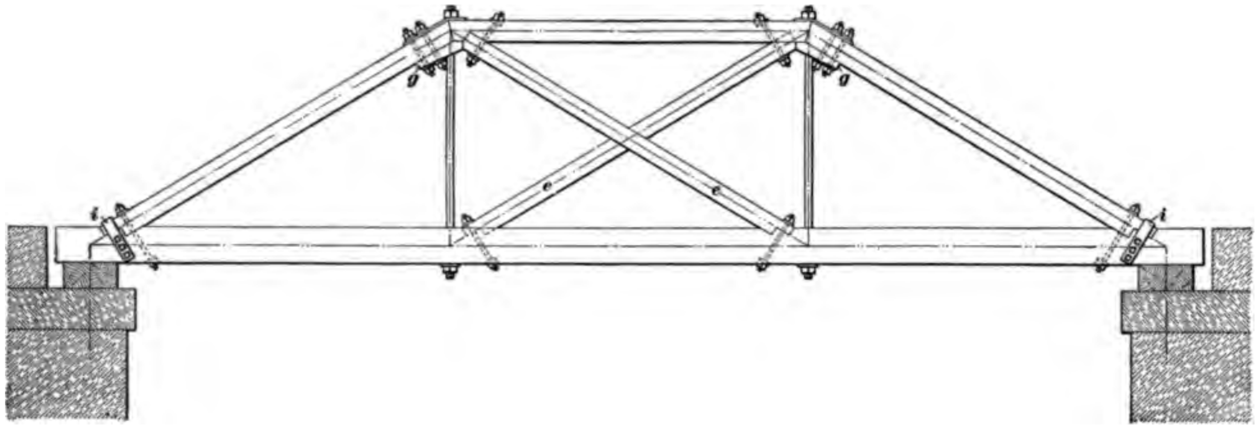


Fig. 9

stresses, and can be found by any of the methods explained in *Stresses in Bridge Trusses*.

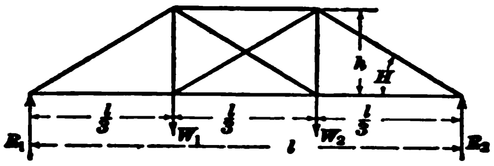


FIG. 12

24. Case II.—The condition of loading for Case II is shown in Fig. 13. The external forces are the reactions R_1 and R_2, and the uniform load per linear foot. In addition to the direct stress in the bottom chord $a\,b\,b'\,a'$, there is a bending moment on this member due to the loads applied between the panel points. On this account, the stresses in the members are somewhat different from those in Case I. The greatest stresses in the members are found approximately

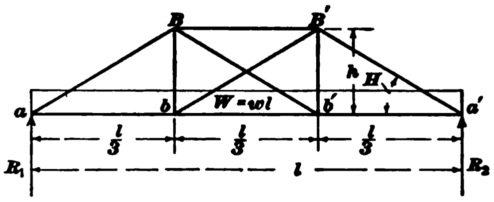


FIG. 13

by the formulas given below, which are derived by the use of advanced mathematics. In these formulas,

W = total load on the truss, in pounds;
l = span, in inches;
h = distance, in inches, between the centers of the chords;
H = angle between the inclined members and the horizontal;
b' = net width of the continuous chord $a\,b\,b'\,a'$;
d' = net depth of the continuous chord $a\,b\,b'\,a'$.

The formulas are as follows, the stresses being in pounds:

Total direct stress in $b\,B$ and $b'\,B'$:

$$S_1 = .367\,W, \text{ tension} \qquad (1)$$

Total direct stress in $a\,B$ and $B'\,a'$:

$$S_2 = .367\,W \csc H, \text{ compression} \qquad (2)$$

Total direct stress in b B′ and b′ B:

$$S_3 = \frac{W}{9} \csc H, \text{ compression} \qquad (3)$$

Total direct stress in B B′ and in a b b′ a′:

$$S_4 = \frac{.122\, W l}{h} \qquad (4)$$

The stress S_4 is compression in $B\,B'$ and tension in $a\,b\,b'\,a'$.

Maximum bending moment on a b b′ a′:

$$M = \frac{W l}{90} \text{ inch-pounds} \qquad (5)$$

Maximum intensity of stress on a b b′ a′:

$$s = \frac{W l}{90\, h\, b'\, d'^2}(11\, d' + 6\, h) \text{ lb. per sq. in., tension} \qquad (6)$$

25. Case III.—The conditions of loading that must be considered in this case are shown in Figs. 14 and 15. In Fig. 14, both panel points are loaded (W_1 and W_2), and when the loads are equal, as is usually the case, the stresses in the members are all direct stresses, and can all be found by the principles explained in *Stresses in Bridge Trusses*. This loading causes the greatest direct stresses in all the members, and controls the size of all the members except the chord $a\,b\,b'\,a'$.

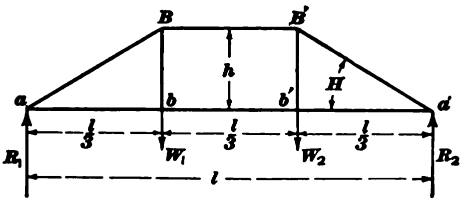


FIG. 14

The loading shown in Fig. 15 causes a bending moment in the member $a\,b\,b'\,a'$, so that in some cases this condition of loading controls the size of that member. In Fig. 15 there is but one panel point loaded; part of the load is supported by the truss, and the remainder by the member $a\,b\,b'\,a'$ acting as a beam. When

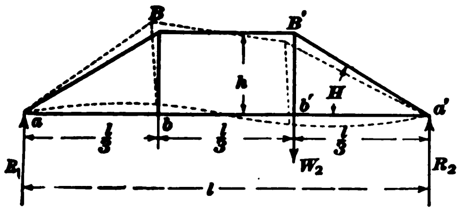


FIG. 15

the truss deflects, it assumes a position similar to that shown in Fig. 15 in dotted lines. It is impossible to tell with accuracy how much of the load is supported by the truss, and how much by the lower chord as a beam; it is customary to assume, however, that one-half the panel load is supported by the truss, and one-half by the member $a\,b\,b'\,a'$ as a beam. On this assumption, the stresses in $a\,b\,b'\,a'$ are as follows:

Total direct stress in $a\,b\,b'\,a'$:

$$S = \frac{W_1 l}{6h} \text{ pounds, tension} \qquad (1)$$

Bending moment in $a\,b\,b'\,a'$:

$$M = \frac{W_1 l}{18} \text{ inch-pounds} \qquad (2)$$

Maximum intensity of stress in $a\,b\,b'\,a'$:

$$s = \frac{W_1 l}{6\,h\,b'\,d'^2}(d' + 2h) \text{ lb. per sq. in., tension} \qquad (3)$$

26. Case IV.—The condition of loading that needs to be considered in this case is shown in Fig. 16. In addition to the direct stress in the bottom chord, there is a bending moment due to the loads that are applied to the truss between the panel points. This condition is similar to that shown in Fig. 13, and discussed under Case II. The stresses in the members of Fig. 16 can be found by means of the formulas given under Case II. When the truss is partly loaded, part of the load is carried to the abutments by the truss and part by the bottom chord acting as a beam in the same way as explained under Case III and illustrated in Fig. 15. In this case, however, the stresses in the members when the truss is fully loaded, as shown in Fig. 16, are greater than when the truss is partly loaded; hence, the latter condition of loading need not be considered.

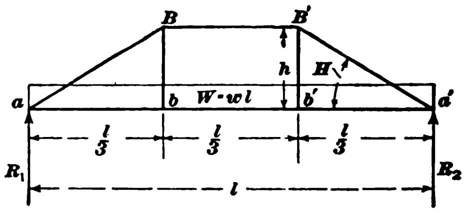


FIG. 16

27. The formulas given in the preceding articles give the stresses in the members of the various types of queenrod trusses. The same formulas can be used for the stresses in the members of the queenpost truss; the results will, in general, be numerically the same as, but opposite in sign to, those found for the corresponding members of the queenrod truss.

EXAMPLE 1.—In the queenrod truss shown in Fig. 13, let the weight w be 1,800 pounds per linear foot, the length 30 feet, and the height 6 feet. To find: (*a*) the total load on the truss; (*b*) the total direct stress in bB; (*c*) the total direct stress in aB; (*d*) the total direct stress in $a\,b\,b'\,a'$; (*e*) the bending moment on $a\,b\,b'\,a'$.

SOLUTION.—(*a*) Since the load w per linear foot is 1,800 lb., and the length is 30 ft., the total load W on the truss is $1,800 \times 30 = 54,000$ lb. Ans.

(*b*) The total direct stress in bB is found by formula **1** of Art. **24.** Here, $W = 54,000$ lb., and, therefore,

$$S_1 = .367 \times 54,000 = 19,800 \text{ lb., tension. Ans.}$$

(*c*) The total direct stress in aB is given by formula **2** of Art. **24.** Here, $W = 54,000$ lb., $\csc H = \frac{\sqrt{10^2 + 6^2}}{6} = 1.944$, and, therefore.

$$S_2 = .367 \times 54,000 \times 1.944 = 38,500 \text{ lb., tension. Ans.}$$

(*d*) The total direct stress in $a\,b\,b'\,a'$ is given by formula **4** of Art. **24.** Here, $w = 54,000$ lb., $l = (30 \times 12)$ in., and $h = (6 \times 12)$ in. Therefore,

$$S_4 = \frac{.122 \times 54,000 \times 30 \times 12}{6 \times 12} = 32,900 \text{ lb., tension. Ans.}$$

(*e*) The bending moment on $a\,b\,b'\,a'$ is given by formula **5** of Art. **24.** Here, $W = 54,000$ lb., and $l = (30 \times 12)$ in. Therefore,

$$M = \frac{54,000 \times 30 \times 12}{90} = 216,000 \text{ in.-lb. Ans.}$$

EXAMPLE 2.—If the member $a\,b\,b'\,a'$ in the preceding example is composed of two sticks each 7 in. $\times$ 14 in. in cross-section, what is the maximum intensity of stress in the member?

SOLUTION.—The maximum intensity of stress is given by formula **6** of Art. **24.** In the present case, $W = 54,000$ lb., $l = (30 \times 12)$ in., and $h = (6 \times 12)$ in. It will be assumed that the width of each stick is decreased 1 in. by connection bolts, making the net width b' of the two combined $2 \times (7 - 1) = 12$ in. It will be assumed that the depth is decreased $\frac{1}{2}$ in. at connections, leaving the net depth $d' = 14 - \frac{1}{2} = 13.5$ in. Substituting in the formula,

$$s = \frac{54,000 \times 30 \times 12}{90 \times 6 \times 12 \times 12 \times 13.5^2} \times (11 \times 13.5 + 6 \times 72)$$
$$= 796 \text{ lb. per sq. in., tension. Ans.}$$

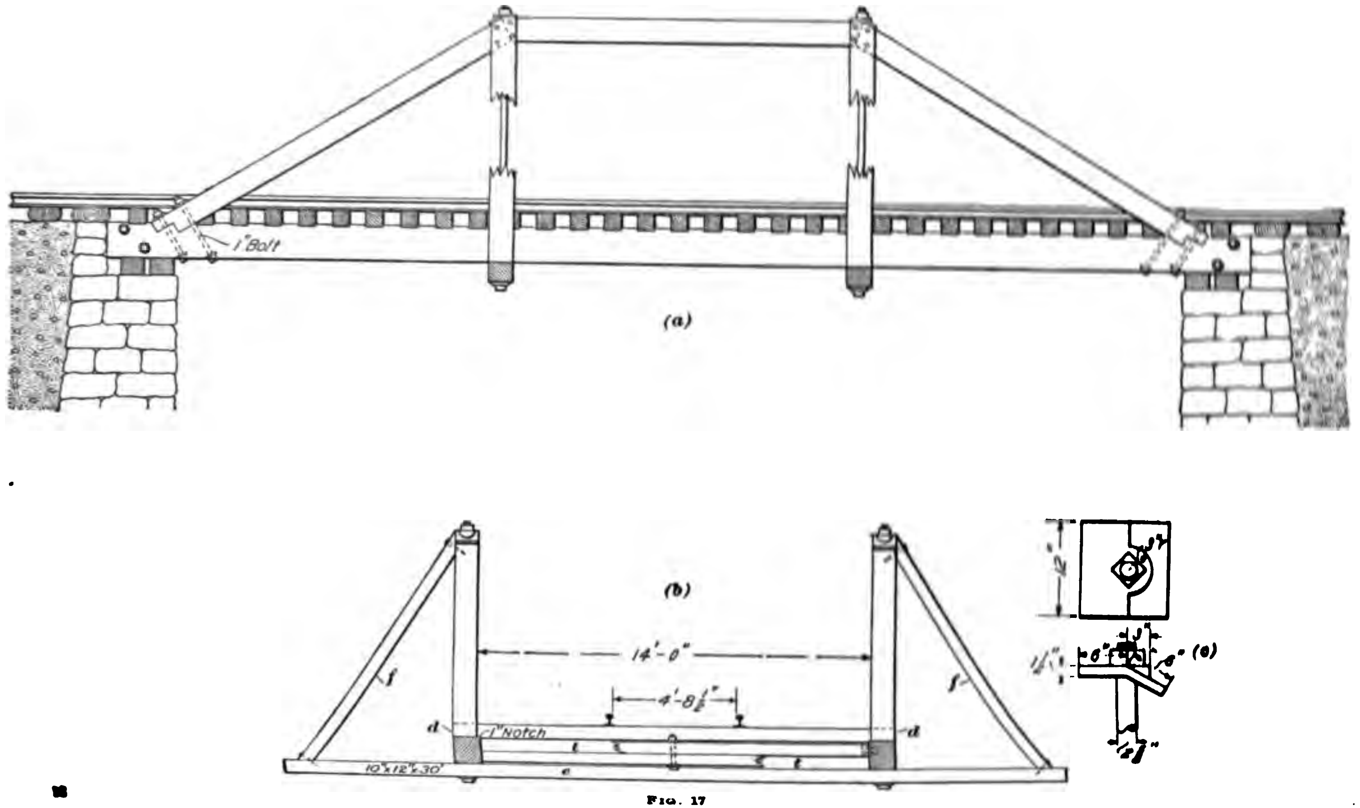


FIG. 17

28. Queenrod Bridge Without Diagonals.—Fig. 17 shows a small railroad bridge composed of two queenrod trusses with no diagonals in the center panel: (*a*) is the elevation of the bridge, (*b*) is the end view, and (*c*) is the detail of the casting that holds the ends of the members in place at the top of the truss. In this bridge, there are no stringers or floorbeams; the ends of the ties *d*, *d* rest directly on top of the bottom chord, and are notched to hold them in position. Bottom lateral braces *t*, *t* are inserted between the trusses. Bottom transverse struts *e*, usually about 10 in. × 12 in. × 30 ft., are placed under the trusses and extended a short distance beyond the trusses at each side of the bridge. Inclined struts *f*, *f*, usually about 6 in. × 8 in., connect the ends of the transverse struts with the top chord of the trusses in order to hold the trusses in position. This form of bridge is very serviceable for temporary purposes on a railroad operating light locomotives and cars.

EXAMPLES FOR PRACTICE

1. In the queenrod truss shown in Fig. 16, let w = 1,200 pounds per linear foot, l = 36 feet, and h = 6 feet. What is the total direct stress in the member $a B$? Ans. 35,450 lb., compression

2. What is the total direct stress in the member $b B$ of the truss referred to in example 1? Ans. 15,850 lb., tension

3. In the queenrod truss shown in Fig. 15, let W_1 = 10,000 pounds, l = 33 feet, and h = 6 feet. What is the total direct stress in $a b b' a'$? Ans. 9,170 lb., tension

4. What is the bending moment on the member $a b b' a'$ of the truss referred to in example 3? Ans. 220,000 in.-lb.

THE HOWE TRUSS

29. General Description.—The method of calculating the stresses in the members of the Howe truss was explained in *Stresses in Bridge Trusses*, Part 2. The general description of the truss, as built in wood, and the details of the connections are all that it is necessary to give here.

30. Fig. 18 is an elevation of a wooden Howe truss. The top and bottom chords and the inclined web members are composed of timbers; the vertical rods are composed of wrought iron or steel. The chords are usually composed of several timbers of nearly the same size; these timbers are commonly called **sticks.** The inclined web members and the verticals are connected to each other and to the chord by means of a cast-iron or cast-steel block, commonly called a **chord block**, as shown at *a b c* in Fig. 19 (*a*). This figure shows a bottom chord joint; the block *a b c* is set on top of the chord, and lugs *e, e*, cast on the block, fit into grooves of the same size in the top of the chord. The vertical rods *f, f* pass through the chord block and between the several sticks that compose the chord; they are held in place by means of the plates *g* and nuts *h* on the bottom of the rods. The upper surface of the chord block is inclined on each side of the vertical rod so as to give the diagonals Ed' and $d'C'$ a square bearing. In some cases, lugs *i, i* are cast on the inclined surfaces of the casting so that the inclined members will remain in place. The chord blocks are made just large enough to give the inclined members the proper width of bearing. They are not made solid, but with hollow spaces to decrease the weight. The outside shell is usually made from 1 inch to $1\frac{1}{2}$ inches thick. The lugs *e, e* are seldom made more than 1 inch deep or more than 2 inches wide. The plates *g* are made from 1 to 2 inches in thickness.

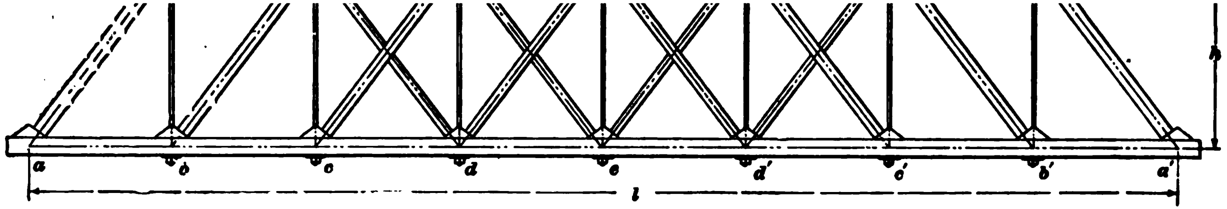


FIG. 18

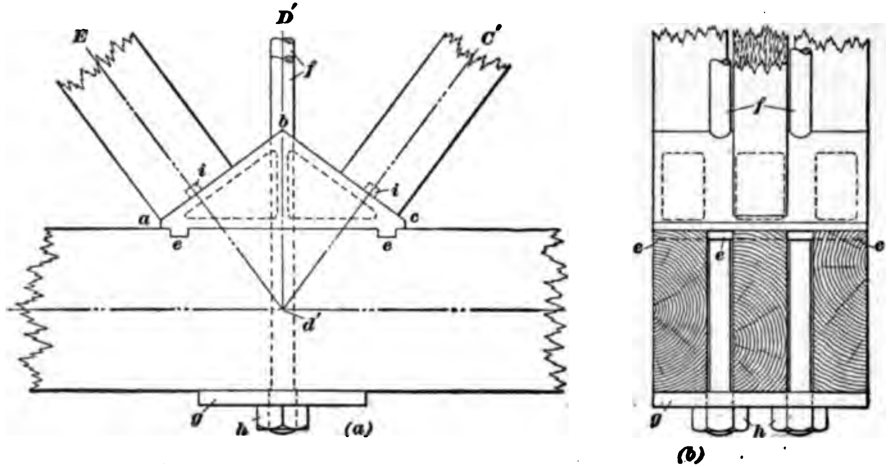
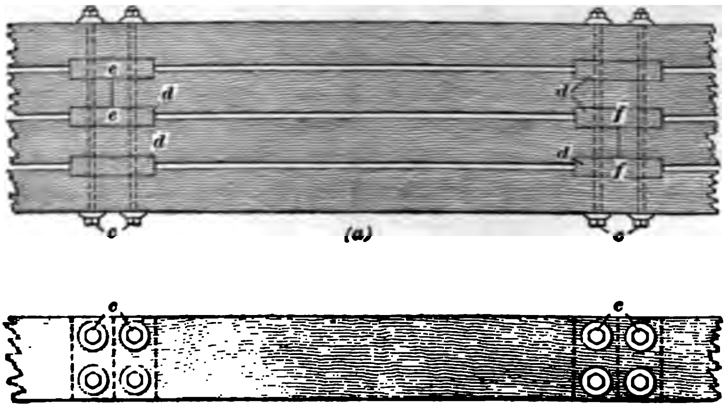


FIG. 19

31. Top Chords.—The top chord is usually composed of several sticks, as shown in Fig. 20, in which (*a*) is a top view and (*b*) a side elevation of a top chord composed of four sticks. The four sticks are bolted together by means of bolts *c*, *c*, so they will act as a single member, and are held at the proper distance apart (usually 1 or 2 inches for ventilation to prevent rot) by means of oak keys *d*, *d* about 3 inches thick and 12 inches long, set into the sides of the sticks. The sticks are usually made about as long as it is possible to get them, and in the top chord are spliced at the centers of the keys, as shown at *e*, *e* and *f*, *f*. The ends of the spliced portions



(a)

(b)

FIG. 20

are cut off perfectly square and brought into close contact throughout the width and depth of the stick. Not more than one stick should be spliced at one set of keys.

32. Bottom Chord.—The bottom chord is composed of several sticks bolted together, at intervals of about 6 feet, in the same way as the top chord. The difference between the two chords lies in the method of splicing. Since the stress in the bottom chord is tension, some means must be employed to transmit the stress from one portion of a spliced stick to the other portion. A common form of splice is shown in Fig. 21, in which (*a*) is a plan and (*b*) an elevation of a portion of a bottom chord, showing a splice at c, c. The ends of the portions to be spliced are cut off square in the same way as for the top chord; oak cleats d, d, about 6 or 7 feet long, are fitted into the sides of the sticks that are to

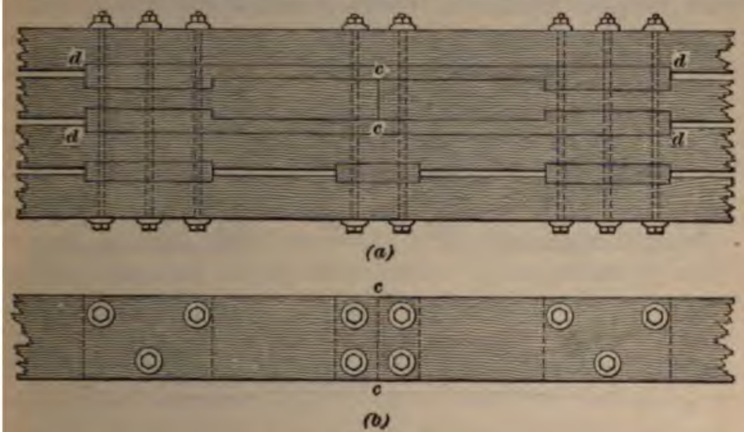


FIG. 21

be spliced, and the whole is bolted together as shown in the figure. The stress is transmitted to the cleats and by them is transmitted around the joint.

Another form of splice is shown in Fig. 22; it differs from the splice shown in Fig. 21 principally in that the cleats d, d are made of iron or steel instead of wood.

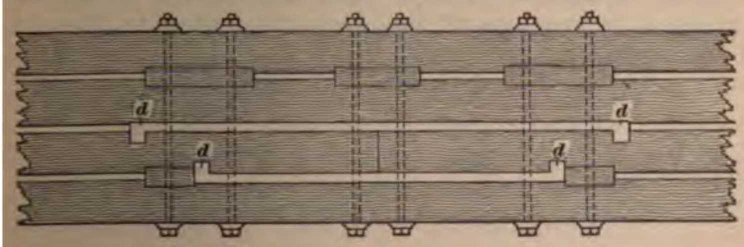


FIG. 22

Fig. 23 shows a combination of parts that is sometimes used to splice the bottom chords of Howe trusses. In this figure, (*a*) and (*b*) are castings with short cylindrical projections e, e. Holes are bored in the sides of the members to be spliced, and the castings placed in such a position that the projections *e*, *e* fit exactly into the bored holes. The castings are bolted in this position. The cleat *f* shown at (*d*) is then hooked over the projections *g*, *g* on the castings, and the wedge *w*, shown at (*e*), is inserted between

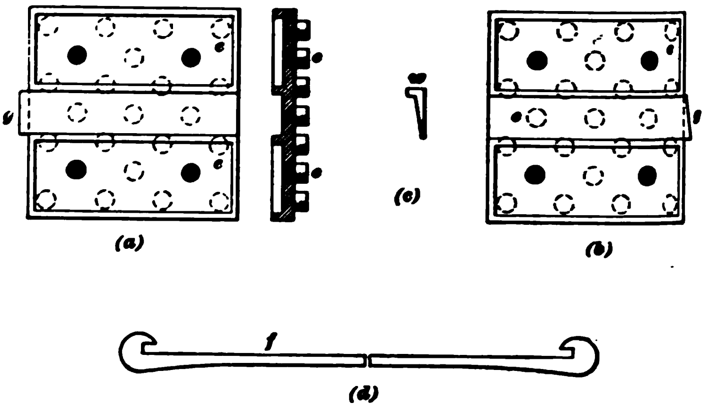


FIG. 23

the end of the right-hand projection and the inside of the cleat. The wedge is driven in so as to make the cleat fit tight at both ends. Two cleats are used in this case, one on each side, in the same way as two are used in Figs. 21 and 22.

33. Design of Splice.—The splice shown in Fig. 21 may fail in any one of five ways. Referring to Fig. 24,

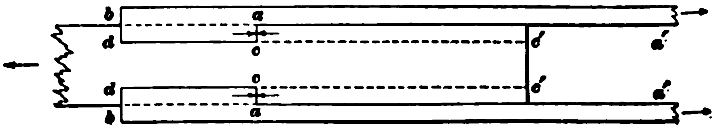


FIG. 24

they are as follows: (1) by the breaking of the cleats between *a* and *a'* in tension; (2) by the shearing of the cleats from *a* to *b* along the grain; (3) by the crushing of the ends of the fibers where the cleats and the shoulders on the stick come together at *a c* in bearing along the grain; (4) by the shearing of the end of the stick from c to c' along the grain; (5) by the breaking of the main stick where it is cut into the most; that is, between c and d, in direct tension. In a well-designed joint, the resistance to failure from each of these causes should be as nearly as possible the same, and, in designing, the dimensions are so proportioned that the working stresses given in Art. 8 are not exceeded.

34. Case I.—In the first case mentioned in the preceding article, the amount of stress that can be transmitted by the cleats in tension is the product of the net area of the cleats and the working stress in tension. In finding the net area of a cleat, it is necessary to deduct from the gross section the area of the bolt holes. As a rule, it is sufficiently accurate to take the net depth of the cleat as 2 inches less than the gross depth. For example, if each cleat is made of white oak 16 inches deep and $1\frac{1}{2}$ inches thick, the net area of two cleats [since the net depth is 2 inches less than the gross depth, Fig. 21 (b)] is $2 \times (16 - 2) \times 1\frac{1}{2} = 42$ square inches. The working stress s_t in tension for white oak is given in Art. 8 as 900 pounds per square inch; then, the amount of stress that can be transmitted by the two cleats in tension is $42 \times 900 = 37{,}800$ pounds.

35. Case II.—In the second case, the amount of stress that can be transmitted by the two cleats in shearing along the grain is the product of the area in shear and the allowable shearing stress. For example, if each cleat is made of white oak 16 inches deep, and the length $a\,b$ (Fig. 24) subjected to shear is 12 inches, the area subjected to shearing is $2 \times 12 \times 16 = 384$ square inches. Then, since the working stress s_s in shear along the grain is, for white oak, 100 pounds per square inch (Art. 8), the amount of stress that can be transmitted is $384 \times 100 = 38{,}400$ pounds.

36. Case III.—In the third case, the amount of stress that can be transmitted by the bearing area between the cleats and the shoulders on the member is the product of the bearing area and the working stress in bearing along the grain. For example, if the shoulders $a\,c$ are $1\frac{1}{2}$ inches wide, the depth of the member is 16 inches, and the cleats are of white oak and the member of white pine, the area of bearing is $2 \times 16 \times 1\frac{1}{2} = 48$ square inches. The allowable intensity s_r of bearing along the grain of white oak is given in Art. 8 as 1,250 pounds, and on white pine as 800 pounds per square inch. Since the latter is the smaller, it must be used; the stress that can be transmitted is, therefore, $48 \times 800 = 38,400$ pounds.

37. Case IV.—In the fourth case, the amount of stress that can be transmitted by the section cc', Fig. 24, of the member in shearing along the grain is the product of the area in shear and the allowable shearing stress. For example, if the member is white pine, the depth of the member 16 inches, and the distance cc' 24 inches, the area in shear is $2 \times 16 \times 24 = 768$ square inches. Then, since the working stress s_s in shear is, for white pine, 50 pounds per square inch, the amount of stress that can be transmitted is $768 \times 50 = 38,400$ pounds.

38. Case V.—In the fifth case, the amount of stress that can be transmitted by the member where it is cut into to admit the cleats is the product of the net area of the decreased portion of the member and the working stress in tension. In finding the net area of the stick, it is customary to deduct the area of the bolt holes from the gross section of the member. As a rule, the bolts are staggered at this connection, so that it is sufficient to take the net depth of the stick as 1 inch less than the gross depth. For example, if the stick is white pine and has a gross width of 7 inches, the gross depth is 16 inches and the notches are $1\frac{1}{2}$ inches deep, the net width is $7 - 2 \times 1\frac{1}{2} = 4$ inches, and the net depth (decreased by one bolt, Fig. 21) is $16 - 1 = 15$ inches; then, the net area is $4 \times 15 = 60$ square inches. Since the working stress s_t in tension is 650 pounds per square inch (Art. 8), the amount of stress that can be transmitted is $60 \times 650 = 39,000$ pounds.

39. The dimensions of the splice that has been considered in the examples in the preceding articles are shown in

Fig. 25. This form of splice is frequently used. The bolts that hold together the various parts of the splice should not be considered as transmitting any part of the stress; they

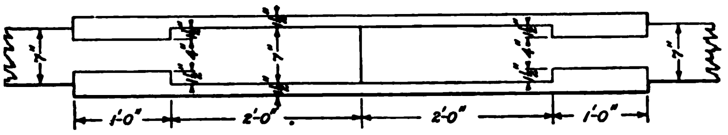


FIG. 25

erve simply to hold the parts so that they will work ogether.

40. Lateral System.—The lateral trusses are formed ı the same way as the vertical trusses. The transverse ıembers are composed of rods and the inclined members of imber. They are connected to the sides of the chords by hord blocks, or castings, in the same way as the web mem-ers of the vertical trusses are connected to the chords.

EXAMPLES FOR PRACTICE

1. If the cleats shown in Fig. 24 are of white oak, 14 inches deep nd $1\frac{1}{4}$ inches thick, how much stress can they transmit without xceeding the working stress in tension? Ans. 27,000 lb.

2. If, in example 1, the length of each cleat subjected to shear is 2 inches, how much stress can the cleats transmit without exceeding ıe working stress in shearing along the grain? Ans. 33,600 lb.

3. If the main member is of white pine and the width of the houlder where the cleat bears on the member is $1\frac{1}{4}$ inches, how much :ress can be transmitted without exceeding the working stress in earing along the grain? Ans. 28,000 lb.

4. If the length of the member from the shoulder to the end is) inches, how much stress can be transmitted by the member without xceeding the working stress in shearing along the grain? Ans. 28,000 lb.

5. If the thickness of the member where it is cut into for the ends f the cleats is $3\frac{1}{2}$ inches, how much stress can be transmitted without xceeding the working stress in tension? Ans. 29,600 lb.

TOWNE LATTICE TRUSS

41. General Description.—Fig. 26 shows a type of truss, called the **Towne lattice truss,** that is used to some extent at the present time: (*a*) is the elevation of the end portion of a truss, and (*b*) is a cross-section on *CC*. In this truss, all the members are wood, the only metal being the bolts used to hold the members together. The chords consist of the horizontal members *cc*, *dd*, *ee*, and *ff*. The two portions *cc* and *dd* form the top chord; the two portions *ee* and *ff* form the bottom chord. Each portion of the chords is composed of several sticks, as shown in cross-section in Fig. 26 (*b*.) The web consists of a large number of flat planks, the ends of which are connected to each other and to the chords. The horizontal member *gg* consists of two pieces and serves the purpose of stiffening the web members near the center of the length. The trusses are stiffened at the ends by means of vertical timbers *hh* and *ii* bolted to the sides of the web members. The ends of the trusses rest on blocking *jj* that is supported on the bridge seats.

42. Floorbeams.—When these trusses are used, the floorbeams are connected below the bottom chord as shown at *kk* in Fig. 26. The bolts *l*, *l* pass through the lower portion of the bottom chord and through the ends of the floorbeams. The latter are usually spaced from 2 to 3 feet apart in this type of bridge.

43. Connection of Web Members to Chords.—The method of connecting the web members to the chords is shown in detail in Fig. 27, in which *aa* and *bb* are web members and *cc* is the chord member. At *d*, the intersection of the center lines of the members, an iron bolt about 1 inch in diameter is driven through a bored hole, and tightened up. Oak pins *e*, *e*, commonly called **treenails,** having the

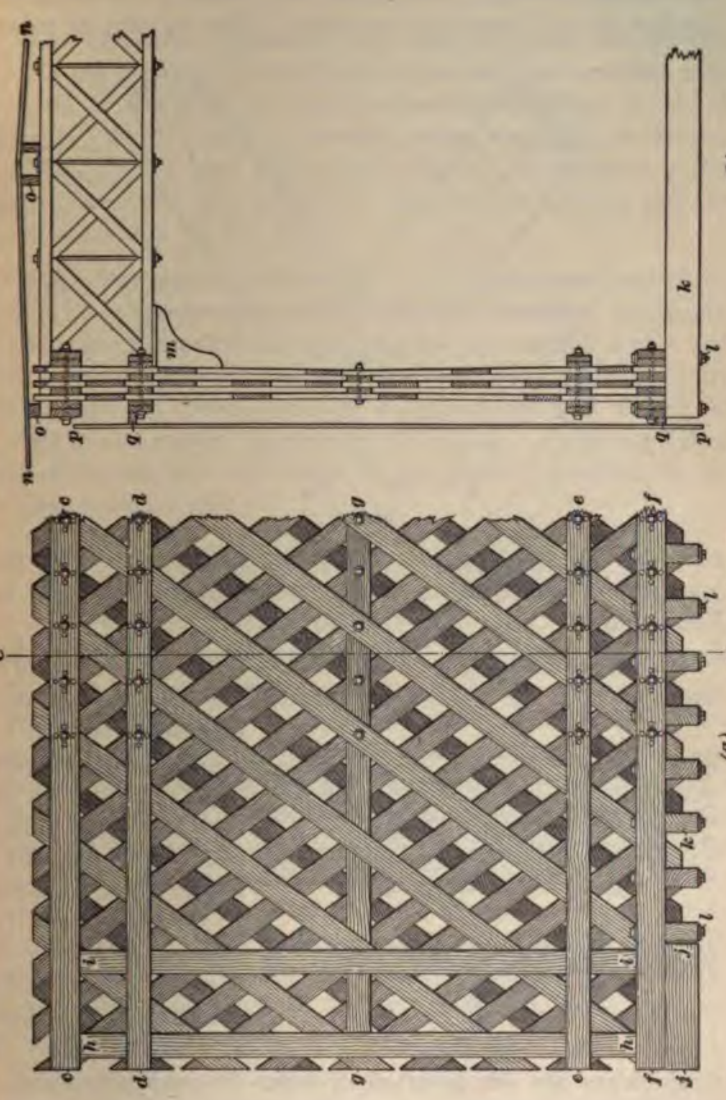


FIG. 26

same diameter and nearly the same length as the bolt, are also driven through bored holes, and serve to transmit the stresses to and from the members. The amount of stress

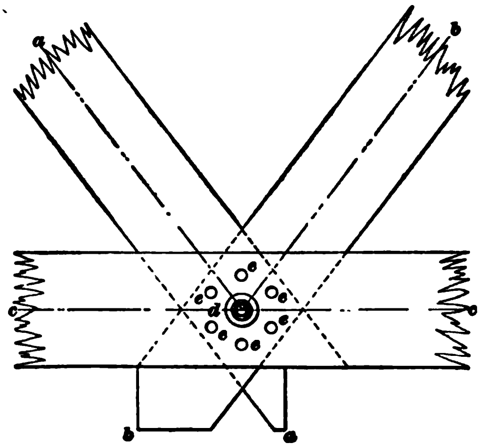


FIG. 27

that can be transmitted to any of the members by the joint shown in Fig. 27 is usually assumed in practice to be 7,500 pounds.

44. Splices in Chord Members.—The sticks that form the top chord are joined by cutting the ends square and bringing them into good contact in much the same way as in the Howe truss. The several sticks that form the bottom chord are usually spliced as shown in Fig. 28. The ends are brought together, and wrought-iron or steel pieces *d*, *d* are inserted in holes cut in the members; long **U**-shaped bolts *e*, *e* are placed over the ends of the pieces *d*, *d*. The nuts *f*, *f* serve to tighten up the bolts

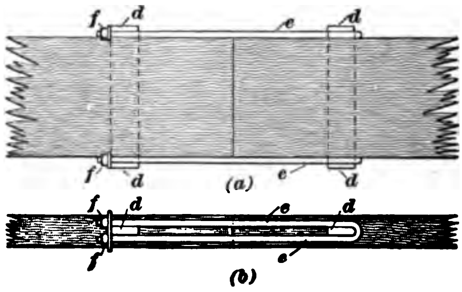


FIG. 28

so that they will bear firmly against the top and bottom of the pieces *d*, *d*.

45. Lateral System.—The lateral trusses are usually made the same as for the Howe truss, and the members connected to the sides of the members by castings or chord blocks. In addition, transverse frames are put in, as shown in Fig. 26 (*b*), and curved knee braces or brackets *m* are placed between the bottom of the transverse frame and the web members of the truss.

46. Protection of Truss.—To prevent deterioration on account of the weather, it is customary to shelter this truss, and in some cases Howe trusses also, by building a roof and sides, as shown in Fig. 26 (*b*). The roof *n n* is usually composed of 1-inch boards supported by and nailed to joists *o*, *o* on top of the truss. The sides *p*, *p* also consist of 1-inch boards that are nailed to the pieces *q*, *q* bolted to the sides of the trusses.

COMBINATION TRUSSES

47. General Description.—Although, strictly speaking, a **combination** truss is any truss in which some members are of wood and some of metal, the name is generally restricted to pin-connected trusses in which the compression members are wood and the tension members are steel or iron. The Pratt and the Baltimore are the two forms of truss that are most used for this construction. In all combination trusses, the details of the connections of the wooden members are arranged in such a way that the members can be removed and replaced without the necessity of taking down the truss. This arrangement is necessary because the wooden members decay and must be renewed from time to time, while the steel, if properly taken care of, will last almost indefinitely.

48. Bottom Chord Joint.—Fig. 29 shows a bottom chord joint of a combination truss: (*a*) is the elevation and (*b*) the side view of the joint. The only difference between this joint and that in a steel pin-connected truss is in the connection of the vertical member to the pin. This is accom-

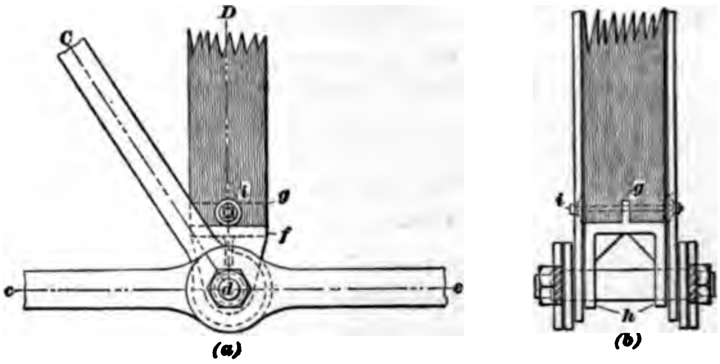


FIG. 29

plished by attaching to the end of the member a casting *f*, having a projection *g* that fits into a groove in the end of the

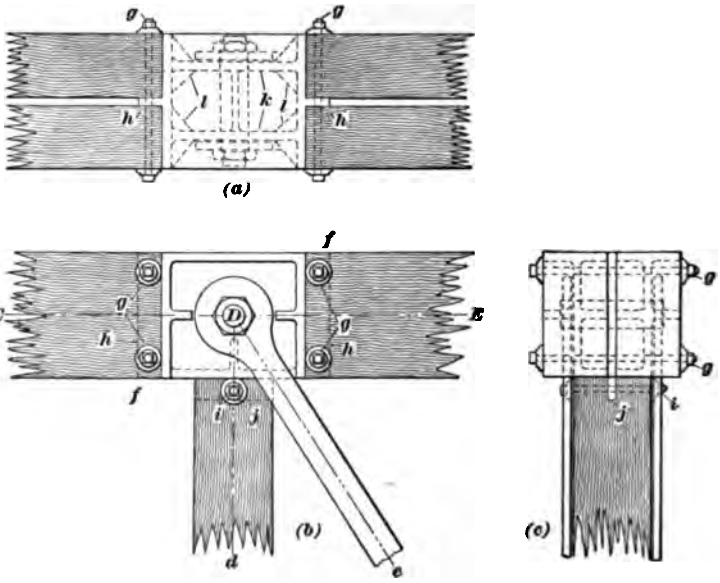


FIG. 30

member. The casting has two sides or webs *h* that bear on the pin. The bolt *i* is simply for the purpose of holding the member in place on the casting, and does not transmit any stress. The thickness of metal in the casting, which should always be a steel casting, is usually not less than 1 inch.

49. Top Chord Joint.—Fig. 30 shows a top chord joint of a combination truss: (*a*) is the top view, (*b*) is the elevation, and (*c*) is the side view of the joint. The ends of the top chord members abut on the sides of a casting *f f*, and are

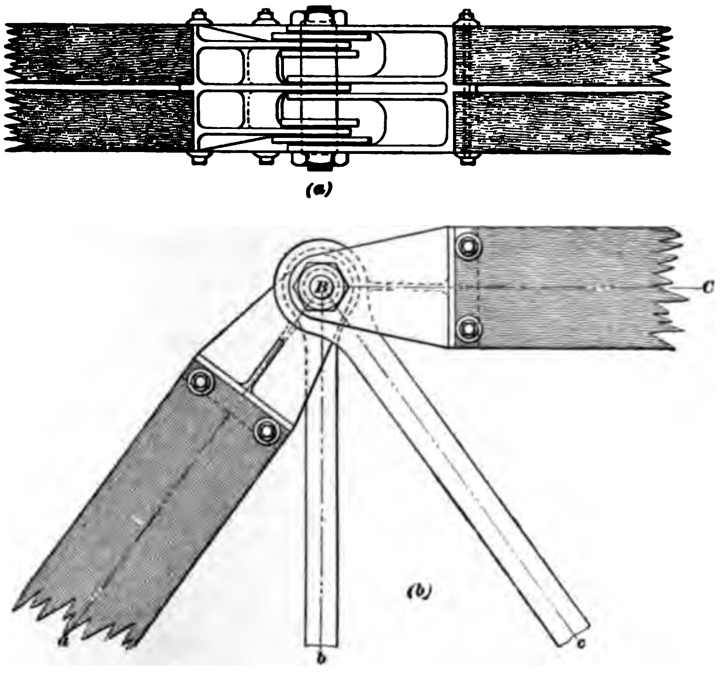


Fig. 31

held in place on the casting by means of bolts *g*, *g* that pass through the ends of the members and through projections *h*, *h* on the sides of the casting. The vertical abuts on the under side of the casting, and is held in place by a bolt *i* that passes through a projection *j* on the under side. The sides of the casting that are in contact with the ends of the chord members are connected by webs or diaphragms *k*, *k* about opposite the centers of the sticks. These webs provide also a bearing for the pin that passes through the casting and holds the ends of the eyebars in place. The casting is further stiffened by inclined webs *l*, *l* between the webs *k*, *k* and the bearing surfaces.

50. Hip Joint.—The detail at the hip joint is somewhat different from the other top chord joints. Fig. 31 shows a hip joint: (*a*) is the top view, and (*b*) the elevation of the joint. The end post *a B* and the top chord *B C* are bolted to separate castings; these are furnished with webs or diaphragms for bearing on the pin arranged in such a way that the hip vertical *b B* and the end diagonal *B c* can also connect to the pin. In the figure, the hip vertical is placed inside of both castings, and the end diagonal is placed between the two castings.

51. Design of Pins.—The pins in a combination truss are designed in the same general manner as those in a steel pin-connected truss. The method of procedure is fully described in *Design of a Highway Truss Bridge*, Part 2.

ROOF TRUSSES

INTRODUCTION

Conditions Governing Use of Roof Trusses. ıses are frequently employed to support the roofs of lings in cases where a large area of floor clear of inter- iate walls and columns is desired, and when so used are d **roof trusses.** Roof trusses are usually set at right es to the length of the building, so as to make the span ıort as possible; and their ends either rest on top of the walls or are supported by columns embedded therein.

Types of Roof Trusses.—The same general types usses are used for roofs as for bridges, except that the nations of the chords of roof trusses are made to conform the slope of the roof and the required underneath clear- . This gives rise to special types, some of which are trated in Figs. 1 to 15. The simplest type of roof truss ıown in Fig. 1; it consists simply of the two inclined

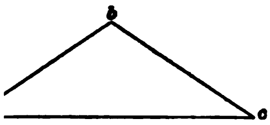


FIG. 1

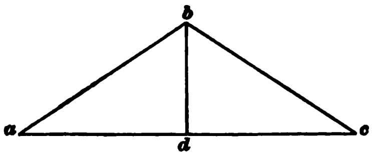


FIG. 2

s ab and bc, the lower joints of which are connected by ıorizontal chord or **bottom tie** ac. This type of truss be used for spans up to 20 feet. In Fig. 2, the long ;ontal chord or tie ac is supported at the center by the cal tie bd. This type of truss may be used for spans up to 30 feet. For spans from 30 to 40 feet, the form shown in Fig. 3 may be used. In this truss, the inclined struts *ab* and *bc* are supported at their center points by the inclined struts *de* and *df*; the lower ends of these struts are supported by the vertical tie *bd*. For spans up to 40 feet, the type shown in Fig. 4 is sometimes used. When built of timber, with the tie *ac* continuous, or in one piece

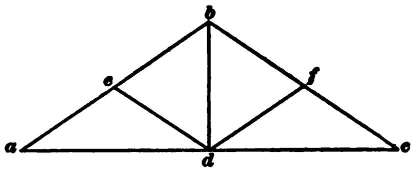


FIG. 3

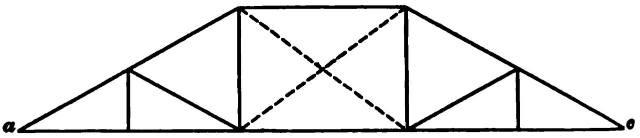


FIG. 4

from *a* to *c*, the diagonals in the center panel are sometimes omitted. The types shown in Figs. 5, 6, 7, 8, and 9 are used

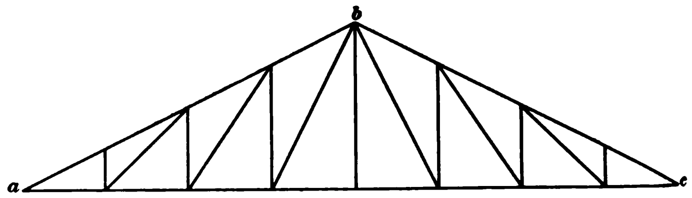


FIG. 5

for all spans, the particular type chosen for any special case depending on local conditions, and, to a great extent, on the

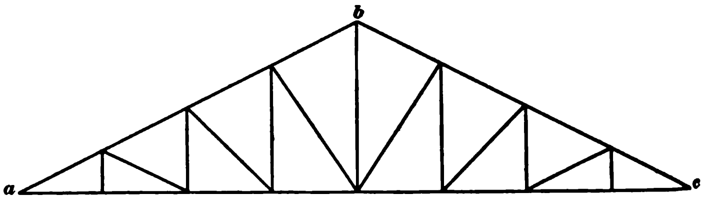


FIG. 6

judgment of the designer. In Figs. 5, 6, and 7, the slopes *ab* and *bc* are each divided into a number of equal spaces. In Fig. 7, the web members *dd* and *ee* are at right angles to the slope. In Figs. 8 and 9, the construction shown above the trusses in dotted lines is for the purpose of giving the surface of the roof an even slope.

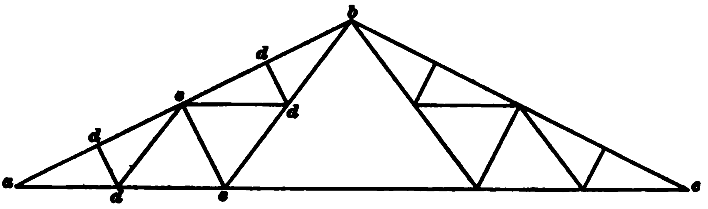


FIG. 7

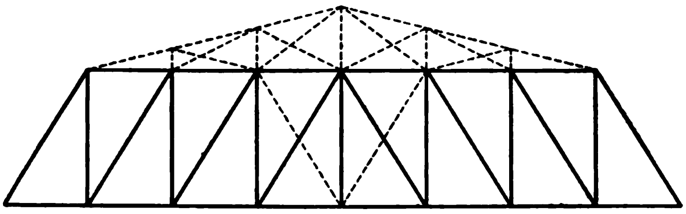


FIG. 8

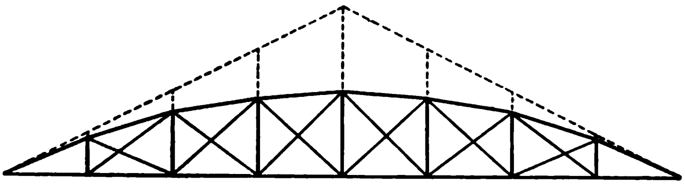


FIG. 9

3. All the trusses considered in the preceding article have horizontal lower chords. It is often desired to have greater headroom, or clearance, beneath the trusses at the center of the building than at the sides. In such a case, the walls or columns that support the ends of the roof trusses are frequently not carried as high as the required height at the center of the building, and one of the types of roof trusses shown in Figs. 10 to 13 is used.

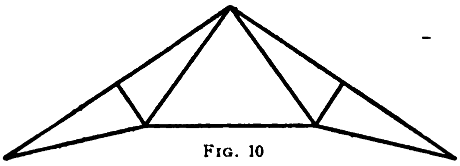


FIG. 10

4. For supporting the roofs of station platforms, grand stands, etc., the arrangement shown in Fig. 14 is frequently employed. It consists of a truss resting on two columns; the ends of the truss overhang the supports, forming cantilevers. In shop and mill buildings, in which the roof trusses are supported on columns, the lower chord of the truss is frequently connected to the columns by inclined struts, as shown at *b* in Fig. 15; these struts are usually projections of one of the web members. The projection at the top of Fig. 15 is commonly called a **monitor,** and serves the purpose of providing vertical skylights at *c, c*. The members of the monitor do not form part of the truss, but are simply for the purpose of transmitting to the truss any loads that may come on the monitor.

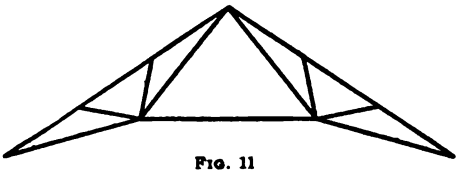


Fig. 11

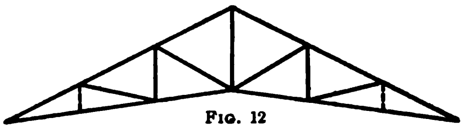


Fig. 12

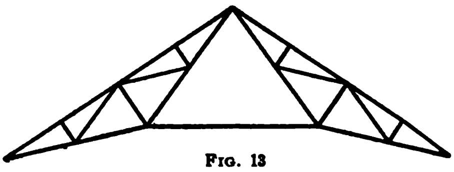


Fig. 13

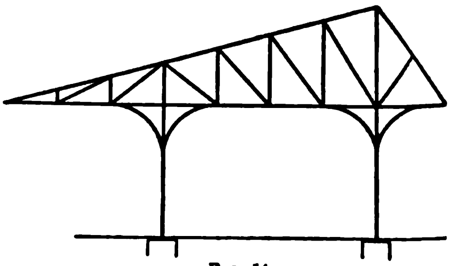


Fig. 14

5. Distance Between Trusses. The distance between trusses depends to some extent on the architectural design; that is, the location of windows, doors, etc. It is customary, in designing a building, to arrange it in a number of sections, or **bays,** and to have the same openings for windows in each bay. Between the bays, the wall is carried up solid from the foundation. The trusses usually rest on this solid on columns embedded in it. It is bad practice to truss rest on a wall directly over the opening for a a window. There is usually a roof truss at the junc- each two bays each end wall. been found in e that it is well e the distance n trusses about urth the span. are seldom less than 10 d never more bout 50 feet to center.

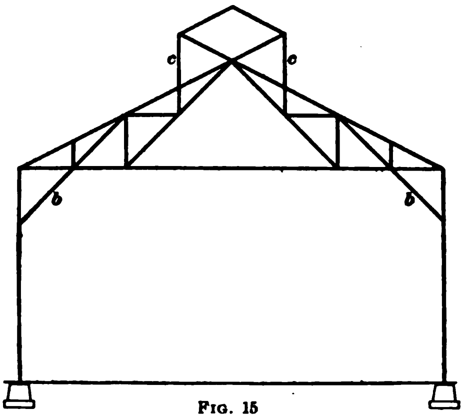


Fig. 15

Span and -The horizontal distance l, Fig. 16, between the sup- f a roof truss is called the **span**. The vertical dis- from the top of the truss to the level of the supports d the **rise**. The vertical distance h from the inter- s of the inclined top members to the center of the lower chord at the center of the span is called the **depth** of the truss. In some cases, h is made equal to r, in others h is made less than r.

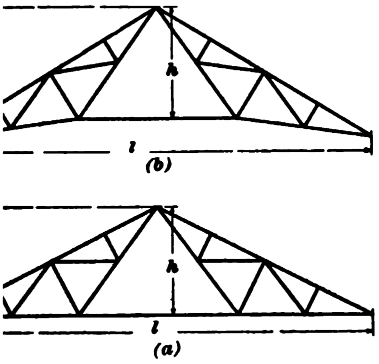


Fig. 16

7. Rafters, Purlins, and Chords. The inclined upper members of a roof truss, as ab in Figs. , 6, 7, are called **main rafters**, or simply **rafters**. wer chord or member ac is called simply the **chord**. trusses are connected to each other at the joints of the by beams called **purlins**, as shown in cross-section in Fig. 17 (*a*). When the distance between trusses is small, channels or I beams are used for purlins; when the distance between trusses is greater than 20 or 25 feet, riveted girders or trusses are used. The ends of the purlins sometimes rest on top of the rafters, as shown in Fig. 17 (*a*), and are sometimes connected to the web members, as shown in Fig. 17 (*b*).

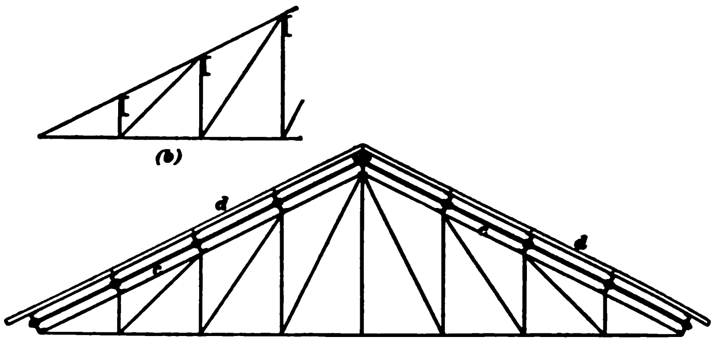


FIG. 17

As the surface of the roof is not horizontal, there is a tendency for the purlins to sag or deflect sidewise. To counteract this tendency, the purlins are connected at one or more points intermediate between the trusses by means of tie-rods *c*, Fig. 17 (*a*), that run from the purlin at the top, or **ridge**, down both sides of the roof.

8. On top of the purlins, and at right angles to them, are smaller beams *d*, Fig. 17 (*a*), close together. They are called **common rafters**, and are usually spaced about 2 feet apart. They support the roofing material and covering, and transmit any load on the roof to the purlins. The purlins transmit the load to the trusses at the joints of the main rafter.

9. Panel Length.—The **panel length** of a roof truss is the distance between two successive joints of the rafter, measured along the slope. It may be found by the formula

$$p = \frac{\sqrt{l^2 + 4r^2}}{n}$$

in which p is the panel length and n is the number of panels.

10. Slope of Roof.—The slope of the roof is frequently poken of in terms of the ratio between the rise and the span. This ratio is called the **pitch**. Thus, a pitch of $\frac{1}{4}$ means that he rise of the roof is $\frac{1}{4}$ of the span. The slope is also spoken of as the ratio of the vertical distance, called the **rise**, to the orresponding horizontal distance, called the **run**; thus, 1 of ise to 2 of run means that for every 2 feet measured horiontally, the roof rises 1 foot vertically. This ratio is usually poken of as 1 in 2, 1 in 3, etc. This form will be used in he following articles.

11. Roof Covering.—The materials commonly used or roof coverings are shingles, slate, tile, copper, tin, corrugated iron, felt, asphalt, tar, and gravel. Thin slabs of reinorced concrete have also been used in a number of cases. With all of these materials, except the reinforced-concrete lab, it is customary to put wooden sheathing about 1 inch hick directly on top of the common rafters, so as to afford he roofing material a flat surface on which to rest. Shingles and tile may be used on all slopes greater than 1 in 2, and late and corrugated iron on all slopes greater than 1 in 3. Copper and tin with the joints well soldered, and also reinorced concrete, may be used on all slopes. Felt, with asphalt or tar and gravel, is used on flat roofs having slopes ess than 1 in 4. If used on greater slopes, the tar or asphalt will run down when it gets warm, and leave the upper part of the roof exposed. The kind of roof covering adopted depends mainly on the type of building and on the amount of money available.

LOADS AND STRESSES

12. Loads.—The loads that must be supported by roof trusses are the dead load, which consists of the weight of roof covering, rafters, purlins, and trusses; the live load, which consists of the heaviest snowfall; and the wind load, which is due to the pressure of the wind. In addition, if ceilings or balconies are attached to the chords of the trusses, the latter must be designed to support them.

13. Dead Load.—The dead load depends on the kind of roof covering used, the distance between rafters and purlins, and the distance between trusses and supports. The approximate weights of the usual roof coverings and ceiling are as follows:

Materials	Pounds per Square Foot
Shingles	2
Slate	8
Corrugated tile	9
Tile on 3-inch fireproof blocks	35
Copper and tin	$1\frac{1}{4}$
Corrugated iron	2
Felt with tar or asphalt and gravel	9
Wooden sheathing 1 inch thick	4
Lath-and-plaster ceiling	10
Glass skylight, including frames	8
Concrete slab, including reinforcement (per inch of thickness)	12

The weight of the common rafters, purlins, and trusses should first be assumed, and the weights revised after the members have been designed. If balconies are attached to the trusses, their weight must be calculated.

14. Snow Load.—The weight of snow that may fall and remain on a roof in winter depends on the slope of the roof ıd on the climate. It is customary to assume a weight of ' pounds per square foot in the northern part of the United ates, and 10 pounds per square foot in the southern part, ı roofs the slope of which is not greater than 1 in 2. These ads are gradually decreased for steeper roofs, up to a slope 2 in 1, for which the snow load is neglected, it being sumed that the snow will slide off by its own weight.

15. Wind Pressure.—In considering the pressure of e wind on a roof, it is customary to resolve it into two ›mponents, one parallel with the surface of the roof, and e other normal to that surface. In calculating the stresses ıe to the wind pressure, the former component is neglected. he intensity of the normal component varies with the slope, ꞉ing greater for steep than for flat roofs. The normal tensities of wind pressure per square foot on roofs of different slopes are usually taken as follows:

Slope of Roof	Normal Intensity, Pounds per Square Foot
Flat roof	0
1 in 5 ($\frac{1}{10}$ pitch)	10
1 in $2\frac{1}{2}$ ($\frac{1}{5}$ pitch)	20
1 in 2 ($\frac{1}{4}$ pitch)	25
1 in $1\frac{1}{2}$ ($\frac{1}{3}$ pitch)	30
1 in 1 ($\frac{1}{2}$ pitch)	36
1 in $\frac{3}{4}$, and steeper	40

Since wind can blow in but one direction at any one time, is customary to consider but one side of a roof truss ꞁaded with wind load at any one time.

16. Panel Loads.—Each truss, except those at the ds of the building, is assumed to support one-half of the ad in each of the two adjacent bays. If the lengths b of e bays are all equal, and the panel lengths p of the rafter e equal, and the load per square foot of roof surface for y loading is w, the panel load W for that loading is given ' the formula

$$W = w\,b\,p$$

For dead and snow loads, the panel loads are vertical, and the stresses in the members are greatest when there is a full panel load at each of the joints of the rafter, except at the supports, where the panel loads will be $\frac{W}{2}$, as shown in Fig. 18. For wind loads, the panel loads are inclined and considered normal to the roof. The stresses caused in the members by the wind load are greatest when there is a full panel load at each joint of the rafter on one-half of the truss, except at the top and bottom joints, where the loads will be $\frac{W}{2}$, as shown in Fig. 19. The panel loads on the trusses at the ends of the building are, as a rule, one-half those on the other trusses.

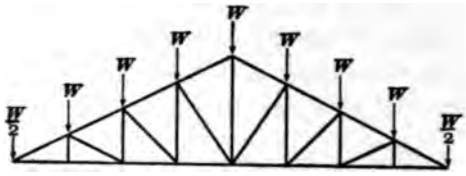


FIG. 18

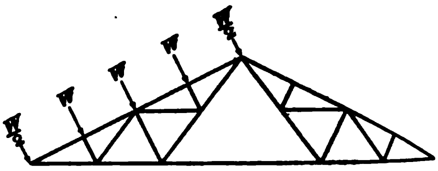


FIG. 19

17. The panel loads on the sloping parts of the roof and monitor shown in Fig. 20 are found in the same way as for other trusses. When the length of the sloping part of the roof of the monitor is equal to a panel length of the truss, the panel loads $\frac{W'}{2}$ on the roof of the monitor are equal to the half panel loads $\frac{W}{2}$ on the truss. In addition, there are horizontal wind forces of $\frac{W''}{2}$ at the top and bottom of the vertical side of the monitor, W'' being the total wind pressure on one bay of the vertical side, computed for a wind pressure of 40 pounds per square foot.

18. If the side walls of the building are of masonry, the wind pressure on them need not be considered in the design of the roof trusses. If the sides are exposed to the action of the wind, and are of wood, corrugated iron, or other building material attached to the outside of the columns, the wind pressure on the side of the building causes stresses in the roof trusses, and hence must be taken into account. It will be assumed that the wind pressure on the side of the building for a length of one bay is W''', and that one-half of this is transmitted to the bottom and one-half to the top of the column, giving a concentrated load of $\frac{W'''}{2}$ at each of these points, as shown in Fig. 20.

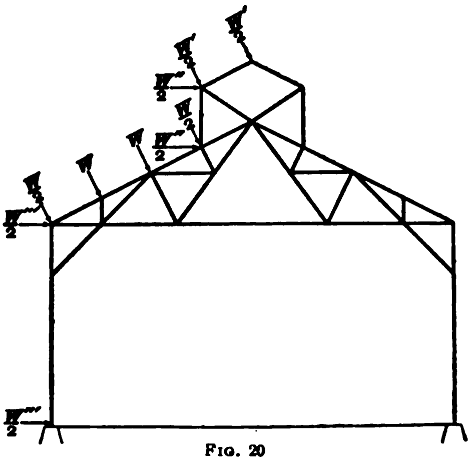


FIG. 20

19. Reactions.—The reactions due to the dead and snow loads are vertical, and are found in the same way as for a simple beam. The reactions due to the wind load may

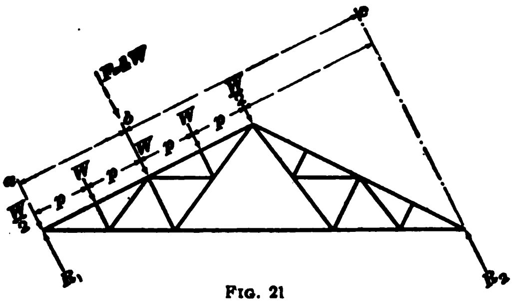


FIG. 21

be vertical or inclined, according to the manner in which the ends of the trusses are connected to the supports. When the ends of the trusses rest on top of the walls, and both ends are anchored down, as is usual with spans less than about

70 feet long, it is customary to assume that both reactions are parallel to the direction of the wind panel loads, as

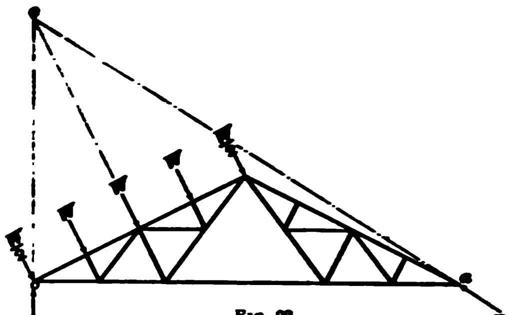


FIG. 22

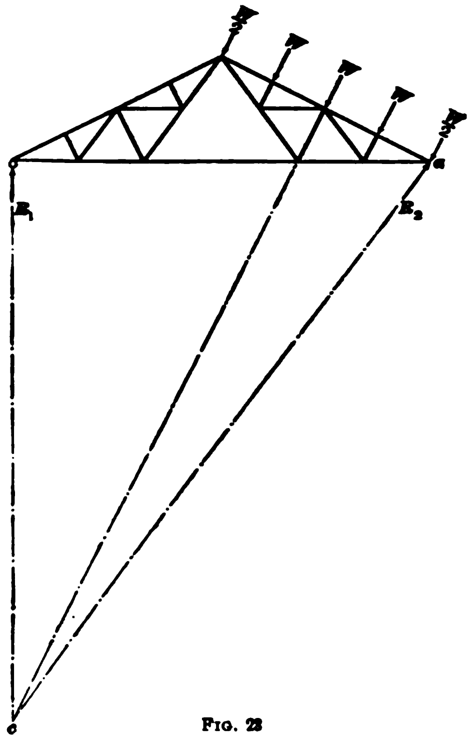


FIG. 23

shown in Fig. 21. The magnitude of the reactions R_1 and R_2 can be found by resolving the resultant F of the wind anel loads into two components passing through the points f support and parallel to the resultant.

20. For longer spans, it is customary to place rollers nder one end to provide for changes in length due to hanges in temperature. Since the truss is free to move orizontally at the expansion end, no horizontal force can e transmitted to the abutment or support at that end, xcept the friction of the rollers, which is usually neglected. ll the horizontal components of the inclined wind panel ads are then assumed to be transmitted to the support at ie fixed end. The directions of the reactions when the wind blowing on the expansion end of the truss are shown in ig. 22. The directions of the reactions when the wind is lowing on the fixed end of the truss are shown in Fig. 23. a each of these figures the circle at the left-hand end repre-ents the rollers under that end of the truss, at which end ie reaction is vertical.

21. In the case represented in Fig. 24, it is customary o find the horizontal and vertical components of the reac-

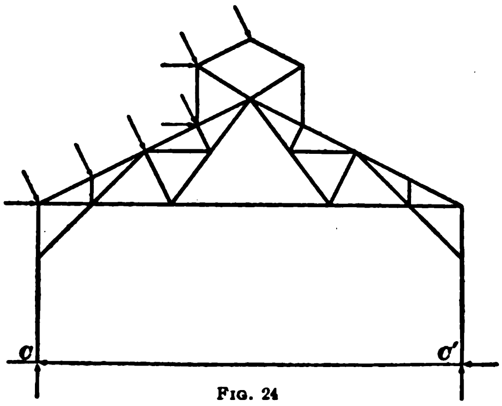


FIG. 24

ons separately. For this purpose, the various wind forces re resolved into their vertical and horizontal components. he vertical components of the reactions at C and C' are hen found by taking moments about C' and C, respectively. he horizontal components of the reactions are found on the assumption that one-half the sum of the horizontal components of the wind forces goes to each support.

If the columns are fixed in direction at the bottom, as is usually the case, points of inflection may be taken half way between the lower ends of the inclined braces and the bases of the columns, the same as in the case of the end post of a through truss bridge.

22. **Method of Calculation.**—The stresses in the members of roof trusses can be found by either the analytic or the graphic method. The members of most roof trusses have so many different inclinations, however, that the work required by the analytic method is comparatively great; so this method is seldom used in practice. The graphic method is especially useful for this purpose. The stresses should be found separately for the vertical and for the inclined loads. When one end of the truss rests on rollers, the stresses due to the wind should be found separately for the wind blowing in each direction. In the latter case, there are three conditions of loading for which the stresses in the members must be found; namely, (1) full snow load together with the dead load; (2) wind pressure on the expansion end; (3) wind pressure on the fixed end. The stresses found in (1) must be combined with those found in (2) or (3) to obtain the greatest stress in each member. In some trusses, there are members in which the stress is tension when the wind acts on one side, and compression when it acts on the other.

The methods of calculation will now be illustrated by practical examples.

FIRST ILLUSTRATIVE EXAMPLE

23. **Data.**—The first illustrative example will be the truss shown in Fig. 16 (*a*), the data for which will be assumed as follows:

Distance between trusses	b = 15 feet
Span	l = 60 feet
Rise	r = 15 feet
Number of panels	n = 8

Roofing material . Slate on 1-inch wooden sheathing
Dead load of rafters, purlins,
and trusses . . . . 10 pounds per square foot
Snow load . . . . . . 30 pounds per square foot
Supports . . Truss anchored to the tops of the walls at both ends

24. Panel Length.—The panel length p is found by the formula in Art. **9.** Since l = 60 feet, r = 15 feet, and n = 8, the formula gives

$$p = \frac{\sqrt{60^2 + 4 \times 15^2}}{8} = 8.385 \text{ feet}$$

25. Panel Loads.—The panel load W for any loading is found by the formula in Art. **16.** In the present case, b = 15 feet and p = 8.385 feet.

1. *Dead Panel Load.*—The dead panel load consists of the weight of the slate, the sheathing, and the rafters, purlins, and trusses. The weight of the slate is given in Art. **13** as 8 pounds per square foot, and the weight of the sheathing as 4 pounds per square foot. The weight of the rafters, purlins, and trusses is given in the data as 10 pounds per square foot. Then, the total dead load w is $8 + 4 + 10 = 22$ pounds per square foot, and for the dead panel load W_d we have

$$W_d = 22 \times 15 \times 8.385 = 2{,}767 \text{ pounds}$$

There will be seven full panel loads on the truss, and, in addition, two half panel loads over the supports. Since the latter loads do not cause any stresses in the members of the truss, they will not be considered here.

2. *Snow Panel Load.*—The snow load is 30 pounds per square foot. Then, for the snow panel load W_s, we have

$$W_s = 30 \times 15 \times 8.385 = 3{,}773 \text{ pounds}$$

3. *Wind Panel Load.*—Since the rise is 15 feet and the span is 60 feet, the roof has a pitch of $\frac{1}{4}$, or a slope of 1 in 2. In Art. **15,** the normal pressure of the wind on a roof having a slope of 1 in 2 is given as 25 pounds per square foot. Then, denoting the wind panel load by W_w,

$$W_w = 25 \times 15 \times 8.385 = 3{,}144 \text{ pounds}$$

There will be three full wind panel loads, and, in addition, the two half panel loads at the bottom and top, respectively. Since the wind panel loads are inclined, it is well to consider the panel load over the support.

26. Stresses Due to Dead and Snow Loads.—In this example, it is best to consider the reactions and stresses due to the vertical loads (combined dead and snow loads) separately from those due to the inclined loads (wind pressure). The dead panel load was found in Art. 25 to be 2,767 pounds; and the snow panel load, 3,773 pounds. Then, the total vertical panel load is 2,767 + 3,773 = 6,540 pounds. There are seven full panel loads, as shown in Fig. 25 (*a*). Since the loading is symmetrical, the reactions are equal, and each is $\frac{7 \times 6,540}{2} = 22,890$ pounds. The stress diagram for this loading is shown in Fig. 25 (*b*).

The panel loads are laid off on the line *0–7*, and, since the reactions are equal, the point *8* is located half way between *0* and *7*. The remainder of the diagram is constructed, as usual, by drawing vectors in the stress diagram parallel to the members in the truss.

It has been explained in *Graphic Statics* that, in constructing the stress diagram for a truss, it is necessary to consider, one after another, the joints at which but two of the stresses are unknown. In the truss under consideration, if the diagram is started for the joint *a*, it will be found possible to construct it also for the joints *b* and *B*. At each of the joints *c* and *C* there will then be three members in which the stresses are unknown, and the stress diagram cannot be drawn directly. In this case, it is customary to consider the next joint *d* on the rafter, and to assume, temporarily, the stress in one of the rafter members. This assumed stress is only an auxiliary quantity used for the purposes of calculation. It is known that the point *13* in the stress diagram will lie on a line *2–13″* through *2* parallel to the member *cd*, and that the point *15* lies in a line through *8* parallel to *C C*. The stress in *cd* will be temporarily assumed equal to *13′–2*, and the polygon *13′–2–3–14′–13′* drawn for the joint *d*. The polygon *13′–14′–15′–12′–13′* is then drawn for the joint *D*. This locates the point *12′*. The stress in *c C*, however, will be represented by a vector through the point *11* parallel

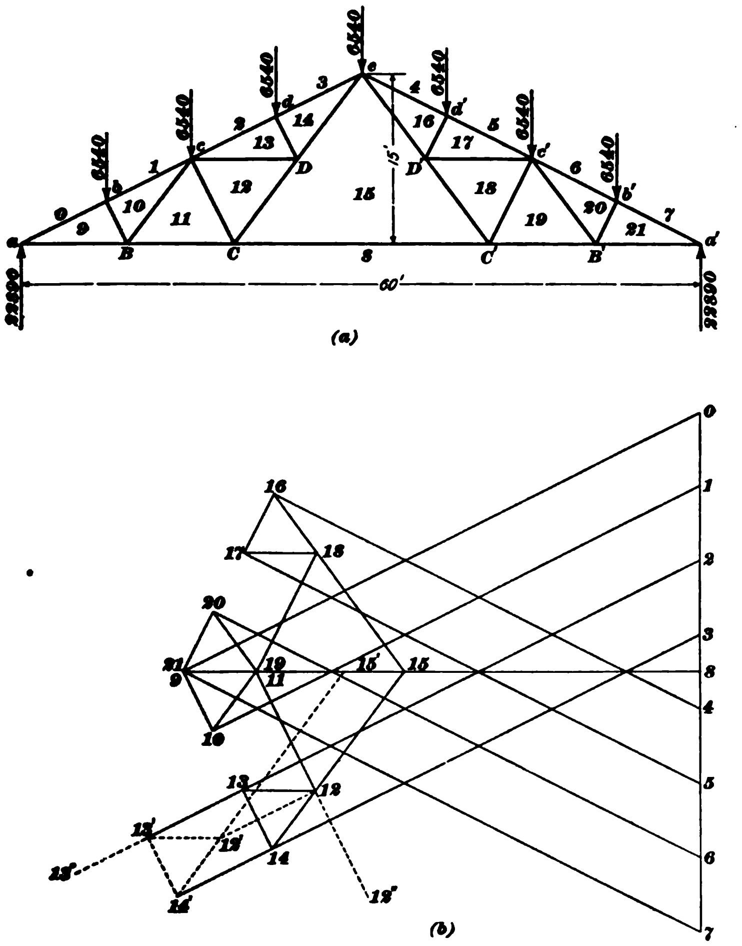


Fig. 25

to *c C*, and point *12* must lie on this line. The actual location of point *12* is then found by drawing the line *12′–12*, parallel to *13′–2* and *3–14′*, to its intersection with *11–12″*, the latter line being parallel to *cC*. This gives the stress in *cC* (*11–12*) and makes it possible to complete the stress diagram without difficulty.

When the loading is symmetrical, the stress diagram is sometimes drawn for only one-half of the truss, but it is well, as in the present case, to draw the entire diagram for a check. The stresses in the members on one side of the center, as scaled from the stress diagram, are as follows, using the plus sign for compression and the minus sign for tension:

Member	Vector	Stress (Pounds)
ab	*0– 9*	+ 51,200
bc	*1–10*	+ 48,300
cd	*2–13*	+ 45,300
de	*3–14*	+ 42,400
aB	*8– 9*	– 45,800
BC	*8–11*	– 39,200
CC'	*8–15*	– 26,200
bB	*9–10*	+ 5,800
Bc	*10–11*	– 6,500
cC	*11–12*	+ 11,700
cD	*12–13*	– 6,500
dD	*13–14*	+ 5,800
CD	*12–15*	– 13,100
De	*14–15*	– 19,600

27. Stresses Due to Wind Load.—The wind panel load was found in Art. 25 to be 3,144 pounds. The loading on the truss is shown in Fig. 26 (*a*). Since the loading is inclined and unsymmetrical, the reactions can be found by the graphic method with much less work than by the analytic method. For convenience of reference, the same notation is used as in Fig. 25; the forces *4–5*, *5–6*, and *6–7* may be taken as zero. The loads are laid off on the line *8'–4*, Fig. 26 (*b*), the pole *P* is chosen, and the rays *P–8'* and *P–4* are drawn. The resultant of all the wind panel loads is equal to *8'–4*, and acts through the center of the left-hand side of the rafter; that is, through the joint *c*, and in the direction *cg*.

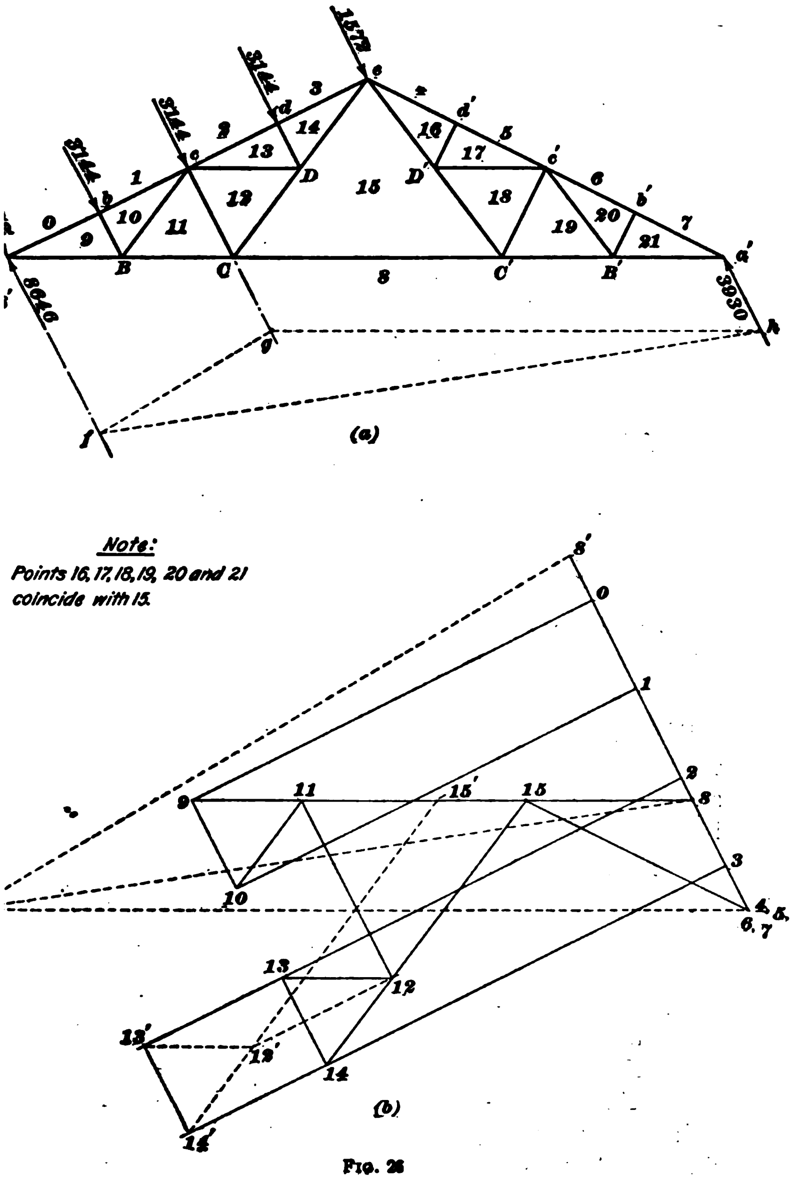

3144
3144
3144
1572
3646
3930
(a)
Note:
Points 16, 17, 18, 19, 20 and 21 coincide with 15.
(b)

Fig. 26

The funicular *gfhg* is drawn, and the reactions *4–8* and *8–8'* are found by drawing the ray *P–8* parallel to the closing line *fh*. The remainder of the work consists simply in drawing the stress diagram shown in Fig. 26 (*b*). The points *16*, *17*, *18*, *19*, *20*, and *21* are not shown in the stress diagram, since they coincide with *15*. This indicates that for this loading there are no stresses in the web members to the right of *15*. The wind stresses in the members, as scaled from the stress diagram, are as follows:

Member	Vector	Stress (Pounds)
ab, *bc*, *cd*, *de*	*0–9*, *1–10*, *2–13*, *3–14*	+ 14,100
aB	*8–9*	– 15,800
BC	*8–11*	– 12,300
CC', *C'B'*, *B'a'*	*8–15*	– 5,300
bB, *dD*	*9–10*, *13–14*	+ 3,100
Bc, *cD*	*10–11*, *12–13*	– 3,500
cC	*11–12*	+ 6,300
CD	*12–15*	– 7,000
De	*14–15*	– 10,500
ed', *d'c'*, *c'b'*, *b'a'*	*4–15*	+ 7,900
(*eD'*, *D'C'*, *d'D'*, *D'c'*, *c'C'*, *c'B'*, *b'B'*)		0

28. Total or Combined Stresses.—The maximum total stresses in the members are found by combining the stresses due to the vertical loads with those due to the inclined loads. The total combined stresses are:

Member	Stress (Pounds)	Member	Stress (Pounds)
ab	+ 65,300	*bB*	+ 8,900
bc	+ 62,400	*Bc*	– 10,000
cd	+ 59,400	*cC*	+ 18,000
de	+ 56,500	*cD*	– 10,000
aB	– 61,600	*dD*	+ 8,900
BC	– 51,500	*CD*	– 20,100
CC'	– 31,500	*De*	– 30,100

It is not necessary to find the stresses in the members on the right-hand side of the center, as they are the same as those in the corresponding members on the left-hand side.

SECOND ILLUSTRATIVE EXAMPLE

29. Data.—The second illustrative example will be of the same general type as the truss shown in Fig. 5. The data will be assumed as follows:

Distance between trusses	b = 22 feet
Span	l = 100 feet
Rise	r = 20 feet
Number of panels	n = 10
Roofing material . .	Corrugated iron, no sheathing
Dead load of rafters, purlins, and trusses .	90 pounds per square foot
Snow load	25 pounds per square foot
Supports	Rollers at left end; right end fixed

The outline of the truss is shown in Fig. 27.

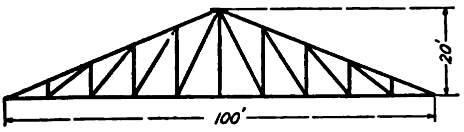


FIG. 27

30. Panel Length.—Since l = 100 feet, r = 20 feet, and n = 10, the formula of Art. **9** gives

$$p = \frac{\sqrt{100^2 + 4 \times 20^2}}{10} = 10.77 \text{ feet}$$

31. Panel Loads.—The panel load W for any loading is given by the formula in Art. **16**. In the present case, b = 22 feet and p = 10.77 feet.

1. *Dead Panel Load.*—The dead load consists of the weight of the corrugated iron, and the rafters, purlins, and trusses. The weight of the corrugated iron is given in Art. **13** as 2 pounds per square foot. The weight of the rafters, purlins, and trusses is given in the data as 20 pounds per square foot. Then, the total dead load is 2 + 20 = 22 pounds per square foot, and for the dead panel load W_d we have

$$W_d = 22 \times 22 \times 10.77 = 5{,}213 \text{ pounds}$$

There will be nine full panel loads, and, in addition, two half panel loads over the supports. Since the latter loads do not cause any stresses in the members of the truss, they will not be considered.

2. *Snow Panel Load.*—The snow load is 25 pounds per square foot (Art. **29**). Then, for the snow panel load W_s we have

$$W_s = 25 \times 22 \times 10.77 = 5{,}923 \text{ pounds}$$

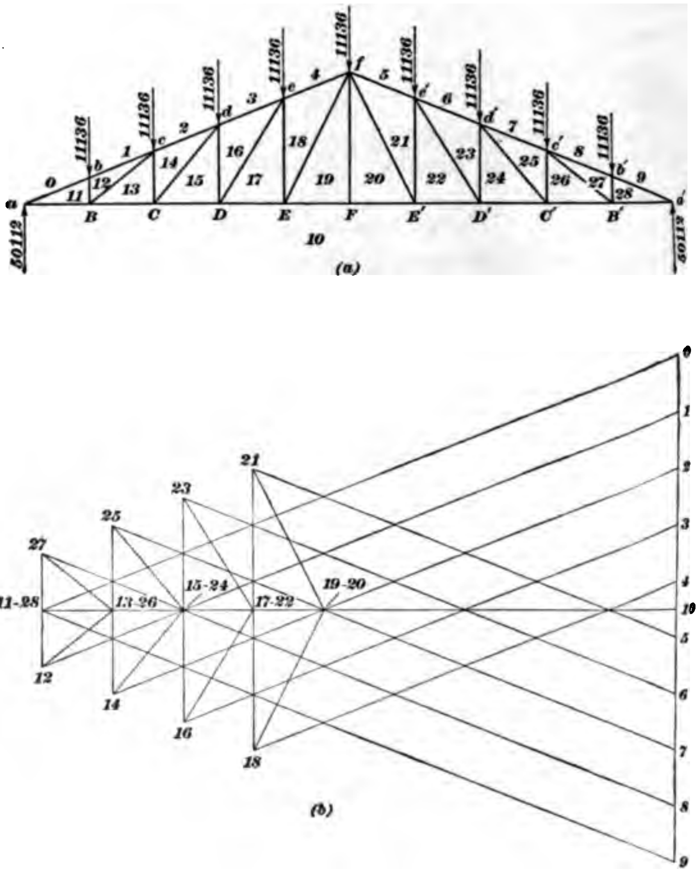


FIG. 28

3. *Wind Panel Load.*—Since the rise is 20 feet and the span is 100 feet, the roof has a $\frac{1}{5}$ pitch or a slope of 1 in $2\frac{1}{2}$. In Art. **15**, the normal pressure of the wind on a roof having

a slope of 1 in $2\frac{1}{2}$ is given as 20 pounds per square foot. Then, for the wind panel load W_w we have

$$W_w = 20 \times 22 \times 10.77 = 4{,}739 \text{ pounds}$$

There will be four full wind panel loads, and, in addition, the two half panel loads at the bottom and top, respectively. Since the wind panel loads are inclined, the load over the support may cause stresses in the members of the truss, and it must be considered.

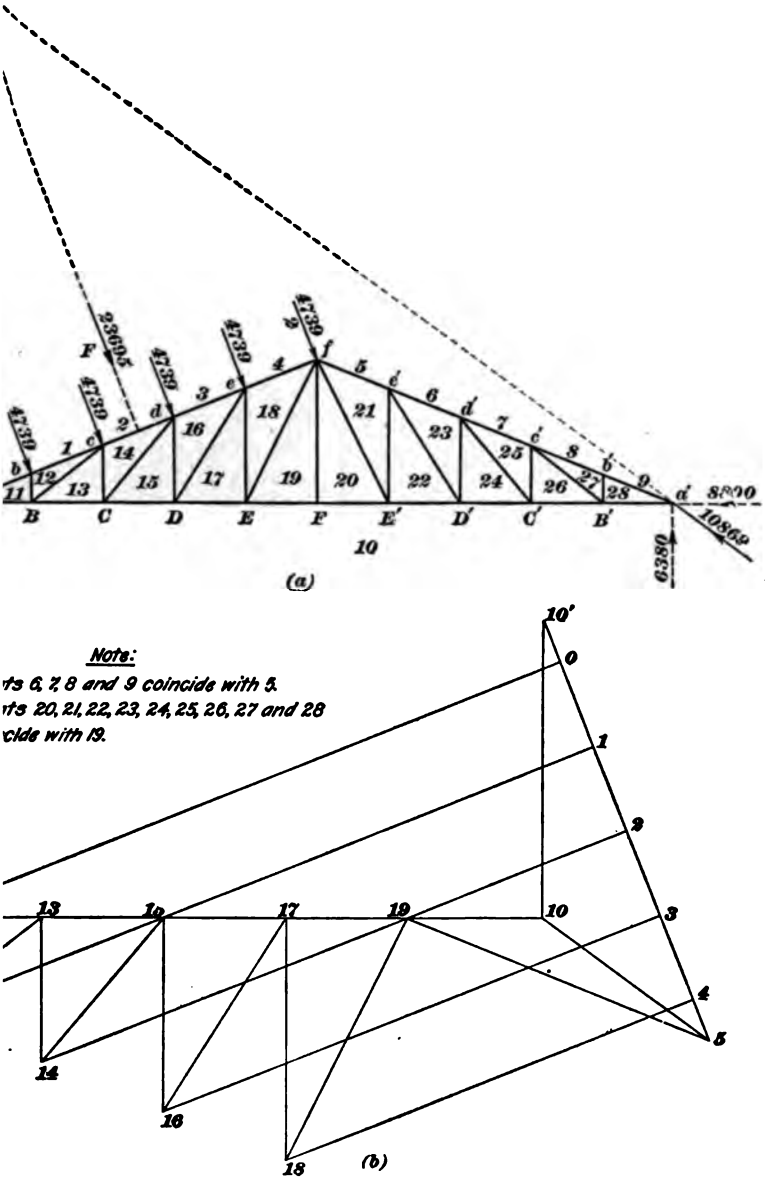
32. Stresses Due to Dead and Snow Loads.—In this example, it is best to consider the reactions and stresses due to the vertical loads (combined dead and snow loads) separately from those due to the inclined loads (wind pressure). The dead panel load was found in Art. 31 to be 5,213 pounds; and the snow panel load, 5,923 pounds. Then, the total vertical panel load is 5,213 + 5,923 = 11,136 pounds. There are nine full panel loads, as shown in Fig. 28 (*a*). Since the loading is symmetrical, each reaction is $\frac{9 \times 11{,}136}{2}$ = 50,112 pounds. The stress diagram for this loading is shown in Fig. 28 (*b*). The panel loads are laid off on the line *0–9*, and, since the reactions are equal, the point *10* is located half way between *0* and *9*. The remainder of the diagram is drawn as usual, presenting no special difficulty. This diagram is symmetrical about the line *10–28*, so that it is necessary to draw but one-half. For a check, however, it is always advisable to draw the complete diagram. The stresses in the members on one side of the center, as scaled from the stress diagram, are as follows:

Member	Vector	Stress (Pounds)
ab	*0–11*	+ 134,900
bc	*1–12*	+ 134,900
cd	*2–14*	+ 119,900
de	*3–16*	+ 104,900
ef	*4–18*	+ 90,000
aB	*10–11*	− 125,300
BC	*10–13*	− 111,400
CD	*10–15*	− 97,400

Member	Vector	Stress (Pounds)
DE	*10–17*	– 83,500
EF	*10–19*	– 69,600
bB	*11–12*	+ 11,100
Bc	*12–13*	– 17,800
cC	*13–14*	+ 16,700
Cd	*14–15*	– 21,700
dD	*15–16*	+ 22,300
De	*16–17*	– 26,300
eE	*17–18*	+ 27,800
Ef	*18–19*	– 31,100
fF	*19–20*	0

33. Stresses Due to Wind Load.—As it is stated in the data that one end of the truss rests on rollers, it is necessary to find the stresses both when the wind blows on the expansion end and when the wind blows on the fixed end. The wind panel load was given in Art. 31 as 4,739 pounds.

1. *Wind on Expansion End.*—The loading when the wind blows on the expansion end is shown in Fig. 29 (*a*). The reaction at the left end is vertical; that at the right end is inclined, and its direction is found by resolving the resultant *F* of the wind panel loads into two components, one vertical and passing through *a*, and the other inclined and passing through *a'*. A vertical line through *a* intersects the resultant of the wind panel loads at *g*, and the line *ga'* gives the line of action of the reaction at *a'*. The wind panel loads are laid off on the line *10'–5*, Fig. 29 (*b*). The magnitudes of the reactions are found by drawing a vertical line through *10'* to its intersection with a line through *5* parallel to *a'g*. The stresses in the members are found by completing the stress diagram in the usual way. Since the diagram is unsymmetrical, it is necessary to draw it complete. The stresses are as follows:

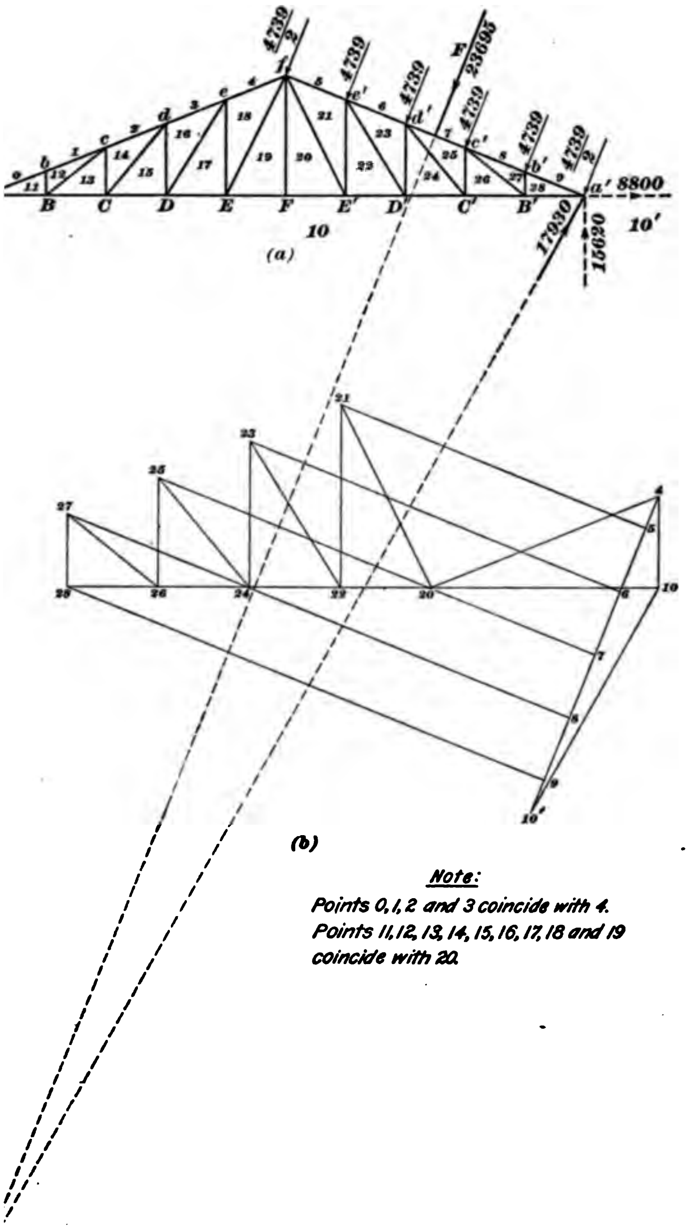


4739
23695
F
4739
4739
4739
8800
10869
6380
B
C
D
E
F
E'
D'
C'
B'
10
(a)
Note:
ts 6, 7, 8 and 9 coincide with 5.
ts 20, 21, 22, 23, 24, 25, 26, 27 and 28
cide with 19.
10'
0
1
2
3
4
5
10
13
17
19
14
16
18
(b)

FIG. 29

Member	Vector	Stress (Pounds)
ab	0–11	+ 36,100
bc	1–12	+ 38,000
cd	2–14	+ 33,100
de	3–16	+ 28,100
ef	4–18	+ 23,100
aB	10–11	− 32,700
BC	10–13	− 26,300
CD	10–15	− 19,900
DE	10–17	− 13,500
$\left(\begin{array}{c} EF, FE', E'D', \\ D'C', C'B', B'a' \end{array}\right)$	10–19	− 7,200
bB	11–12	+ 5,100
Bc	12–13	− 8,200
cC	13–14	+ 7,700
Cd	14–15	− 10,000
dD	15–16	+ 10,200
De	16–17	− 12,000
eE	17–18	+ 12,800
Ef	18–19	− 14,300
$\left(\begin{array}{c} fF, fE', E'e', e'D', \\ D'd', d'C', C'c', \\ c'B', B'b' \end{array}\right)$		0
$fe', e'd, d'c', c'b', b'a'$	5–19	+ 17,200

2. *Wind on Fixed End.*—The loading when the wind blows on the fixed end is shown in Fig. 30 (*a*). The reaction at the left end is vertical; that at the right end is inclined, and its direction is found by resolving the resultant F of the wind panel loads into two components, one vertical and passing through a, and the other inclined and passing through a'. A vertical line through a intersects the resultant of the wind panel loads at g, and the line ga' gives the direction of the reaction at a'. The wind panel loads are laid off on the line *4–10'*, Fig. 30 (*b*). The magnitudes of the reactions are found by drawing a vertical line through *4* to its intersection with a line through *10'* parallel to ga'. The stresses in the members are found by computing the stress diagram in the usual way. They are as follows:



Note:
Points 0, 1, 2 and 3 coincide with 4.
Points 11, 12, 13, 14, 15, 16, 17, 18 and 19 coincide with 20.

FIG. 30

Member	Vector	Stress (Pounds)
ab, bc, cd, de, ef	4–20	+ 17,200
(*bB, Bc, cC, Cd, dD, De, eE, Ef, fF*)		0
(*aB, BC, CD, DE, EF, FE'*)	10–20	– 16,000
fE'	20–21	– 14,300
E'e'	21–22	+ 12,800
e'D'	22–23	– 12,000
D'd'	23–24	+ 10,200
d'C'	24–25	– 10,000
C'c'	25–26	+ 7,700
c'B'	26–27	– 8,200
B'b'	27–28	+ 5,100
E'D'	10–22	– 22,300
D'C'	10–24	– 28,700
C'B'	10–26	– 35,100
B'a'	10–28	– 41,500
fe'	5–21	+ 23,100
e'd'	6–23	+ 28,100
d'c'	7–25	+ 33,100
c'b'	8–27	+ 38,000
b'a'	9–28	+ 36,100

34. **Combined Stresses.**—In order to find the total maximum stresses in the members, it is advisable to tabulate the stresses due to the different loadings, as shown in the table on the next page. Column 1 gives the members, column 2 gives the stresses in the members due to the vertical load, column 3 gives the stresses due to the wind pressure on the expansion end, and column 4 gives the stresses due to the wind pressure on the fixed end. The stresses given in column 5 are the sums of those given in columns 2 and 3; those given in column 6 are the sums of those given in columns 2 and 4. The stresses in column 7 are the maximum stresses in the members. They are taken from columns 5 and 6, the larger being taken in each case.

TABLE OF STRESSES IN MEMBERS

(Pounds)

	2	3	4	5	6	7
m- r	Stresses Due to Vertical Loads	Stress Due to Wind on Expansion End	Stress Due to Wind on Fixed End	Total Stress Wind on Expansion End	Total Stress Wind on Fixed End	Maximum Total Stress
b	+134,900	+36,100	+17,200	+171,000	+152,100	+171,000
c	+134,900	+38,000	+17,200	+172,900	+152,100	+172,900
d	+119,900	+33,100	+17,200	+153,000	+137,100	+153,000
e	+104,900	+28,100	+17,200	+133,000	+122,100	+133,000
f	+ 90,000	+23,100	+17,200	+113,100	+107,200	+113,100
B	−125,300	−32,700	−16,000	−158,000	−141,300	−158,000
C	−111,400	−26,300	−16,000	−137,700	−127,400	−137,700
D	− 97,400	−19,900	−16,000	−117,300	−113,400	−117,300
E	− 83,500	−13,500	−16,000	− 97,000	− 99,500	− 99,500
F	− 69,600	− 7,200	−16,000	− 76,800	− 85,600	− 85,600
B	+ 11,100	+ 5,100	0	+ 16,200	+ 11,100	+ 16,200
c	− 17,800	− 8,200	0	− 26,000	− 17,800	− 26,000
C	+ 16,700	+ 7,700	0	+ 24,400	+ 16,700	+ 24,400
d	− 21,700	−10,000	0	− 31,700	− 21,700	− 31,700
D	+ 22,300	+10,200	0	+ 32,500	+ 22,300	+ 32,500
e	− 26,300	−12,000	0	− 38,300	− 26,300	− 38,300
E	+ 27,800	+12,800	0	+ 40,600	+ 27,800	+ 40,600
f	− 31,100	−14,300	0	− 45,400	− 31,100	− 45,400
F	0	0	0	0	0	0
E′	− 31,100	0	−14,300	− 31,100	− 45,400	− 45,400
e′	+ 27,800	0	+12,800	+ 27,800	+ 40,600	+ 40,600
D′	− 26,300	0	−12,000	− 26,300	− 38,300	− 38,300
d′	+ 22,300	0	+10,200	+ 22,300	+ 32,500	+ 32,500
C′	− 21,700	0	−10,000	− 21,700	− 31,700	− 30,700
c′	+ 16,700	0	+ 7,700	+ 16,700	+ 24,400	+ 24,400
B′	− 17,800	0	− 8,200	− 17,800	− 26,000	− 26,000
b′	+ 11,100	0	+ 5,100	+ 11,100	+ 16,200	+ 16,200
E′	− 69,600	− 7,200	−16,000	− 76,800	− 85,600	− 85,600
D′	− 83,500	− 7,200	−22,300	− 90,700	−105,800	−105,800
C′	− 97,400	− 7,200	−28,700	−104,600	−126,100	−126,100
B′	−111,400	− 7,200	−35,100	−118,600	−146,500	−146,500
a′	−125,300	− 7,200	−41,500	−132,500	−166,800	−166,800
e′	+ 90,000	+17,200	+23,100	+107,200	+113,100	+113,100
d′	+104,900	+17,200	+28,100	+122,100	+133,000	+133,000
c′	+119,900	+17,200	+33,100	+137,100	+153,000	+153,000
b′	+134,900	+17,200	+38,000	+152,100	+172,900	+172,900
a′	+134,900	+17,200	+36,100	+152,100	+171,000	+171,000

It is not necessary to find the minimum stresses in the members of roof trusses, unless the stresses in some of the members are reversed by the wind. The stress in fF is given as zero. This member simply serves to support the bottom chord at the center, and is sometimes omitted.

THIRD ILLUSTRATIVE EXAMPLE

35. Data.—The third illustrative example will be of the same general type as the truss shown in Fig. 15. The data will be assumed as follows:

Distance between trusses . . . . . . b =	18 feet
Span . . . . l =	80 feet
Rise . . . . r =	20 feet
Height of monitor	11 feet
Number of panels . . . . n =	8
Distance from bottom chord to base of columns	30 feet
Roofing material . . Concrete slab $2\frac{1}{2}$ inches thick	
Vertical sides of monitor . . . . Glass skylights	
Dead load of rafters, purlins, and trusses . 18 pounds per square foot	
Snow load . . . . . . 30 pounds per square foot	
Supports . . . . . Columns fixed at the bottoms	

The outline of the truss is shown in Fig. 31.

36. Panel Length.—Since l = 80 feet, r = 20 feet, and n = 8, the formula of Art. **9** gives

$$p = \frac{\sqrt{80^2 + 4 \times 20^2}}{8} = 11.18 \text{ feet}$$

37. Panel Loads.—The panel load W for any loading is given by the formula in Art. **16**. In the present case, b = 18 feet and p = 11.18 feet.

1. *Dead Panel Load.*—The dead load consists of the weight of the concrete slab, and that of the rafters, purlins, and trusses. In addition, the weight of the vertical sides of the monitor must be considered. The weight of concrete per square foot is given in Art. **13** as 12 pounds per inch

of thickness; since the slab is $2\frac{1}{2}$ inches thick, the weight is $2\frac{1}{2} \times 12 = 30$ pounds per square foot. The weight of rafters, purlins, and trusses is 18 pounds per square foot. Then, the total dead load on the sloping portion is $30 + 18 = 48$ pounds per square foot, and for the dead panel load W_d we have:

$$W_d = 48 \times 18 \times 11.18 = 9{,}660 \text{ pounds}$$

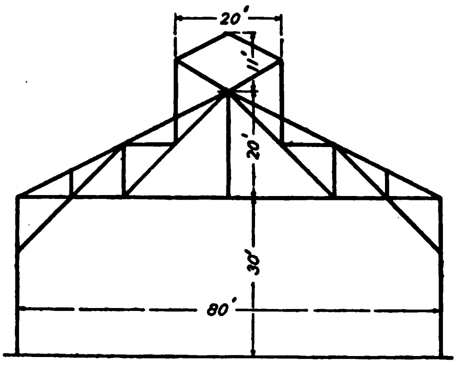


FIG. 31

There are full panel loads of 9,660 pounds at b, c, g, c', and b', Fig. 32 (a), and half panel loads of 4,830 pounds, at a, d, e, e', d', and a'. Since the half panel loads at e and d (and the same applies to e' and d') are in the same straight line, they will be considered together in laying out the force polygon as though they were both applied at e (e' for the other side). The half panel loads at a and a' must be considered, since they cause stresses in the columns.

The weight of the sides of the monitor is 8 pounds per square foot. (See Art. 13.) Then, for the weight W_m of each side per bay, we have

$$W_m = 8 \times 11 \times 18 = 1{,}584 \text{ pounds}$$

All this weight is applied at d and d', but in laying out the force polygon it is better to consider it applied at e and e'.

The effect of considering the loads at d and d' as applied at e and e', is to make the stresses in the verticals de and $d'e'$ greater than the actual stresses by the amount of the loads at d and d'. The correction can easily be made later when the stresses are scaled from the stress diagram.

2. *Snow Panel Load.*—The snow load is 30 pounds per square foot. Then, for the snow panel load W_s we have

$$W_s = 30 \times 18 \times 11.18 = 6{,}037 \text{ pounds}$$

There are full panel loads at b, c, g, c', and b', and half panel loads at a, d, e, e', d', and a'. The half panel loads at

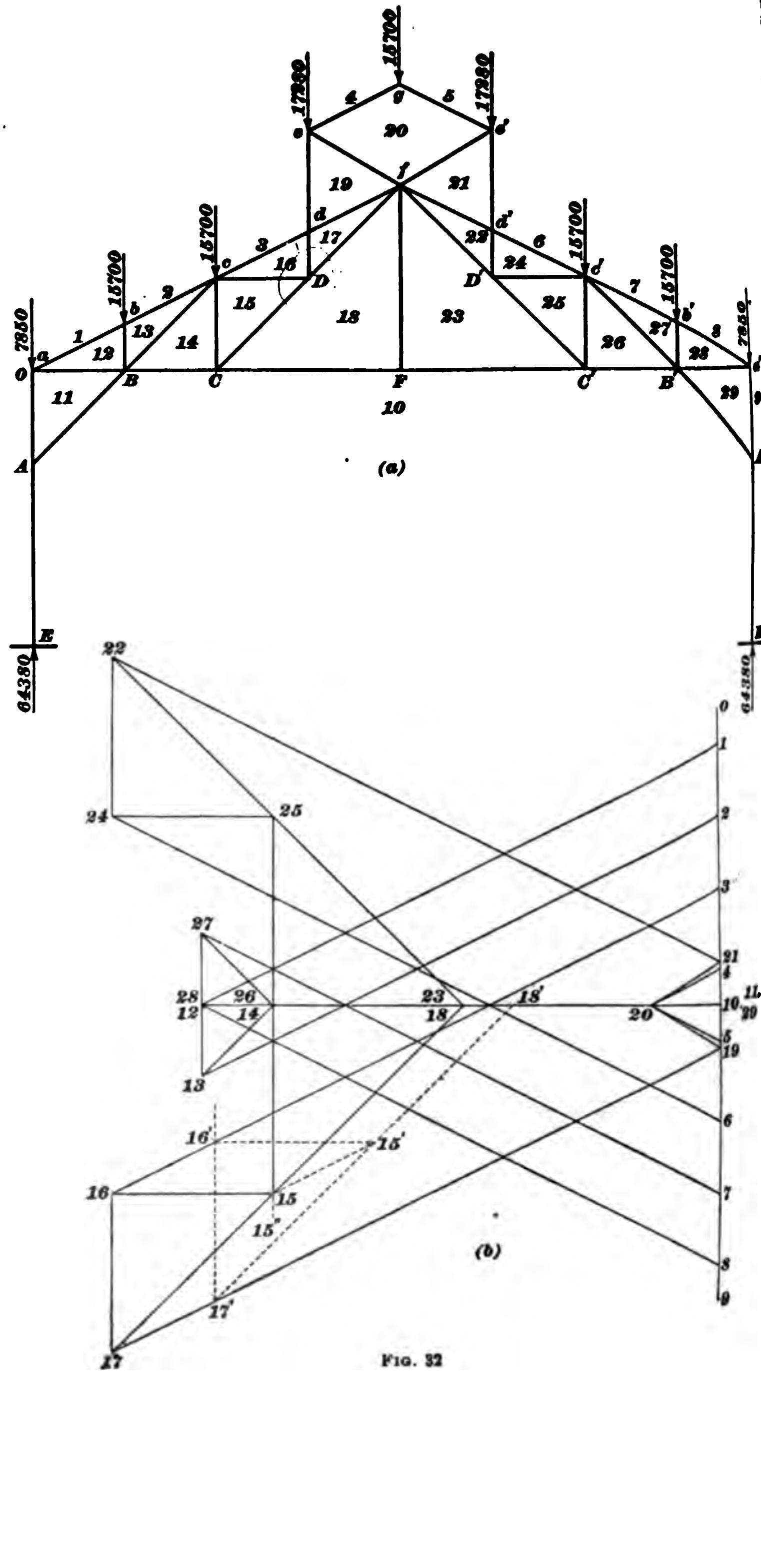


FIG. 32

c and d (c' and d') will be considered together as [illegible] loads at c (c') in laying out the force polygon. [illegible] snow load on the sides of the monitor.

3. *Wind Panel Load.*—Since the rise is 20 feet [illegible] is 80 feet, the roof has a $\frac{1}{4}$ pitch or a [illegible] Art. 15, the normal pressure on a roof having [illegible] is given as 25 pounds per square foot. [illegible] panel load W_n on the inclined portion [illegible]

$$W_n = 25 \times 18 \times 11.18 = 5{,}0[illegible]$$

There are full panel loads at b and c, [illegible] at a, d, e, and g.

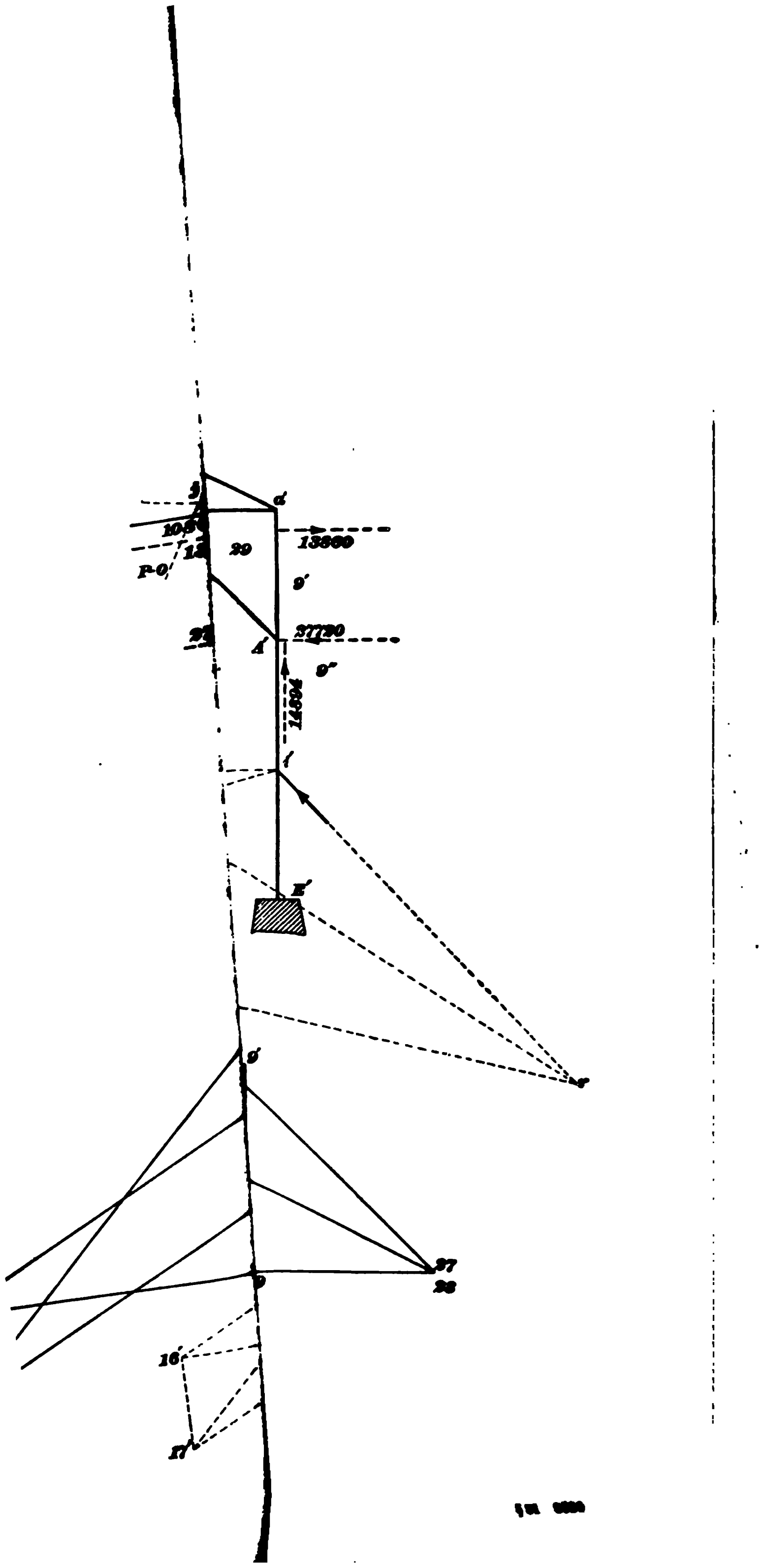
In addition to the above, the wind [illegible] sides of the monitor and the building [illegible] is customary to consider this pressure as [illegible] to 40 pounds per square foot. The total [illegible] of one side of the monitor is $40 \times 1[illegible]$ [illegible] one-half of which, 3,960 pounds, may be [illegible] joints c and d. The total pressure [illegible] side of the building is $40 \times 18 \times 30$ [illegible] half of which, 10,800 pounds, may be [illegible]

38. Stresses Due to Dead and Snow Loads. [illegible] example, as in those preceding, [illegible] cal loads will be treated separately [illegible] inclined loads. The vertical [illegible] the truss are shown in Fig. 32 [illegible] and a' are the sum of the [illegible] due to the weight of concrete [illegible] loads at c and c' include [illegible] Since the loading is symmetrical [illegible] In the present case of [illegible] assume that the stresses in AB [illegible]

The stress diagram is [illegible] in the same way as though [illegible] at a and a', the members AB [illegible] panel loads are laid off [illegible] tions are equal, the point [illegible] and 9. The stress diagram [illegible] without difficulty. It is then necessary to consider joint *g*, then *e* and *e'*, and then *d*. At the latter joint, the stress in *df* is temporarily assumed equal to *19–17'*, and the diagram *19–17'–16'–3–19* is drawn in dotted lines for the joints *d*; joint *D* is then considered, and the diagram *16'–17'–18'–15'–16'* drawn in dotted lines. The point *14* has already been found, and it is known that *15* lies on a line *14–15''* passing through *14* and parallel to *c C*. The point *15* is then found by drawing *15–15'* parallel to *3–16'* to its intersection *15* with *14–15''*. The remainder of the stress diagram can now be drawn without further difficulty. The stresses in *de* and *d'e'*, as indicated by the stress diagram, are each too large by 9,430 pounds, on account of the loads at *d* and *d'* having been considered as applied at *e* and *e'*.

The stresses in the members on one side of the center, as scaled from the stress diagram, are as follows:

Member	Vector	Stress (Pounds)
EA	*10– 0*	+ 64,400
Aa	*10– 0*	+ 64,400
AB	*10–11*	0
ab	*1–12*	+ 126,400
bc	*2–13*	+ 126,400
cd	*3–16*	+ 147,500
df	*19–17*	+ 147,500
fe	*19–20*	– 18,300
de	*3–19*	(34,500 – 9,400 + 25,100)
eg	*4–20*	+ 17,600
aB	*11–12*	– 113,100
BC	*10–14*	– 97,400
CF	*10–18*	– 56,500
bB	*12–13*	+ 15,700
Bc	*13–14*	– 22,200
cC	*14–15*	+ 40,800
cD	*15–16*	– 34,600
CD	*15–18*	– 57,700
dD	*16–17*	+ 34,600
Df	*17–18*	– 106,600
fF	*18–23*	0



13860
27720
14504
9'
9''
A'
E'
27
28
16'
17'

!9. **Stresses Due to Wind Load.**—The diagram of the
;s and the wind loads is shown in Fig. 33 (*a*). At joint *a*
re is a horizontal pressure of 10,800 pounds due to the
ssure against the side of the building, and also the
ined pressure of $\frac{5031}{2}$ pounds, due to the normal pressure
the roof. At joints *b* and *c* there are full inclined panel
ls of 5,031 pounds, due to the normal pressure on the
f. At *d* and *e* there are inclined panel loads of $\frac{5031}{2}$ pounds
to the normal pressure on the roof, and horizontal pres-
es of 3,960 pounds due to the pressure against the side of
monitor. At the peak *g* there is an inclined pressure
$\frac{5031}{2}$ pounds. The forces shown in dotted lines will be
lained later.

l0. In finding the reactions, it is necessary to consider
condition of the columns at *E* and *E'*. In Art. **35,** it is
ted that the columns are fixed at the bottoms, so that
nts of inflection may be assumed at *i* and *i'*, half way
ween *A* and *E*, *A'* and *E'*, respectively. The reactions at *i*
l *i'* will be inclined, and it is impossible to determine their
ections except by means of some arbitrary assumption con-
ning the distribution of the forces between *i* and *i'*. In
ctice it is frequently assumed that the horizontal com-
ent of each reaction is equal to one-half the sum of the
izontal components of the wind forces. This assumption
bably gives the reactions as close to their actual values as
other assumption that can be made. It becomes neces-
y, therefore, to find the resultant of all the wind forces,
l then to resolve it into two components, one passing
ough *i* and one passing through *i'*, their horizontal com-
ents being equal to each other.

The force polygon *0-1-2-3-4-5-6-7-8-9*, Fig. 33 (*b*),
irst laid out in a convenient position, the first point, *0*,
ng located for convenience on the line *i i'* connecting the
nts *i* and *i'*. The point *P* is then chosen for a pole, and
located arbitrarily, to simplify the work, on a horizontal
through the last point, *9*, of the force polygon. The
s *P-0*, *P-2*, *P-3*, *P-4*, *P-6*, *P-8*, and *P-9* are now drawn.

The ray *P–1* is left out because the dotted line *0–2* in the force polygon and *j a* in the space diagram represent the resultant of the forces *0–1* and *1–2*, and this resultant can be considered instead of its two components. In like manner, the force *4–6* in the force polygon and *m d* in the space diagram may be considered instead of the two components *4–5* and *5–6*; the resultant *e n* may be considered instead of *6–7* and *7–8*. The line *0–9*, Fig. 33 (*b*), gives the magnitude of the resultant of all the wind forces; the intersection *q* of the end strings of the funicular *q j k l m n p q*, drawn in the usual way, gives a point in the line of action of the resultant. A line is then drawn through *q* parallel to *0–9* to represent the resultant. The vertical and horizontal components of *0–9* are given by *0–s* and *s–9*, Fig. 33 (*b*). The point *9''*, half way between *s* and *9*, divides the horizontal component of *0–9* into two equal parts, each of which is equal to the horizontal component of one of the reactions.

41. The vertical components of the reactions are found by considering the resultant *F* resolved into its vertical and horizontal components at *t*, the intersection of its line of action with *i i'*. The vertical component *0–s* is laid off on *A E*, vertically below *i*, so that *i u* = *0–s*. The line *u i'*, the vertical *t v*, and horizontal *v v'* are then drawn. Then *u v'* is the vertical component of the reaction at *i'*, and *v' i* is the vertical component of the reaction at *i*. The point *10*, where a vertical through *9''* intersects *v v'*, is a point on the lines in the force polygon that represent the reactions. Drawing the lines *9–10* and *10–0*, the magnitudes and directions of the reactions are found. The lines of action of the reactions are then found by drawing *i r* through *i* parallel to *10–0*, and *i' r* through *i'* parallel to *9–10*. The lines *i r* and *i' r* should intersect on the line of action of the resultant. This is a very good check on the work thus far.

The vertical components of the reactions cause direct stresses in the columns. The horizontal components of the reactions cause bending moments and shearing stresses in the columns.

42. In order to find the stresses in the members of the truss by the method of the stress diagram, it is necessary to treat the shearing stresses in the posts or columns $a\,i$ and $a'\,i'$ as external forces applied at the joints a, A, a', and A'. To find the values of these shearing stresses, vertical lines are drawn through s and $9''$, Fig. 33 (b), the line s–$9''$ representing the horizontal component of the reaction at i. The line drawn through s intersects a horizontal line drawn through A at the point s'; the line through $9''$ intersects a horizontal line drawn through a, which in this case coincides with the chord of the truss, at the point w. The line $w\,s'$ is then drawn and continued to the point $10''$, its intersection with $i\,i'$. Then, $10''$–0, equal to 13,860 pounds, is equal to the shear in $a\,A$ (and also in $a'\,A'$) and may be represented as an external force acting at a (and also at a'), as shown by the dotted lines in the figure. The shear in $i\,A$, equal to 13,860 pounds, is equal to $10'$–0; then, the horizontal force acting at the joint A due to the shears in $a\,A$ and $A\,i$ is equal to the sum of these shears, which is 27,720 pounds. This may be treated as an external force acting at A (and A'), as shown by dotted lines. Those portions of the columns that lie below A and A' can be assumed to be removed and the direct stresses in them represented by the vertical external forces shown at A and A'. The force polygon for the external forces, including the forces shown in dotted lines at a, A, a', and A', is 10–$10'$–$10''$–0–1–2–3–4–5–6–7–8–9–$9'$–$9''$–10, Fig. 33 (b). The stress diagram can now be drawn in the usual way, ignoring the reaction at i and i', and considering the parts $i\,E$ and $i'\,E'$ of the posts to be removed. It is not necessary to draw the stress diagram for wind load when the wind is blowing on the right of the truss; for it is obvious that when the wind blows from the right, the stress in any member is the same as the stress in the corresponding member on the other side of the center when the wind blows from the left. The stresses in the members, when the wind is blowing from the left, have the values given in the table on page 38, these values having been scaled, as usual, from the stress diagram.

Member	Vector	Stress (Pounds)
ab	2–12	+ 63,900
bc	3–13	+ 66,400
cd	4–16	+ 38,200
df	19–17	+ 43,900
fe	19–20	+ 4,000
eg	8–20	+ 1,900
AB	10–11	– 39,200
bB	12–13	+ 5,600
Bc	13–14	– 47,200
cC	14–15	+ 25,200
cD	15–16	– 5,800
CD	15–18	– 35,700
Df	17–18	– 43,900
dD	16–17	+ 5,800
ed	6–19	+ 1,100
aB	11–12	– 31,400
BC	10–14	– 25,700
CF	10–18	– 500
FC'	10–23	– 500
$C'B'$	10–26	+ 11,800
$B'a'$	28–29	+ 11,800
$d'e'$	21– 9	+ 3,100
$d'D'$	22–24	+ 3,100
$D'f$	22–23	+ 13,000
$C'D'$	23–25	+ 17,400
$c'D'$	24–25	– 3,100
$c'C'$	25–26	– 12,300
$B'c'$	26–27	+ 39,200
$b'B'$	27–28	0
$A'B'$	10–29	+ 39,200
$e'g$	20– 9	+ 3,100
fe'	20–21	– 3,300
$d'f$	21–22	+ 5,800
$c'd'$	9–24	+ 5,800
$b'c'$	9–27	– 28,700
$b'a'$	9–28	– 28,700
EA'	10–10′	+ 3,100
Aa	10″–11	+ 30,800
$E'A'$	9″–10	+ 14,900
$A'a'$	9′–29	– 12,800
fF	18–23	0

43. Combined Stresses.—The combined stresses are ven in the table on page 40. In column 2 are given the resses due to vertical loads, as found in Art. **38.** In ·lumn 3 are given the stresses in all the members due to ind on the left, as found in Art. **42.** The combined or ·tal stresses in the members when the wind blows from the :ft are given in column 4; they are found by adding algeraically for each member the stresses given for that member n columns 2 and 3. The combined or total stresses in the nembers when the wind blows from the right are given in olumn 5; they are found by taking for each member the tress given in column 4 for the corresponding member on ie other side of the center. The maximum and minimum ombined stresses are given in column 6; the former are rinted in heavy type for the members on the left of the enter; the latter are printed in ordinary type for the memers on the right of the center. The maximum stresses, iven in column 6 for the members on the left of the center, re the same for corresponding members on the right of the enter; so it is necessary to give them but once. The same tatement is true as regards the minimum stresses.

EXAMPLES FOR PRACTICE

1. A truss having the form shown in Fig. 1 has a span of 20 feet id a rise of 5 feet. If the trusses are 8 feet apart, and the dead load 40 lb. per square foot, what is the dead-load stress in *a c*?
Ans. —3,600 lb.

2. A truss having the form shown in Fig. 2 has a span of 30 feet id a rise of 6 feet. If the trusses are 10 feet apart, and the ends are ced at the tops of both walls, what is the wind stress in *b c* when the ind is coming from the left? Ans. +2,300 lb.

3. A truss having the form shown in Fig. 3 has a span of 36 feet and rise of 12 feet. If the trusses are 9 feet apart, and the left end of ich truss rests on rollers, what is the wind stress in *a d* when the wind blowing on the left side of the truss? Ans. —2,000 lb.

4. What is the wind stress in *a d* in the truss described in example 3, hen the wind is blowing on the right side of the truss?
Ans. —2,600 lb.

TABLE OF STRESSES

(Pounds)

1	2	3	4	5	6
Member	Stress Due to Vertical Loads	Stress Due to Wind on Left	Combined Stress Wind on Left	Combined Stress Wind on Right	Maximum or Minimum Combined Stress
a b	+ 126,400	+ 63,900	+ 190,300	+ 97,700	**+ 190,300**
b c	+ 126,400	+ 66,400	+ 192,800	+ 97,700	**+ 192,800**
c d	+ 147,500	+ 38,200	+ 185,700	+ 153,300	**+ 185,700**
d f	+ 147,500	+ 43,900	+ 191,400	+ 153,300	**+ 191,400**
f e	− 18,300	+ 4,000	− 14,300	− 21,600	**− 21,600**
e g	+ 17,600	+ 1,900	+ 19,500	+ 20,700	**+ 20,700**
A B	0	− 39,200	− 39,200	+ 39,200	**+ 39,200**
b B	+ 15,700	+ 5,600	+ 21,300	+ 15,700	**+ 21,300**
B c	− 22,200	− 47,200	− 69,400	+ 17,000	**− 69,400**
c C	+ 40,800	+ 25,200	+ 66,000	+ 28,500	**+ 66,000**
c D	− 34,600	− 5,800	− 40,400	− 37,700	**− 40,400**
C D	− 57,700	− 35,700	− 93,400	− 40,300	**− 93,400**
D f	− 106,600	− 43,900	− 150,500	− 93,600	**− 150,500**
d D	+ 34,600	+ 5,800	+ 40,400	+ 37,700	**+ 40,400**
e d	+ 25,100	+ 1,100	+ 26,200	+ 28,200	**+ 28,200**
a B	− 113,100	− 31,400	− 144,500	− 101,300	**− 144,500**
B C	− 97,400	− 25,700	− 123,100	− 85,600	**− 123,100**
C F	− 56,500	− 500	− 57,000	− 57,000	**− 57,000**
F C′	− 56,500	− 500	− 57,000	− 57,000	− 56,500
C′ B′	− 97,400	+ 11,800	− 85,600	− 123,100	− 85,600
B′ a′	− 113,100	+ 11,800	− 101,300	− 144,500	− 101,300
d′ e′	+ 25,100	+ 3,100	+ 28,200	+ 26,200	+ 25,100
d′ D′	+ 34,600	+ 3,100	+ 37,700	+ 40,400	+ 34,600
D′ f	− 106,600	+ 13,000	− 93,600	− 150,500	− 93,600
C′ D′	− 57,700	+ 17,400	− 40,300	− 93,400	− 40,300
c′ D′	− 34,600	− 3,100	− 37,700	− 40,400	− 34,600
c′ C′	+ 40,800	− 12,300	+ 28,500	+ 66,000	+ 28,500
B′ c′	− 22,200	+ 39,200	+ 17,000	− 69,400	+ 17,000
b′ B′	+ 15,700	0	+ 15,700	+ 21,300	+ 15,700
A′ B′	0	+ 39,200	+ 39,200	− 39,200	− 39,200
e′ g	+ 17,600	+ 3,100	+ 20,700	+ 19,500	+ 17,600
f e′	− 18,300	− 3,300	− 21,600	− 14,300	− 14,300
d′ f	+ 147,500	+ 5,800	+ 153,300	+ 191,400	+ 147,500
c′ d′	+ 147,500	+ 5,800	+ 153,300	+ 185,700	+ 147,500
b′ c′	+ 126,400	− 28,700	+ 97,700	+ 192,800	+ 97,700
b′ a′	+ 126,400	− 28,700	+ 97,700	+ 190,300	+ 97,700
E A	+ 64,400	+ 3,100	+ 67,500	+ 79,300	**+ 79,300**
A a	+ 64,400	+ 30,800	+ 95,200	+ 51,600	**+ 95,200**
E′ A′	+ 64,400	+ 14,900	+ 79,300	+ 67,500	+ 64,400
A′ a′	+ 64,400	− 12,800	+ 51,600	+ 95,200	+ 51,600
f F	0	0	0	0	0

DESIGN

TRUSSES AND LATERAL SYSTEMS

44. Kinds of Trusses.—Roof trusses are made of wood and of steel. The kind of material used depends on the particular conditions in each case. In temporary buildings, and in buildings that are not required to be fireproof, wooden trusses may be used. In all permanent work of any importance, and in all fireproof buildings, steel trusses are used. Riveted trusses are used for short spans, less than 75 to 100 feet, and pin-connected trusses for long spans, greater than 75 to 100 feet. The dividing line between these two kinds of trusses is not so clearly defined in the case of roof trusses as in the case of bridge trusses. In general, riveted trusses are preferable when the stresses in some of the members are reversed by the wind.

45. Working Stresses.—The allowable or working stresses for steel roof trusses are the same as those given for highway bridges in *Bridge Specifications*. The working stresses for wooden roof trusses are the same as those given in *Wooden Bridges*.

46. Design of Main Members.—The same general types of members are used for roof trusses as for bridge trusses. In addition, loop-welded rods are sometimes used for main members. The principal difference lies in the fact that, as the stresses in the members of roof trusses are, as a rule, less than the stresses in the members of bridge trusses, the members in the former are smaller than in the latter. For this reason, roof trusses are much narrower than bridge trusses, and in the shorter spans, the web members are connected to the chord and rafter by means of one web connection plate instead of two. The principles that govern the design of the main members of roof trusses are the same as those that have been illustrated and explained in connection with bridge design.

47. Lateral Systems.—Roof trusses are connected by lateral bracing in the planes of the rafters, by transverse bracing between the trusses in vertical planes or at right angles to the rafters, and in some cases by lateral bracing in the plane of the rafters. The same general style of bracing is used as for bridge trusses; but, there being, as a rule, more roof trusses in a roof than bridge trusses in a bridge, the arrangement of the lateral systems is somewhat different.

It is customary to insert lateral trusses in each end bay, and also in every second or third intermediate bay throughout the length of the building, as shown in Fig. 34. The

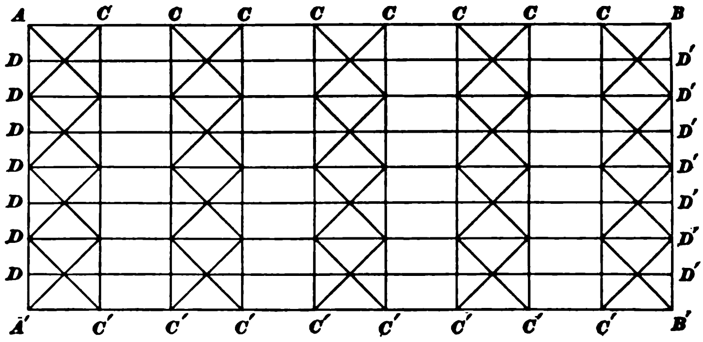


FIG. 34

figure is the plan of the roof of a building: AA' and BB' are the end trusses, and CC' are the intermediate trusses. The purlins AB, DD', and $A'B'$ run the entire length of the building, being spliced at each truss, or at every other truss. The common rafters that run at right angles to the purlins are not shown in the figure. The end bays and every second bay throughout the building are supplied with lateral trusses. The lateral truss in each end bay is designed to resist the wind pressure on that end of the building; the other lateral trusses are then made the same as those in the end bays. Lateral and loop-welded rods are frequently used for the diagonals of lateral systems of roofs.

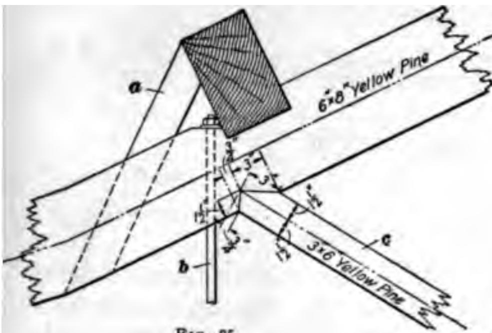


FIG. 35

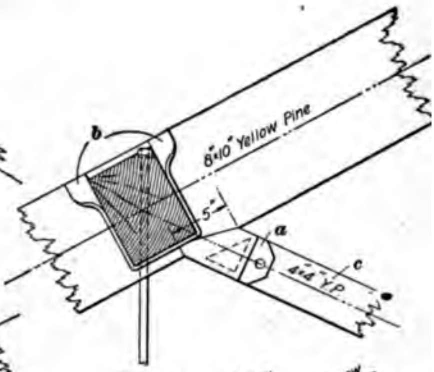


FIG. 36

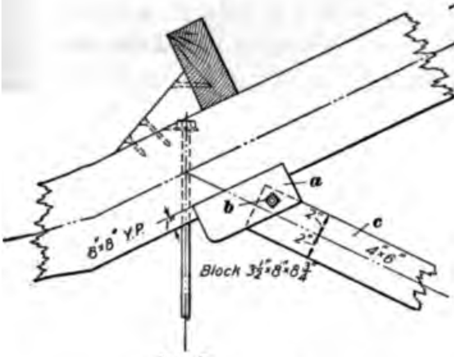


FIG. 37

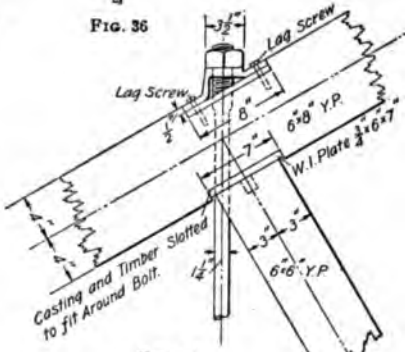


FIG. 38

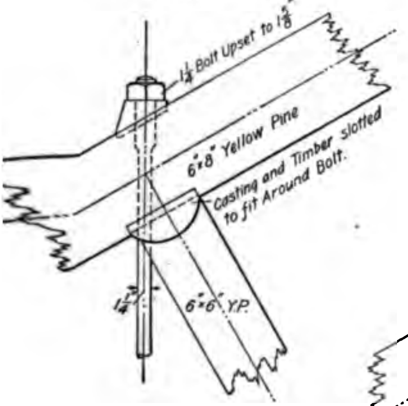


FIG. 39

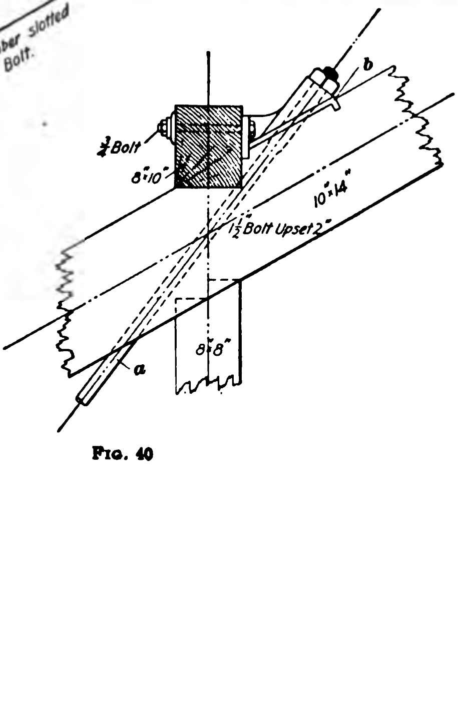


FIG. 40

CONNECTIONS

WOODEN TRUSSES

48. Rafter Joints.—Figs. 35 to 40 show typical joints for the rafters of wooden trusses. In Fig. 35, the purlin, shown in cross-section, is placed on top of the rafter and at right angles to it. Both the purlin and rafter are framed where they connect. The purlin is held in position by the inclined brace *a*. The vertical rod *b* passes through the rafters, and the strut *c* is cut at the end to fit into a shoulder cut in the bottom of the rafter.

In Fig. 36, the purlin is attached to the rafters by means of the beam hanger *b*. The inclined strut *c* is connected to the casting *a* that bears against the rafter. In Fig. 37, a wooden block *a* is inserted in the bottom of the rafter, and the inclined strut *c* is connected to it by means of the bolt *b*. The end of the strut fits into a triangular hole cut in the block.

In Figs. 38 and 39, the purlins are not shown; these figures show methods of providing a greater bearing area for the upper ends of the vertical rods and inclined struts by means of plates and castings.

Fig. 40 shows the connection when the rod is inclined and the strut is vertical. In this figure, the purlin is shown vertical, and one side of it is in contact with the casting *b* that provides a bearing for the inclined rod *a*.

49. Peak Joint.—The joint at the top of a roof truss is usually called the **peak joint.** Figs. 41 to 45 show several peak joints. In Fig. 41, the rod *c* passes between the ends of the two rafters *a* and *b*; the plate *d* provides a bearing for the end of the rod. In Fig. 42, the plates *a* are bolted to the sides of the rafter sticks to assist in holding them in place. In Fig. 43, the upper ends of the rafter members are cut off square and bear on a casting *a*. The rods (two in this case) pass through flanges in the casting, and are held in place by the nuts *b*. The chair or

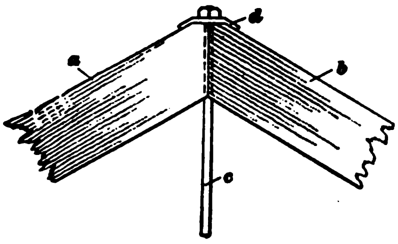


FIG. 41

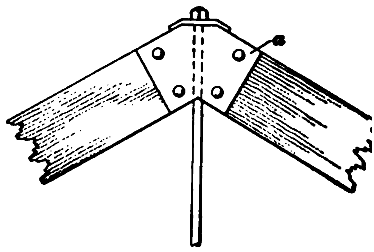


FIG. 42

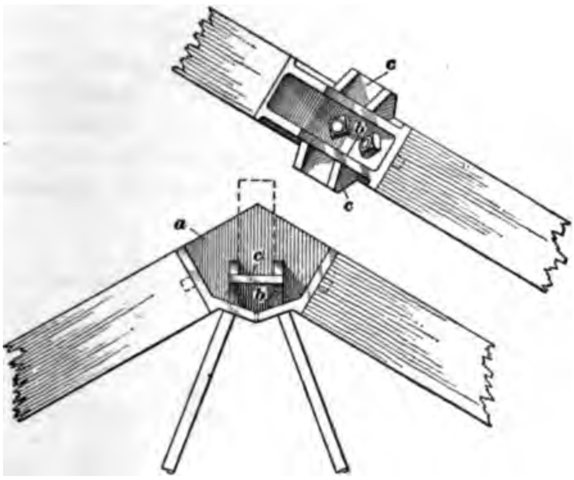


FIG. 43

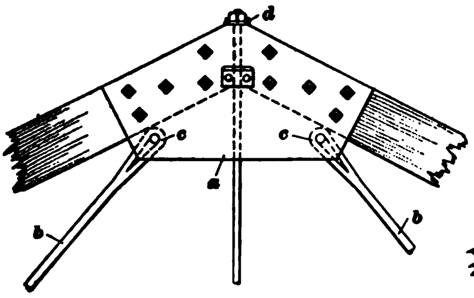


FIG. 44

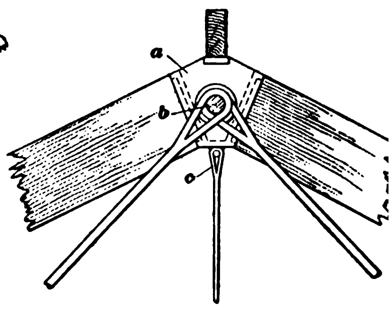


FIG. 45

shelf *c* is for the purpose of supporting the ends of the purlins. Fig. 44 shows another method of connecting the tension rods and compression members; in the form of connection here represented, gussets or connection plates *a* are bolted to the sides of the rafter sticks, and the inclined rods *b* are connected to them by means of pins *c*, which pass through the gussets. The vertical rod passes through the sticks and bears on the plate *d* in the same way as in Fig. 41. In the connection shown in Fig. 45, the rafter sticks bear against the casting *a*, and the inclined rods connect to the pin *b* that passes through the casting. The vertical rod is connected by a pin *c* to a projection on the casting at the bottom.

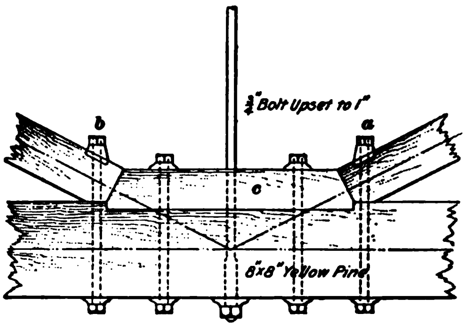


FIG. 46

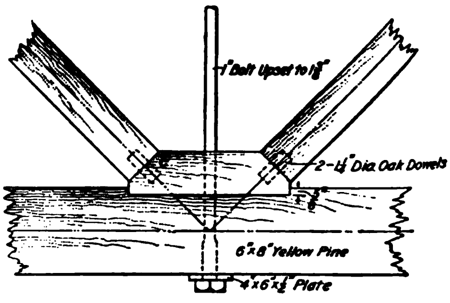


FIG. 47

50. Chord Joints.—Figs. 46 and 47 show the connections of a vertical rod, two inclined struts, and the chord. The blocks *c* are set into the top of the chord and beveled on the ends to give the struts a square bearing. The dowels shown in Fig. 47 are sometimes omitted.

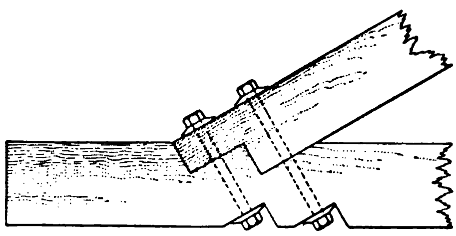


FIG. 48

51. Heel Joints.—The end joint of a roof truss, where the rafter and the chord connect, is sometimes called the **heel.** Fig. 48 shows the simplest form of heel joint. Fig. 49 shows a very good joint where the inclined member is composed of two sticks. Both the inclined sticks and the horizontal chord are cut away so as to form shoulders, and the ends of the sticks are bolted together.

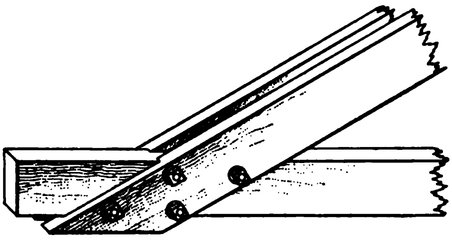


FIG. 49

Fig. 50 shows a form of heel joint that is used when the roof has a flat slope and it is not desired to cut too much into the end of the chord.

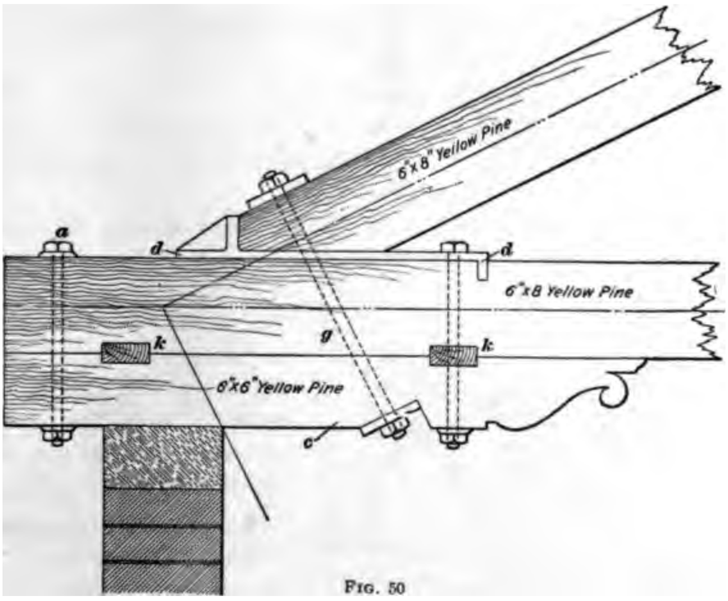


FIG. 50

The casting *dd* is fitted and bolted to the top of the chord, and provides a bearing for the inclined stick. A short piece of timber *c* is bolted and keyed to the bottom of the chord by the bolts *a* and the keys *k*, *k*. The bolt *g* is not assumed to

transmit any stress; it is simply for the purpose of holding the lower end of the inclined member in place.

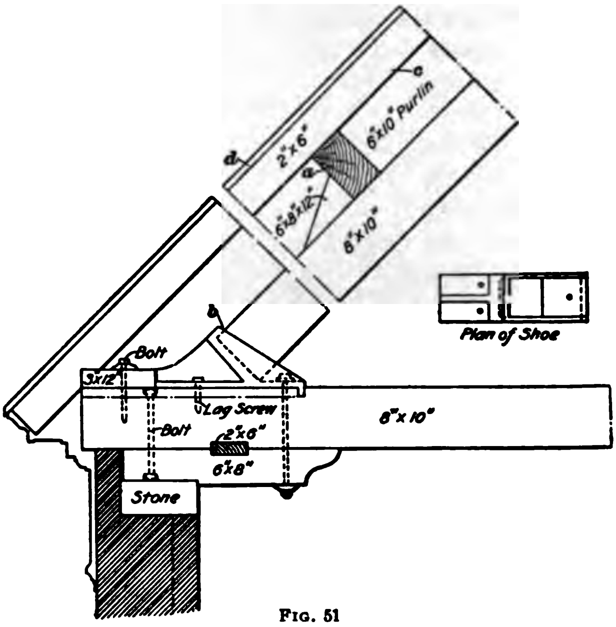


FIG. 51

The joint shown in Fig. 51 is somewhat similar to that shown in Fig. 50; but the method of providing a bearing for the rafter is different. In Fig. 51, the purlin *a* is shown on top of the rafter *b*; the common rafter *c* with the roof covering *d* is on top of the purlins.

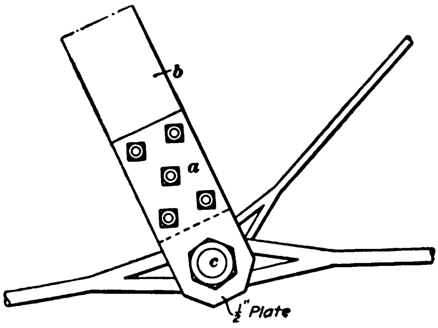


FIG. 52

52. Combination Chord Joint.—Fig. 52 shows a chord joint of a truss in which the compression members are wood and the tension members steel. Pin plates *a* are bolted on the sides of the stick *b*, and the ends of all the members are connected by the pin *c*.

STEEL TRUSSES

53. Rafter Joint.—Figs. 53 to 57 show rafter joints of steel roof trusses. In Fig. 53, a wooden purlin is shown in cross-section; it is held in place by the bracket *a*.

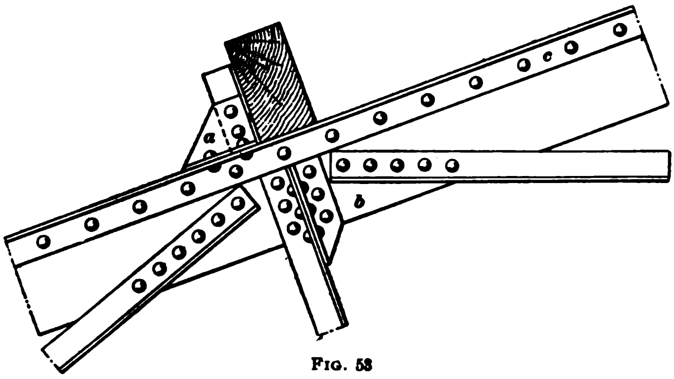
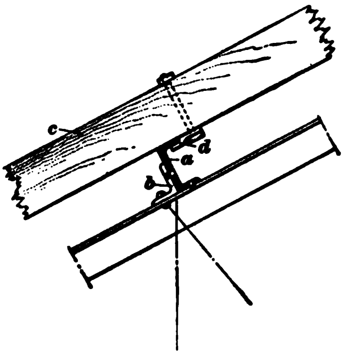


Fig. 53

The rafter in this case consists of a single web *b* and two top flange angles *c*. The web members are riveted directly to the web-plate, and there is no need of a gusset. Fig. 54 shows the method of connecting a steel purlin *a* to the rafter by means of the



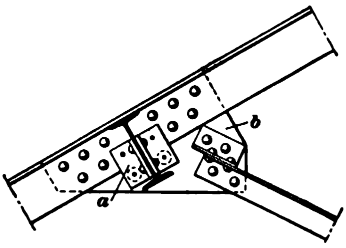


Fig. 54

Fig. 55

connection angle *b*. This figure also shows the wooden common rafter *c* on top of the purlin, held in place by the steel clip *d*. Fig. 55 shows another method of connecting a purlin *a* to the truss. In this case, a gusset *b* is used for the connection of the web members to the rafter. In Fig. 56, the purlin *a* is shown vertical, and the common rafter *b* is connected to it by means of a clip.

Fig. 57 is a rafter joint in a pin-connected truss. Both the rafter and the compression web member are composed of two channels. The rafter is spliced at this joint, being cut at the center of the pin. This splice is simply for the purpose of decreasing the length of the rafter pieces, so that they can be handled more easily. The compression web member is connected to the pin by means of pin plates.

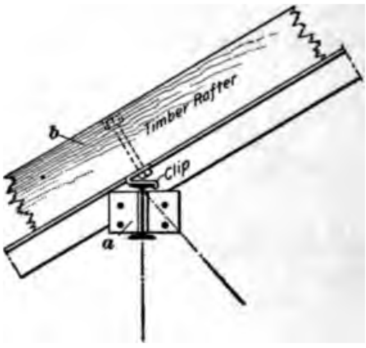


FIG. 56

54. Peak Joint.—Figs. 58 and 59 are peak joints of riveted trusses, and show how the members are connected

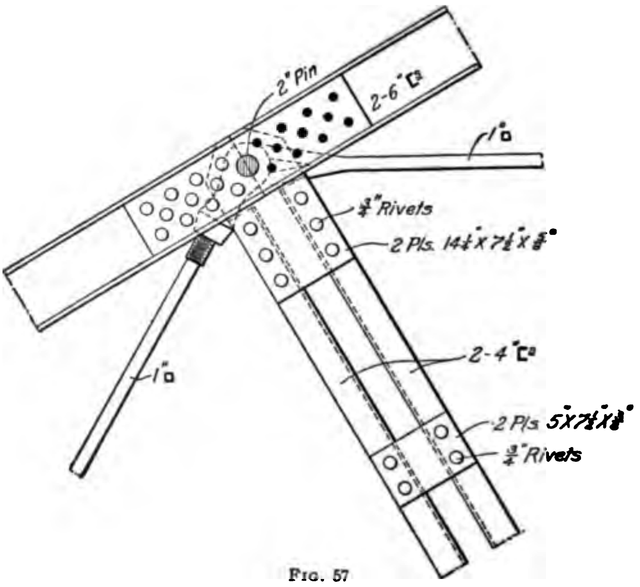


FIG. 57

at the top. In Fig. 58, the rafter members are each composed of two angles, and the vertical member is composed of two flat bars. The gusset *a* is riveted in the shop to one of the rafter members, and the other members are riveted to it in the field. The purlin *c* is composed of two channels, and is connected to the truss by means of the bent bars *d*. This joint also shows the way in which the common rafters *b* may be connected to the top of the purlin.

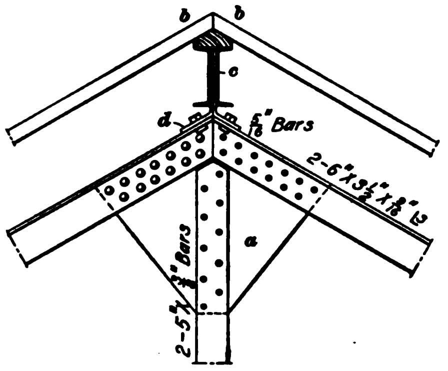


FIG. 58

Fig. 59 is the peak joint of a much larger truss. The vertical webs *a*, *a* that form part of the section of the rafter are spliced at *b*, *b* to the gusset *c*. The gusset is shown connected at the left end of the rafter and one of the web members by shop rivets, and to all the other members by means of field rivets. It is customary to rivet as great a portion of a roof truss together

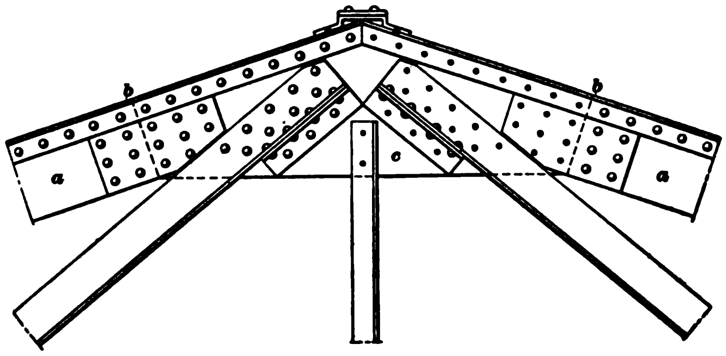


FIG. 59

in the shop as can be conveniently transported to the place where it is to be erected.

55. Heel Joint.—Fig. 60 shows a method of forming the heel joint of a riveted truss when the end rests on a pedestal or on the wall. The chord *a* is continued across the support, and the rafter *b* is connected to it by means of the gusset *c*. Fig. 61 shows a method of forming this joint

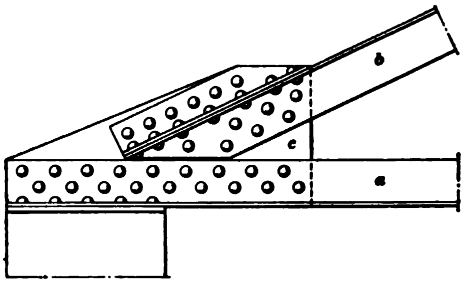


FIG. 60

when the end of the truss is supported on a column. The rafter *a* is continued across the top of the column, and the angles or other shapes *b* of which the column is composed

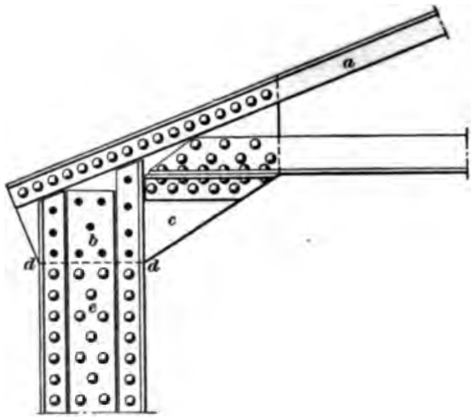


FIG. 61

are continued up to the under side of the rafter. The gusset *c* connects the end of the chord and that of the rafter to the top of the column. In the figure, the web of the column is stopped at *d d*, and the lower edge of the gusset is inserted between the angles at the top. The gusset and web-plate are spliced by the side plates *e*; when there are no side plates, additional plates are used for splicing.

56. Chord Joints.—Fig. 62 shows a method of forming a chord joint of a pin-connected roof truss. In this joint, all the tension members are placed inside the channels that

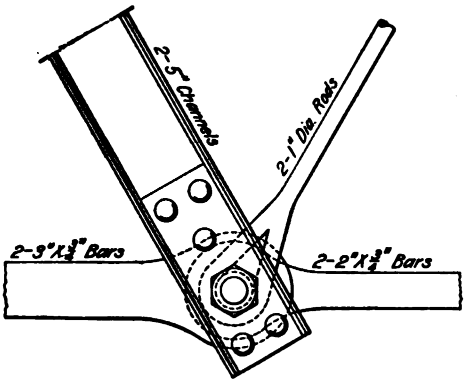


FIG. 62

form the compression member. Fig. 63 shows a chord joint of a riveted roof truss, the chord being spliced at the joint. In this figure, every member is composed of two angles;

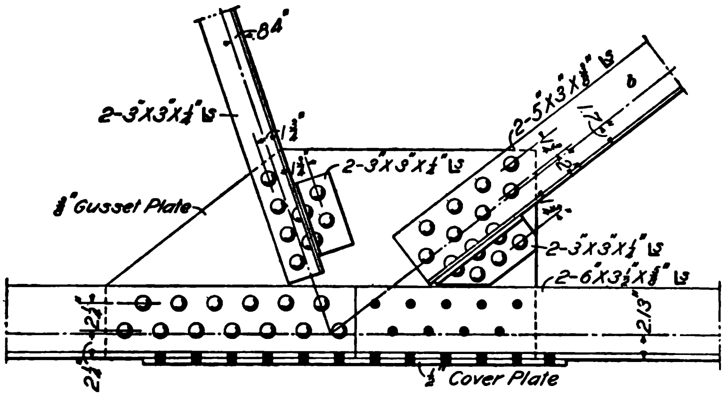


FIG. 63

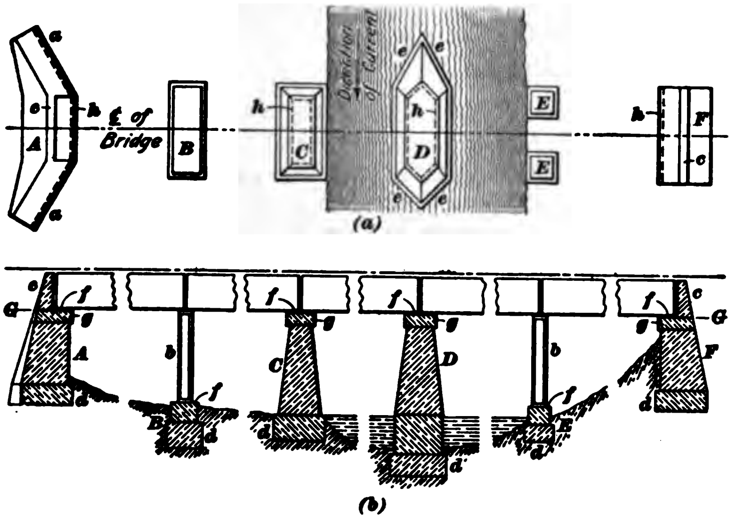
the ends are connected by a single gusset inserted between the angles. This gusset acts also as a splice plate for the chord.

BRIDGE PIERS AND ABUTMENTS

PRELIMINARIES

DEFINITIONS

1. Abutments and Piers.—The supports provided for the ends of a bridge are generally required to hold back the banks, thus acting both as supports for the bridge and as



(b)

Fig. 1

retaining walls; supports of this character are shown at *A* and *F*, Fig. 1, and are called **abutments**. When, as at intermediate points, such as *B*, *C*, *D*, and *E*, Fig. 1, there is no bank to restrain, the supports are called **piers**.

2. An abutment usually has lateral extensions, *a*, *a*, Fig. 1 (*a*), to hold back the earth in the sloping bank. These extensions are called **wings**, and the abutment is sometimes called a **wing abutment**, if it is desired to indicate the existence of wings. Nearly all abutments are wing abutments.

When the wings are omitted, as at *F*, Fig. 1 (*a*), and the filling material is allowed to run around in front of the abutment, the latter is called a **pier abutment**, or **abutment pier**.

3. Piers may be carried up to such a height as to give a direct support to the bridge, as at *C* and *D*, Fig. 1 (*b*), or they may be carried only to such a height as to protect the structure from the destructive influences below or near the surface of the earth, as at *B* and *E*, a trestle *b*, *b* being introduced between the top of the pier and the bottom of the bridge. These low piers are frequently divided into two separate supports, as shown at *E*, Fig. 1 (*a*), called **pedestals**, or **pedestal piers**.

4. Superstructure and Substructure.—The bridge proper, which rests on the piers and abutments, above the level *G G*, Fig. 1 (*b*), is called the **superstructure**. All supporting work below the level *G G*, including trestles or columns, as at *B* and *E*, and all the masonry, is called the **substructure**.

5. Bridge Seat.—The top surface *f*, *f*, Fig. 1 (*b*), of an abutment or pier to which the weight of the bridge is directly transmitted is called the **bridge seat**. The course *g*, *g* of masonry directly under the bridge seat is called the **bridge-seat course, coping course**, or **top course**; it is sometimes called simply the bridge seat or coping. This course usually projects a few inches beyond the outer edge of the masonry directly under it. This projection is called the **overhang** of the bridge seat; it improves the appearance of the pier, and protects the masonry beneath it from the weather. The outer edge *h*, *h*, Fig. 1 (*a*), of the masonry immediately under the top course is called the **neat line**.

6. Pedestal Blocks.—In cases where stringers or floor-beams rest on the masonry at elevations different from the elevations of the base of the trusses or girders, separate top courses are sometimes laid; but more frequently the top course is finished level at the proper elevation for the trusses or girders, and is called the **main bridge seat.** The proper elevation is then secured for the stringers or floor-beams by placing stone blocks, called **pedestal blocks,** on top of the main bridge seat. In many cases, the desired elevation is secured by means of steel castings, called **pedestals.**

7. Parapet.—In order to keep the filling from running over on the bridge seat, a back wall *c, c*, Fig. 1, commonly called a **parapet,** is usually built on top of the bridge seat.

8. Footings.—The lower part *d, d*, Fig. 1 (*b*), of a pier or abutment is called the **footing.** It is usually made to project somewhat beyond the masonry of the main body above it. Footings serve the purpose of securing greater bearing area on the soil and of giving greater stability to the structure.

9. Cutwaters.—When piers are placed in running streams, as at *D*, Fig. 1, they are usually placed with their longest dimension parallel to the direction of the current, and the ends are made pointed instead of flat, in order to reduce the obstruction to the flow of water. The up-stream end is generally made sharper than the down-stream end. The sharp ends *e, e*, Fig. 1 (*a*), are called **nosings** or **cutwaters.** In cold climates, where floating ice is common, cutwaters are sometimes called **ice breakers,** and are frequently protected by means of a bent steel plate fastened to the masonry. This protection should always be provided where large masses of ice are likely to float down the stream.

LOCATION AND ESTIMATES

10. Direction of Crossing.—The direction of a bridge, especially of a railroad bridge, should, if practicable, be so selected that the crossing will be at right angles, or nearly so, to the direction of the stream, road, or depression to be crossed. This is desirable both because such location reduces the length, and accordingly the cost, of the structure, and also because a skew end is objectionable from a purely structural point of view. The problem of making the crossing square or nearly so is not always considered the duty of the bridge engineer; but the locating engineer should endeavor so to arrange the alinement as to avoid the necessity of skew bridges.

11. In the case of a railroad bridge, unless it is made unusually heavy and stiff, the rigidity of the support at the abutment and the elasticity of the unsupported part of the bridge at the opposite side cause an undesirable swing to passing trains. The cause of this swing may be understood

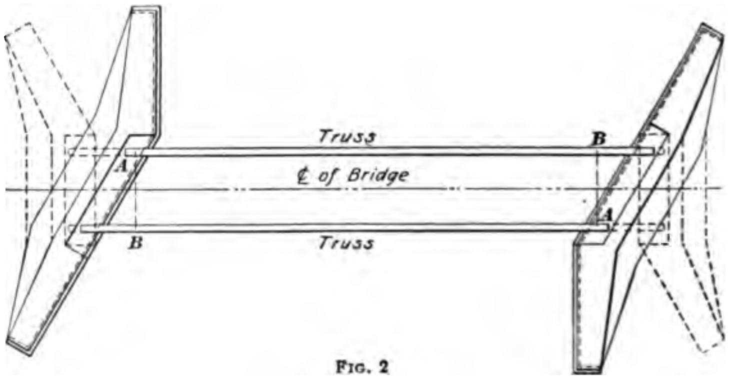


FIG. 2

by referring to Fig. 2, which shows a skew crossing. On account of the rigidity of the abutments, the deflection at such a point as A is practically negligible, while at such a point as B, directly opposite A, the truss deflects an appreciable amount. The result is that the floor, and consequently the train, is inclined sidewise from A to B. This objectionable condition may sometimes be avoided by squaring up the ends, as shown by dotted lines in Fig. 2, but this still further lengthens and increases the cost of the crossing; and, if more than one span is required, the intermediate piers must still be generally left on a skew, so as to present as little obstruction as possible to the current or traffic beneath.

12. Location of Piers.—With the location of the crossing determined, the problem is not necessarily wholly solved. If the bridge is over a railroad, a highway, or a navigable stream, the position and length of one or more spans may be fixed by the requirements of the traffic beneath; for the way previously existing is entitled to the preference and controls to a great extent the locations and dimensions of the piers. The first thing to do after the direction and location of the crossing have been decided is to determine the location of the abutments, and to ascertain to what extent the traffic underneath will affect the selection of the number and length of spans. The piers should be located so as to offer the least possible amount of obstruction. This condition frequently controls the location of every pier. Another element that must be taken into account in determining the positions and number of piers is the cost. It should be borne in mind that a reduction in the number of piers requires an increase in the lengths of the spans, and that the increased cost of the spans may upset the saving resulting from a reduction in the number of piers. If the crossing is a low one, and a good foundation can be obtained at a moderate depth, short spans are often more economical than long spans.

13. Economical Arrangement.—For preliminary estimates, use may be made of the empirical rule that the economic arrangement is approximately that in which the cost of the substructure equals the cost of the superstructure (the trestles, if any, being included in the substructure). Also, in this preliminary study, the cost of the main trusses or girders may be considered as varying directly as the square of the span, and the cost of the floor directly as the length of the span, *unless the change of length of span is sufficient to change the character of the structure.* The cost of the substructure for any one support depends on the length of span, the height of the structure, and the depth of the foundation. By roughly designing piers for both the longest and the shortest span to be considered, the cost of each pier can be approximately determined.

If the crossing is short, the most economical number of spans and length of span may generally be determined immediately by finding a span such that the total cost of substructure divided by the length of the bridge will be approximately equal to the total cost of superstructure divided by the length of the bridge. But if the crossing is long, a more detailed estimate of cost is usually necessary.

14. Approximate Costs.—For the purpose of comparison, the cost of masonry for the substructure and of the steelwork for the superstructure may be assumed. In this Section, the cost of the masonry will be taken as $10 per cubic yard, and that of the excavation for foundations as 50 cents per cubic yard, including the disposal of the material excavated. For rough estimates, the cost per linear foot of the steelwork of the bridge spans and the necessary dimensions of bridge seat for different spans may be taken approximately as follows:

Span Feet	Cost per Linear Foot Dollars	Required Size of Bridge Seat Feet
50	50	4 × 10
100	80	5 × 15
150	110	6 × 20
200	140	7 × 25
250	170	8 × 30

ILLUSTRATIVE EXAMPLE

15. Data.—As an illustration, let it be assumed that a bridge 1,000 feet long for a single-track railroad is to be built across a valley 80 feet deep, the masonry to be carried to a depth of 5 feet below the bottom of the valley and up to such height as to give direct support to the bridge span; and the distance between the tops of piers and the base of rails to be one-fifth of the length of the spans.

16. Possible Spans.—It will be readily seen that a length of 1,000 feet may be divided into two spans of 500 feet each, or three spans of 333 feet 4 inches, or four spans of 250 feet, or any greater number of spans of correspondingly shorter length. But it is also obvious that with the longer spans the cost of the superstructure will be much greater than if short spans are used.

17. Cost of Superstructure and Substructure.—For rough estimates of this kind, it is sufficiently close to assume the top of the pier to have the dimensions given for the bridge seat, and the four sides to have a batter of 1 inch per foot down to the level of the ground; whence, with an offset of 1 foot on each side and one at each end, the same dimensions are maintained down to the subfoundation, 5 feet below the ground surface. With these assumptions, the approximate quantities and cost for the substructure are found to be as given in the following table:

Span Feet	Masonry		Excavation		Total Cost per Pier	Total Cost of Substructure per Foot of Span	Total Cost of Superstructure per Foot of Span
	Cubic Yards	Cost	Cubic Yards	Cost			
50	510	$5,100	78	$39	$5,139	$102.80	$ 50
75	533	5,330	82	41	5,371	71.60	65
100	548	5,480	85	43	5,523	55.20	80
150	558	5,580	92	46	5,626	37.50	110
200	537	5,370	98	49	5,419	27.10	140
250	488	4,880	103	51	4,931	19.70	170

18. In finding the quantities given in the second column of the table, it is convenient to draw a diagram similar to that shown in Fig. 3, which is a pier for a 100-foot span.

The distance from the rail to the ground is given as 80 feet; that from the rail to the bridge seat is one-fifth the span, or, in this case, 20 feet. This makes the pier 60 feet high from the bridge seat to the ground surface. The required size of bridge seat for this span is given in Art. 14 as 5 ft. × 15 ft.; with a batter of 1 inch per foot, the cross-section of the pier increases 1 foot in each direction for every 6 feet of height, which makes 15 ft. × 25 ft. at the ground surface. The prismoidal formula, given in *Geometry*, Part 2, is best adapted to the computation of the volume. The cross-section of the pier half way between the bridge seat and the ground surface is 10 ft. × 20 ft., and the volume of this part of the pier is

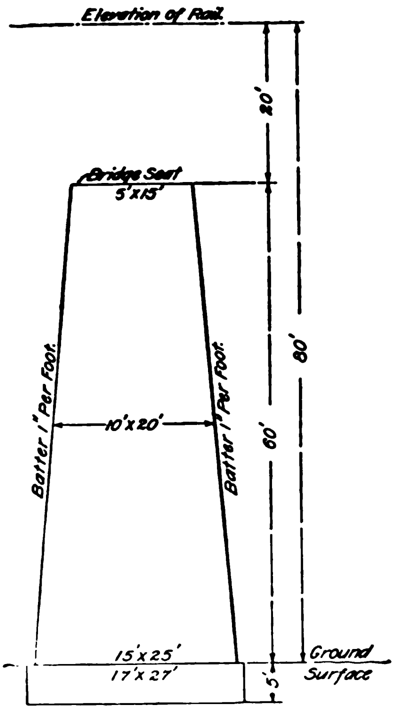


FIG. 3

found to be $\frac{60}{6 \times 27}[(5 \times 15) + (4 \times 10 \times 20) + (15 \times 25)]$ $= 463$ cubic yards. The foundation is made 1 foot larger all around than the bottom of the pier at the ground surface. This makes the base 17 ft. × 27 ft. in cross-section. If the depth is taken as 5 feet, the volume of this part of the pier is $\frac{5 \times 17 \times 27}{27} = 85$ cubic yards. The total volume of masonry is, then, $463 + 85 = 548$ cubic yards. The amount of excavation, given in the fourth column of the table, is assumed to be the same as the volume of the foundation, or, in this case, 85 cubic yards.

The total cost per pier, given in the sixth column, bears no relation to the length of span, but increases and then decreases as the span increases. This is because, in this example, the height of pier is less for the longer spans, since the bridge seat is taken at a distance of one-fifth the span below the rail. On the other hand, the cost of substructure, *per linear foot of bridge*, decreases rapidly as the span increases, so that if this were the only consideration, long spans would be desirable. This decrease is offset, however, by the increase, *per linear foot of bridge*, of the cost of the superstructure.

19. Final Calculation.—From an inspection of the table in Art. **17** it is seen that, on the basis assumed, the cost of one unit of substructure will equal the cost of one unit of superstructure when the length of span is about 80 feet, and the cost per foot of substructure and superstructure will be about $68. This condition would call for twelve or thirteen spans, a number that is approximately right; but, on account of the magnitude of the work, it is advisable to carry the computations further with about this number and length of spans, using the actual profile of the ground at the crossing and considering in detail the depth of each pier, including the foundation. This is necessary because the piers near the ends of the bridge will probably not be so high as those near the center, where the valley is the lowest. The first rough calculation indicates about what number of spans should be considered; in the second, more detailed calculation.

20. Plans.—Unlike some other classes of structures, of which a great many may be built substantially alike and from the same plans, it is generally necessary to make a special design for every pier or abutment; for the dimensions and shape of these structures depend largely on greatly varying local conditions. It is customary in all cases to give the general dimensions, such as height, width, length, etc. In many cases, the sizes of stones to be used are marked on the plans, but it is considered better practice to allow the contractor to take care of this matter. It is sufficient in most cases to specify the limiting sizes that will be permitted.

PIER DESIGN

PRACTICAL CONSIDERATIONS

21. Bridge Seat.—In designing a bridge pier, the first matter to be considered is the area and plan of its top, so as to provide proper support for the superstructure. If the bridge is so designed that all the bearings on the masonry are at the same elevation, the top of the pier is made level throughout its whole area.

The width of the top of a pier can seldom be made less than 4 feet, and can be made as narrow as this only when the adjacent spans are very short. This width is necessary on account of the fact that the girders or trusses that form two adjacent spans are in the same line, and sufficient width must be provided to permit each to have sufficient bearing surface. The nearest edge of the bearing plates should not be allowed to come closer than about 3 inches to the neat line of the masonry under the bridge seat, and the edges at the ends of the adjacent spans should be from 6 to 12 inches apart. The bearing plates for the shortest spans that usually rest on piers can seldom be made less than 18 inches in length, so it can be readily seen that a width of 4 feet is as small as can be used. Allowing for an overhang of 3 inches on each side makes the bridge-seat course 4 feet 6 inches in width. When the bridge is on a skew, it is necessary to make the bridge seat somewhat wider, in order to keep the bearing plates from coming too close to the edges. Fig. 4 shows the bridge seat of a pier and the ends of two adjacent trusses.

22. In the length of a pier, a more liberal allowance is generally made than in the width, the ends of the pier being carried from once to twice the width beyond the centers

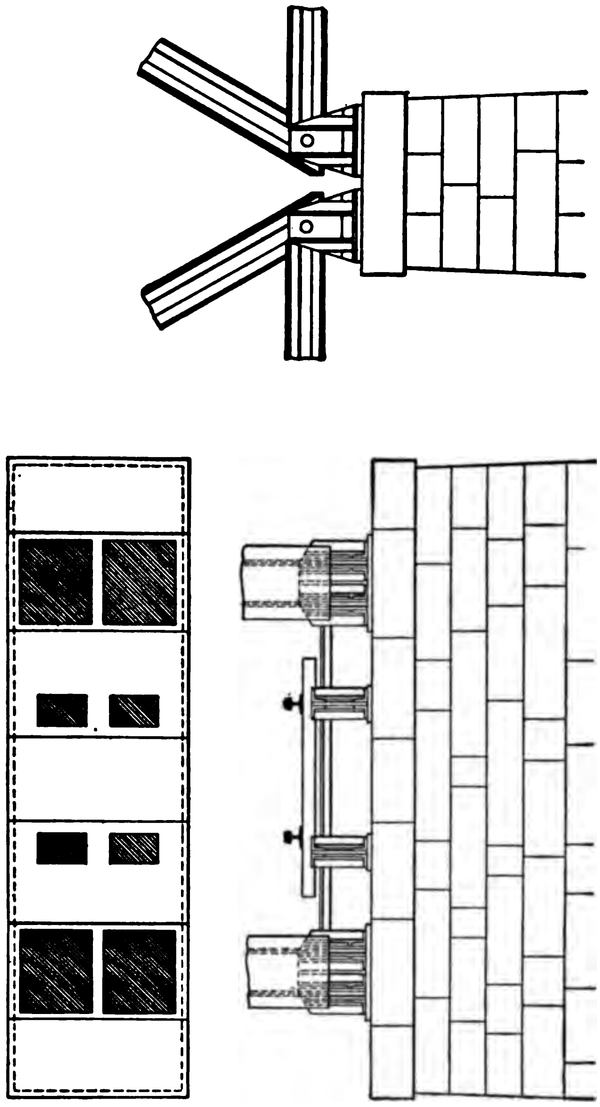


FIG. 4

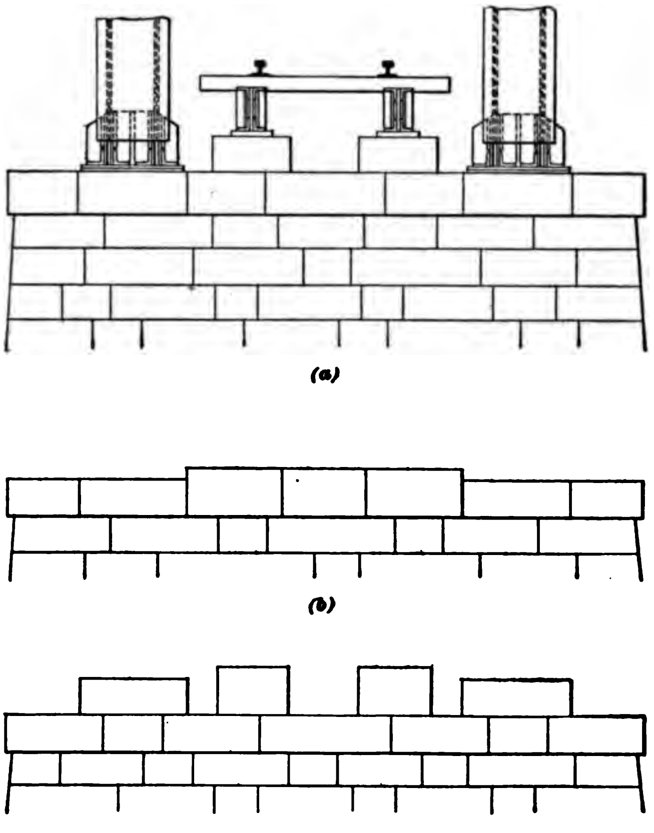
of the outside trusses or girders, unless special conditions render it necessary to adopt a smaller length. The extra length at the ends is considered necessary because it is not advisable to have an edge of the masonry close to two sides of a bearing plate; otherwise, a corner of the masonry may break off and allow the end of the bridge to fall. The increase is made in the length rather than in the width, because the former adds less to the total volume and cost of the pier, and also because it causes less obstruction under the bridge.

23. Pedestal Blocks.—When the design of the bridge is such as to require supports at different elevations for different parts of the superstructure, the top of the bridge-seat course is generally placed at the elevation required for the lowest supports, and the higher elevations are obtained by placing stones or castings on top of the bridge seat, or by using stones of different thicknesses in the bridge-seat course. Fig. 5 shows three ways of accomplishing this. In (*a*) and (*c*) stones of different thicknesses are placed on top of the bridge seat, while in (*b*) different thicknesses are used in the bridge-seat course.

24. Batter.—Sufficient batter for the sides and ends of the pier should always be provided for in the design, whether or not the theoretical conditions require the base to be greater than the top. The batter usually starts at the neat line under the bridge-seat course and continues unbroken down to the top of the footing course. In ordinary practice, a batter of $\frac{1}{2}$ to $1\frac{1}{2}$ inches to the foot is usually considered sufficient. When the horizontal cross-section of the pier is rectangular, the ends are given the same batter as the sides. When there are cutwaters, different batters are used.

25. There are two reasons for giving the pier a batter on all sides: first, it is usually required in order to give the proper size of base; second, the appearance is greatly improved by the batter. If the sides were built vertical, the pier would seem to the eye to be narrower and shorter at the bottom than at the top.

26. Footings.—The base or footing of a pier should always be carried to such a depth that it will be safe from the dangers of frost and other surface disturbances, and should be well protected from the danger of undermining, if run-



(a)

(b)

(c)

FIG. 5

ning water is near. The footing must be of sufficient size to properly distribute the load without exceeding the safe-bearing power of the subfoundation. When only practical conditions govern the size of the pier, the batter is usually continued unbroken to the surface of the ground. The sides and ends are then stepped out or offset 6 inches or 1 foot to form the footing, and the sides are continued vertical downwards to the subfoundation.

27. Height of Pier.—It is very important that the top of a pier be brought to the proper height. For bridges over city streets, the height is usually controlled by the required amount of headroom beneath. When a pier is to be placed in running water, special care must be taken to study all the conditions. The highest point to which the water has ever reached must be ascertained, and the top of the pier placed well above it. In addition, it must be borne in mind that the pier will be an obstruction in the stream; on this account, ice or drift may collect to a height of several feet, causing the water to rise higher than it did before. The steelwork of the bridge should be placed so high that it will never be in danger of being knocked off the pier by blows from the ice or drift. Even where there is no current, the top of the pier should be placed so high that the steelwork of the bridge will not be subject to corrosion on account of contact with the water.

28. Piers in Streets and Highways.—Usually, when a bridge crosses a body of water, the piers are carried up high enough for the bridge to rest directly on them. In many other cases, however, the piers are carried only a short distance above the surface of the ground, and the bridge is supported by columns that rest on the piers. This is the case in many railroad bridges over city streets and highways. The abolition of grade crossings of highways with railroads calls for a great number of bridges, and many of these have low piers. When the street or highway is wide, it is found more economical to use several short spans than one long span. A pier is placed at the center of the street, or at each curb line, or at both the center and the curb lines. Then, since the piers would take up so much of the width of the street, only the footings are built, and the columns, which occupy much less width, are placed directly on the footings. In this case, the base of each column should be covered with concrete to such a width that wheels of passing vehicles cannot hit the columns, and also to protect the bases of the columns from the corrosion that might be caused by the collection of dirt and moisture. The low piers that support the columns are for the purpose of spreading the loads evenly over the proper area of subfoundation.

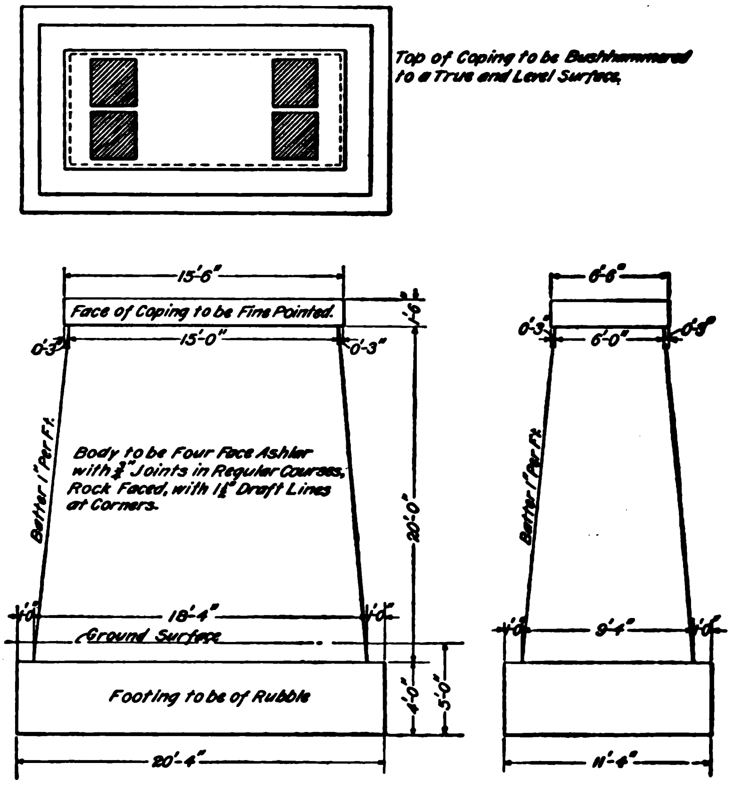


FIG. 6

It should be kept in mind that piers built in city streets in which there is much traffic will be very conspicuous if carried up to the bridge, and the designer should attempt to make them look as artistic as practicable. A very simple way to improve the appearance is to point or round the ends. Fig. 6 gives dimensions for a pier reaching up to the bridge for a single-track railroad bridge suitable for location in a street. Fig. 7 gives the dimensions for a low pier for the support of columns or trestle bents.

29. Piers Near Railroad Tracks.—In the case of bridges over railroad tracks, it is often necessary or advisable to provide supports near the tracks below. Many engineers provide these supports by means of steel trestles resting on low masonry piers similar to that shown in Fig. 7. When columns are used, it is not advisable to have them extend down to the track level, for a derailed train might run into them and cause the bridge to fall. In order

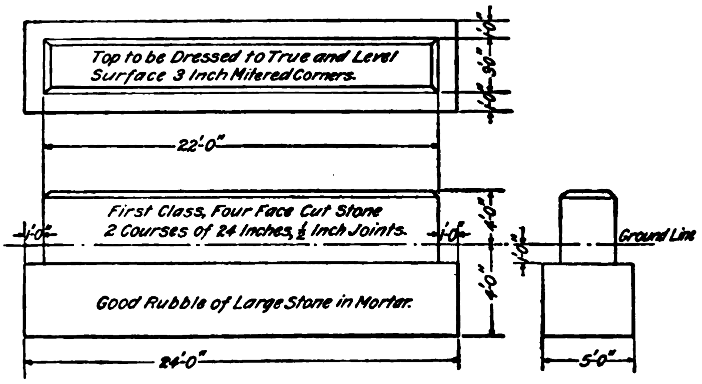
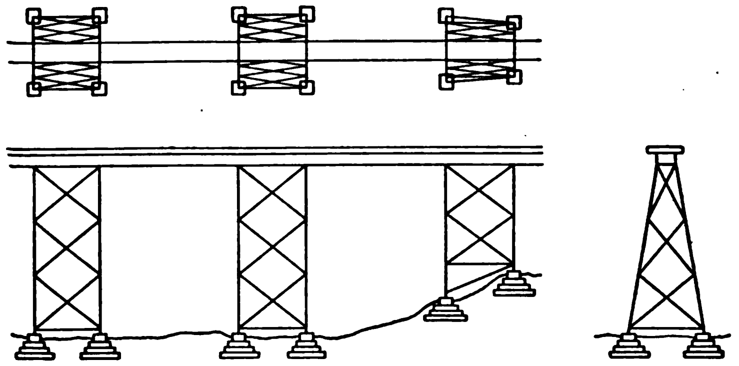


FIG. 7

to avoid all possibility of such an accident, it is advisable to have the masonry extend up to about 10 feet above the track, so that in case of a derailment the masonry pier can resist the shock. Of course, it is a serious matter for a derailed train to strike a masonry pier, but it would be still more serious for it to knock the supports out from under a bridge, causing the latter to fall on and crush the train. For similar reasons, the masonry work of piers in navigable streams should be made of sufficient height to protect the bridge.

30. Piers for Trestles.—For high trestles over land, long spans are sometimes used, and then the piers are continued up to the bottom of the bridge. In the majority of cases, however, shorter spans are used, and it is found to be more economical to have the pier extend only a short distance above the surface of the ground, and to support the bridge by means of a steel tower resting on the pier. For low trestles, the pier is usually made continuous across the track, so that it will support two or more of the columns that compose the trestle bent. The pier shown in Fig. 7 is suitable for such conditions, the length depending on the length of the bent.

31. Pedestals for High Trestles.—High trestles are generally built with short tower spans of from 30 to 50



(a) (b)

FIG. 8

feet, alternating with longer spans, as shown in Fig. 8. The trestle bents that form the towers are placed in vertical planes, as shown at (*a*), but the outside columns in the bent (there are usually but two) are inclined outwards from the track, with a batter of $1\frac{1}{2}$ to 3 inches per foot, as shown at (*b*). For high trestles, this calls for supports at such distances apart that it is generally undesirable to make a continuous pier for each bent. A separate masonry support, usually called a **pedestal**, is then provided for each column. These pedestals should extend far enough into the ground to be able to resist the action of frost and other destructive surface agents, and their bases should be made large enough to eliminate any possibility of settling. The horizontal distance between the pedestals at the base of a tower is generally much less than the distance from the top of the pedestals to the track. On this account, whenever one pedestal settles, the track above is thrown out of line a greater amount than the amount of the settlement, and one rail sinks below the other. If the tower above is very strongly braced, both horizontally and diagonally, as is customary in the best practice, one of the pedestals can settle without causing the column above it to settle, this column being held up by the bracing while there is no load on the bridge. As soon as sufficient load comes on the column, however, the latter will be forced down on top of the pedestal, and, in the case of a railroad bridge, the train may be thrown off the trestle. It is thus seen that it is of the utmost importance to provide a sufficient area of base for pedestals, especially for high trestles, and also to provide proper foundations. The possible result of settlement here is much greater than in the case of continuous piers under low bridges. In addition, if any settlement should occur, it will be a very difficult matter to rectify. The outlines of several pedestals showing the customary method of spreading the courses over a greater base are shown in Fig. 8.

32. As a rule, trestle bents are so designed that, although the columns are inclined outwards, the load on each pier will be vertical, except in case of wind. The outward thrust is usually resisted by the horizontal bracing of the bent. For this reason, the center lines of the pedestals are vertical. For pedestals on a side hill, allowance for the tendency of the pedestals to overturn or slide downwards must be considered and the foundations placed lower. As a rule, this tendency can be overcome by making the pedestals several feet deeper than those on level ground. On a side hill, care must also be taken to make the tops of the piers high enough to keep clear of the earth that may roll down, and to keep the steel or wooden posts out of the dirt at all times; otherwise, it is desirable to keep the tops of the pedestals as low as possible.

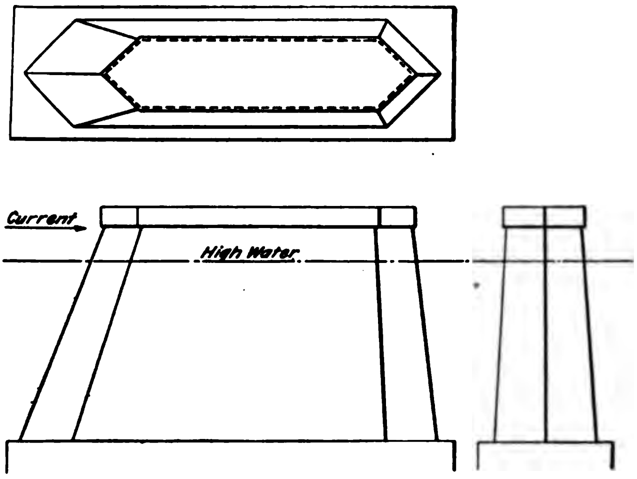


FIG. 9

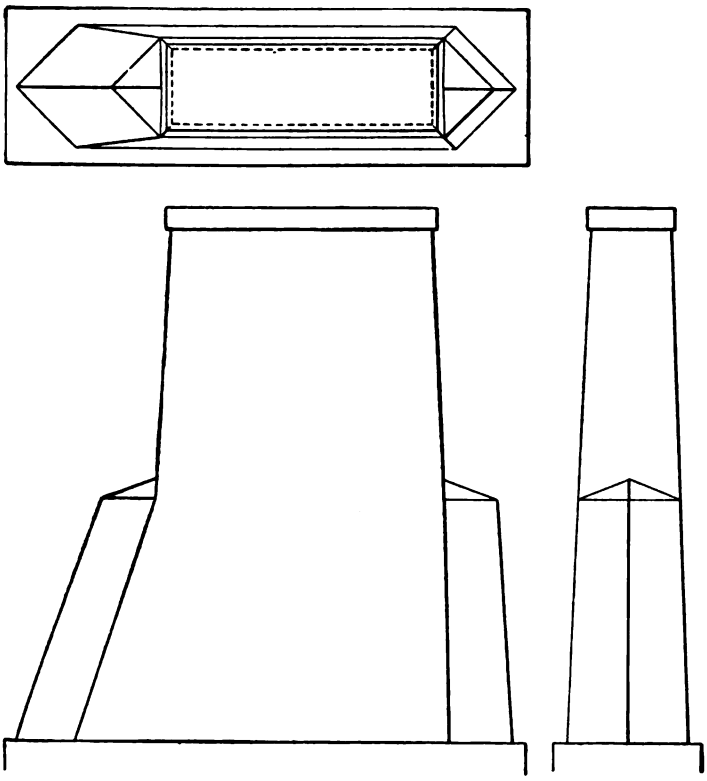


FIG. 10

33. Piers for Running Streams.—Outlines of piers suitable for running stream are shown in Figs. 9, 10, and 11 for moderate heights, and in Fig. 12 for a long span bridge over a deep river. It will be noticed that in Fig. 12 the batter of the sides of the pier is smaller than usual. The batter is frequently decreased for a large pier; for, at best, it is a serious obstruction to the flow of the water, and it is desirable to reduce the amount of obstruction as much as possible. A small batter can be used only for a pier under a long span.

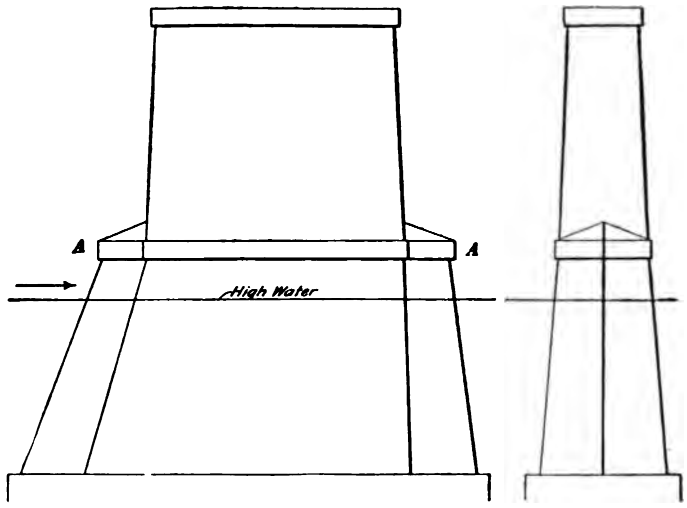
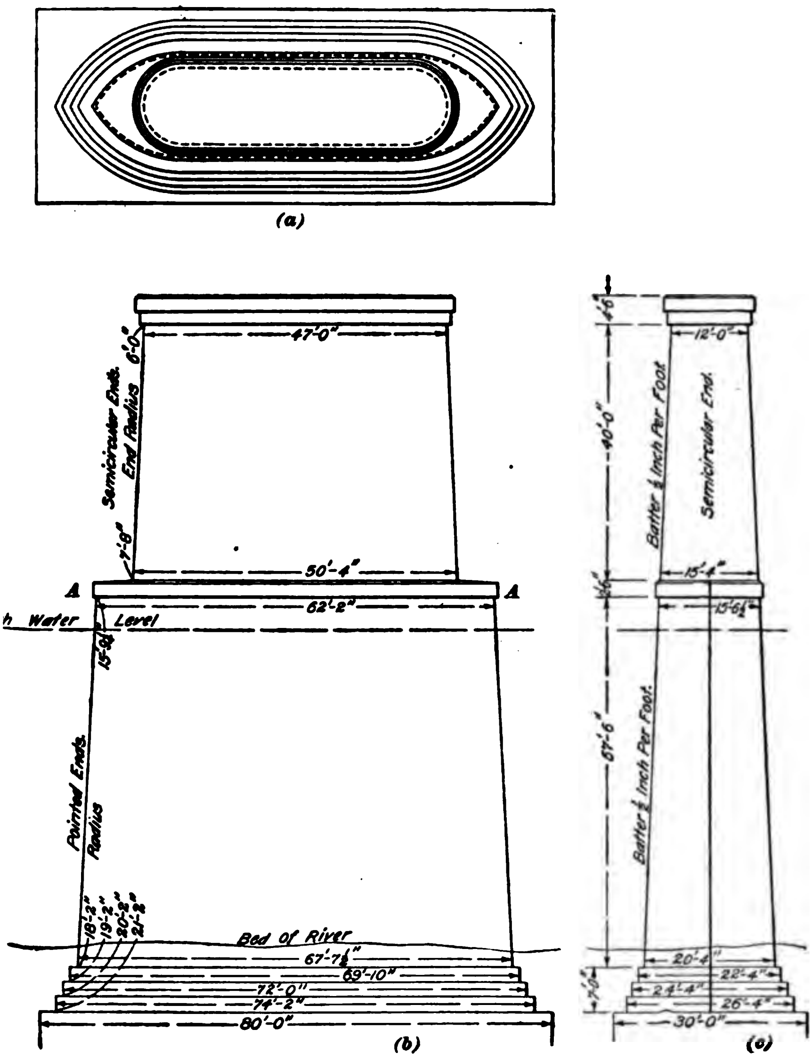


FIG. 11

In such a case, it is necessary to provide a large bridge seat, the width of which is often sufficient for the foundation. A small batter is provided, however, for the sake of appearance. The stability of such a high pier is a subject for careful study, as will be described presently. It may be remarked here, however, that in a pier under a long span, the dead weight is so great that the pier is usually safe against any overturning forces to which it is likely to be subjected.

34. Nosings or Cutwaters.—The form of nosing usually adopted for a pier in running water is the result of a compromise between the form that would reduce the eddies



(a)
47'-0"
6'-0"
Semicircular Ends.
End Radius
7'-8"
50'-4"
A
A
62'-2"
h Water Level
15'-9½"
Pointed Ends.
Radius
18'-2"
19'-2"
20'-2"
21'-2"
Bed of River
67'-7½"
69'-10"
72'-0"
74'-2"
80'-0"
(b)
4'-6"
12'-0"
40'-0"
Batter ½ Inch Per Foot
Semicircular End.
15'-4"
15'-6½"
67'-6"
Batter ½ Inch Per Foot.
20'-4"
22'-4"
24'-4"
26'-4"
7'-0"
30'-0"
(c)

FIG. 12

and whirlpools to a minimum, and the modifications of that form which permanency and economy demand. To reduce the obstruction to the current to a minimum, the ends of the pier should be provided with nosings pointed gently by means of a reversed curve, as shown in Fig. 13. This form, however, would involve expensive stone cutting or concrete forms, and is seldom used. A simpler form, which is almost as efficient in turning aside the water, consists in continuing the long sides by curved surfaces until they meet in a line, as shown in outline in Fig. 14. This form is the best for large piers, from the standpoint of both durability and expense, and is shown on the lower part of the pier in Fig. 12. In the common form for ordinary piers, the end is pointed by two plane surfaces coming together, as shown in outline in Fig. 15, and also in Figs. 9 and 10. The planes forming the nosing should make angles of from 30° to 60° with the center line of the pier. An angle of 45° with the center line, making the point 90°, as shown in Fig. 15, is frequently used.

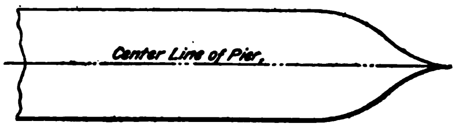


FIG. 13

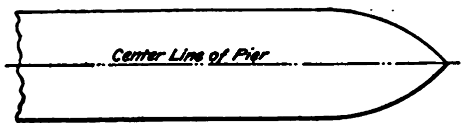


FIG. 14

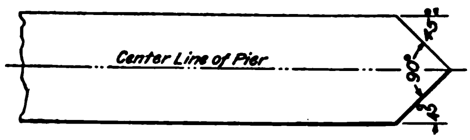


FIG. 15

35. At the down-stream end, the two planes that come together are given the same batter as the sides. In cold regions, where much ice may be expected, the nosing at the up-stream end of the pier is given a greater batter, so that the slope of the nose will incline at the rate of 3 to 6 inches per foot. The sharp edge is frequently shod with iron, which acts as an ice breaker. The momentum of the moving ice and the pressure from the current of water force the cakes of ice to slide up this smooth edge; when high enough out of the water, the ice breaks of its own weight, or by the blows of other ice forced against it.

The increased cost of curved ends on large piers is very small, and since the obstruction to the water is large, it is customary in practice to curve the ends, as previously shown in Fig. 12. If the ends were given the theoretically proper form shown in Fig. 13, the sharp point would soon be worn away. Semicircular ends are sometimes used; but, if there is much current, bad eddies and whirlpools result, and there is danger of the water being so agitated as to undermine the pier.

When the pier does not extend more than 10 or 15 feet above the high-water line, it is customary to have the cutwater extend up to the bottom of the bridge-seat course, as shown in Fig. 9. In very high piers, in which this procedure would involve much needless expense, the nosings are usually carried but a few feet above high-water mark, and the remainder of the pier is made with square ends, as shown in Figs. 10 and 11. When it is desired to give high piers a better appearance, the ends of the upper part may be made semicircular, as shown in Fig. 12.

36. Belt Course.—In case the upper part of the pier is arranged differently from the lower, they are separated from each other by means of a **belt course**, similar to a bridge-seat course. The belt course is indicated by the letters *A*, *A* in Figs. 11 and 12.

THEORETICAL CONDITIONS

37. After the approximate dimensions of a pier have been decided according to the foregoing practical considerations, it is necessary to investigate the stability of the pier and to make all the calculations necessary to determine whether the actual stresses will anywhere exceed the allowable stresses.

CAUSES OF FAILURE

38. The stability of a bridge pier may be destroyed by any one of the following causes:

1. *Overturning* at any section or on the subfoundation on account of: (*a*) pressure of the wind on the structure and on the loads carried by the structure; (*b*) the current in the stream, and the pressure and blows from ice and other floating objects; (*c*) the centrifugal force of trains; and (*d*) the longitudinal thrust due either to the friction of the wheels on the rails when brakes are set, or to the traction of the engine.
2. *Sliding* at any cross-section or on the subfoundation.
3. *Crushing*, either at the bridge seat, at any lower section, or at the base.
4. *Failure of the foundation or subfoundation*, either from insufficient strength, from the upheaval by frost, or from other disturbing agents.
5. *Breaking apart or collapsing*, on account of poor mortar, poor bond between the stones, or poor concrete.

39. Overturning.—In a direction parallel to the axis of the bridge, the forces tending to overturn the pier are the pressure of the wind, the longitudinal thrust from trains in railroad bridges, and the packing of ice or drift between the piers or between a pier and the bank of a stream. In case the earth extends to a greater height on one side of a pier than on the other, the unbalanced pressure must not be overlooked. These forces act to overturn the pier in the direction of its smallest dimension and least resistance.

In a direction at right angles to the axis of bridge, there may be a pressure of the wind not only on the pier but also on the bridge and on the train or other load, and the force of the current may be added to the pressure of the ice and drift; there may also be centrifugal force if the track is on a curve. The centrifugal force can act only toward the outside of the curve, and the force of the current can act only in the direction of the flow; but the other forces may be exerted in either direction.

Although the overturning forces acting on a pier are different in their origin from the pressure of a bank, their effects and the necessary dimensions to resist them are determined in the same way as for retaining walls, and here it will be sufficient to make a few general remarks as to their character and magnitude.

40. With a low pier, or with one of moderate height, the dimensions required for supporting the bridge are such that there is little danger of overturning; but with a very high pier, the overturning forces should be carefully looked into and provision made to resist them.

A pier will never overturn about a sharp edge, unless the sharp edge crushes or sinks into the soil. This must be prevented by so designing the pier that the safe crushing strength of the masonry and the safe bearing power of the soil will not be exceeded. Since the maximum force of the wind and of the longitudinal thrust are of rare occurrence and of brief duration, the intensities of pressure usually permitted may be increased 25 per cent. when these overturning forces are considered as acting at the same time. In the Section on *Foundations*, Part 1, may be found tables giving the safe bearing power of various soils and the safe crushing stresses for masonry.

41. Sliding.—The forces tending to produce sliding of a pier on its subfoundation, or along any horizontal plane within the pier, are the same as those that tend to overturn it. The method of determining the effect of these forces and of providing for them is the same as for retaining walls.

42. Crushing.—The weight of the bridge and its load is usually transmitted to the pier at two or more points so far apart that, if ample provision is made for distributing this load over a sufficient area at the bridge seat, there need be no fear that crushing at any lower plane in the masonry may be caused by that weight. The only surfaces that need be considered are the bridge seat, the bottom of the pier, where it rests on the footing, and the base of the footing. The overturning forces cause the center of pressure to fall near the edge of the masonry, and the danger of crushing from this cause must be carefully investigated.

43. Crushing at Bridge Seat.—In determining the area of bearing of the bridge on the masonry, it should be borne in mind that the load is not generally uniformly distributed over the whole area of bearing; even with a perfectly rigid bearing plate there may be a greater load on one edge than on another, on account of the deflection of the bridge under its load, or on account of the overturning forces. Since it is impossible to compute the actual distribution of the pressure, it is customary to take into account only the vertical loads and to use a much smaller working stress than would be justifiable if the uneven distribution could be properly taken into account. On this basis, a working pressure of about 300 pounds per square inch is commonly used for limestone or sandstone, 400 for concrete, and 500 for granite. This gives an apparent factor of safety of about 10 or more.

44. Crushing at Lower Sections.—In considering the crushing strength of a pier at any horizontal plane below the bridge seat, it is necessary to take into account the uneven pressure due to the overturning forces. The principles involved, including the rule of the middle third, are fully explained in *Foundations*, Part 1; how they are applied will be illustrated by an example to be given presently.

Crushing of the masonry at special places may also be caused by other agents than the forces heretofore considered, as by blows from the wheels of vehicles, from vessels, ice, etc. These causes of destruction or damage will be taken up in a subsequent article.

45. Failure of Foundation.—The most frequent cause of failure of a bridge pier or abutment is a poor foundation, which may either crush entirely under the load so as to cause a settlement of the structure, or yield slightly at the point of maximum pressure, and so increase the already excessive overturning tendency by decreasing the leverage of the vertical forces and thus decreasing the resisting moment. The subfoundation, by its lack of uniformity, may cause uneven settlement and the rupture of the masonry. The upheaving tendency of frost is also a disturbing agent. A subfoundation that may have enough resistance to support the load may fail by being softened or washed away by the current, if in a running stream. A flood may sometimes cause the undermining of a structure that at ordinary stages of the water is far away from the water.

FORCES TO BE RESISTED

46. Wind.—In the design of a bridge and its supports, it is customary to provide for a wind pressure of 50 pounds per square foot of exposed surface when there are no loads on the bridge, or an alternative pressure (for railroad bridges only) of 30 pounds per square foot on the exposed surface of the bridge and the loads combined when there are loads on the bridge. The pressure of 50 pounds per square foot corresponds to a velocity of over 100 miles per hour, and is equivalent to a hurricane that would be commonly reported as "destroying everything in its path." Such wind is of extremely rare occurrence, but since it may occur at any time, even if for a very short time only, it must be provided for. The alternative pressure of 30 pounds per square foot corresponds to a velocity of over 80 miles per hour, and is the greatest pressure considered on moving loads, such as railroad cars, since it is sufficient to overturn empty cars; and it is assumed in practice that if the wind blows harder than this, it will be impossible to operate cars or other vehicles, so that there will probably be no loads on the bridge.

In determining the total pressure on a bridge, the intensity is multiplied by the exposed area. This area is usually taken equal to the area of the longitudinal vertical projection of the bridge, and is found by multiplying the width of each member by its length; in a truss bridge, the sum of the areas of all the trusses should be taken. In the design of highway bridges, the exposed area of the loads is usually very small in comparison with the exposed area of the bridge, and it is customary to neglect it. In railroad bridges, the cars present a vertical surface about 10 feet high and for the full length of an ordinary bridge. This gives 30×10, or 300, pounds per linear foot wind pressure on a train of cars. For purposes of computation, this pressure is treated as a single force acting through the center of pressure, which is from 6 to 9 feet above the rail. In what follows, the center of pressure will be assumed to be 7 feet above the top of the rail.

47. Current of Stream.—The pressure of the water in a flowing stream is given by the following approximate formula:

$$p = \frac{11 v^2}{8} = 2.96 V^2, \qquad (1)$$

in which p = pressure, in pounds per square foot, of exposed surface normal to the current;

v = velocity of current, in feet per second;

V = velocity of current, in miles per hour.

The velocity of the current varies with the depth, being greatest near the surface and much less at the bottom of the stream. Between these depths it varies approximately as the square root of the distance from the bottom. This makes the intensity of pressure due to the current vary approximately as the distance from the bottom, and the *average* pressure is usually assumed as about one-half that at the surface, with the center of pressure at two-thirds the height from the bottom to the top of the stream. In a pier having a cutwater with inclined sides, the resistance to the flow of the water, and consequently the pressure at the end, are reduced. The pressure on the end of such a pier is usually assumed to be three-quarters of the pressure on a pier with a square end. For a pier with a cutwater, formula **1** may, therefore, be written

$$p = 1.03 v^2 = 2.22 V^2 \qquad (2)$$

The pressure decreases from the surface to the bed of the stream the same as for a square-ended pier.

Let p_s = intensity of pressure at surface, in pounds per square foot;

$\left.\begin{matrix} v_s \\ V_s \end{matrix}\right\}$ = velocity of flow at surface in $\left\{\begin{matrix} \text{feet per second;} \\ \text{miles per hour;} \end{matrix}\right.$

A = area, in square feet, of section of pier made by a plane perpendicular to the current, between the surface and the bottom of the stream;

P = total pressure, in pounds, on the submerged portion.

Then, since the average intensity of pressure is $\frac{1}{3} p_s$,

$$P = \tfrac{1}{3} A p_s \qquad (3)$$

The value of p_s is found by substituting v_s or V_s in formula **1** or in formula **2**, according to the character of the end. Making the substitution, formula **3** becomes, for square-end piers,

$$P = .69 A v_s^2 = 1.48 A V_s^2$$

or, practically,

$$P = .7 A v_s^2 = 1.5 A V_s^2 \qquad (4)$$

For piers with cutwaters,

$$P = .52 A v_s^2 = 1.11 A V_s^2$$

or, practically,

$$P = .5 A v_s^2 = 1.1 A V_s^2 \qquad (5)$$

As already stated, the center of pressure is taken at a distance from the bottom equal to two-thirds of the distance between the bottom and the surface.

48. Ice and Drift.—In cold climates, ice, and in all climates, drift, may pack around the pier, and, by adhering to it, present a much wider obstruction to the current than does the pier alone. In some cases, a dam may be formed by the ice or drift packing solid from pier to pier; the result is that the obstruction increases, becoming higher and higher, until the piers fall down or the obstruction is forced through the opening by the pressure of the water. Damage may also result if large masses of ice floating with high velocities strike the pier; in this case, the pier may be turned over or part of it may be torn away. There is also an element of danger in the expansion of water when freezing. If the entire surface of the water between two piers freezes to a considerable depth, there will be a pressure on each pier, which, under some conditions, may be sufficient to dislodge them.

The dangers outlined in the preceding paragraph are real dangers and should never be ignored or minimized in the design of piers. The history of bridge failures and disasters shows that more failures have been due to the collapse of the piers on account of ice and drift than to any other one cause. Owing to the extreme difficulty in obtaining definite information regarding the amount of these forces, it is practically impossible to give any values that will be of help to the designer. Provision is usually made by the use of a large factor of safety.

49. With regard to the method of providing against these dangers, it may be said that the problem is one of judgment rather than of computation. Each case must be considered by itself, and all the conditions and possibilities fully studied before locating the piers. In many cases, the danger from ice and drift will be the determining factor in the location of the piers, and may result in the selection of a single span over the waterway, in order to keep the piers safe. In any case, the problem is rather one for the experienced engineer than for the draftsman or designer. A young engineer should not hesitate to seek advice from more experienced engineers when he has a problem of this kind.

50. Centrifugal Force.—In railroad bridges on which the track is curved, the centrifugal force F of a train causes an outward thrust, the magnitude of which is given by the formula

$$F = .00001167\, V^2 D\, W, \qquad (1)$$

in which V = velocity of train, in miles per hour;
D = degree of curve;
W = weight of train.

For practical work, the following formula may be used (see *Bridge Specifications*):

$$F = \frac{(4.5 - .2\,D)}{100}\, W D \qquad (2)$$

Formula 2 provides for about 61 miles per hour for a 1° curve, 55 miles per hour for a 5° curve, and 46 miles per hour for a 10° curve, which are as high speeds as usually obtain. The centrifugal force F is usually assumed to act about 6 feet above the top of the rail.

51. Longitudinal Thrust.—The longitudinal thrust is due to the friction of the wheels on the rails. When a locomotive is on a bridge and is exerting its maximum tractive force, the rails transmit an equal force to the bridge, and the latter transmits it to the supports. The tractive force has been found in some cases to be as high as 37 per cent. of the total weight on the drivers of the locomotive. When the brakes of a train are set, the wheels tend to slide on the rails, and this causes a frictional force that is transmitted to the supports. In practice, it is customary to assume that the greatest longitudinal thrust that need be provided for is equal to 20 per cent. of the greatest vertical load that can come on the bridge. The longitudinal thrust is usually considered as a single horizontal force acting above the pier at the height of the bridge seat; it should never be neglected, as it acts in the direction in which the pier is weakest.

The longitudinal thrust is transmitted to the piers and abutments of the fixed ends of the spans. The expansion ends, especially of long spans, are provided with rollers, so these ends cannot be relied on to transmit any of the thrust. For this reason, it is necessary to consider the arrangement of the spans and find out which will be expansion and which fixed ends. In case two fixed ends come on the same pier, that pier must be designed to provide for the longitudinal thrust coming from two spans.

ILLUSTRATIVE EXAMPLE

52. Data.—Let the pier shown in Fig. 16 support the fixed ends of two 150-foot spans of a single-track railroad bridge on a tangent. Let the data be assumed as follows:

Dimensions as given in figure, determined from practical considerations.

Weight of granite masonry, 150 pounds per cubic foot.

Weight of bridge, 2,500 pounds per linear foot.

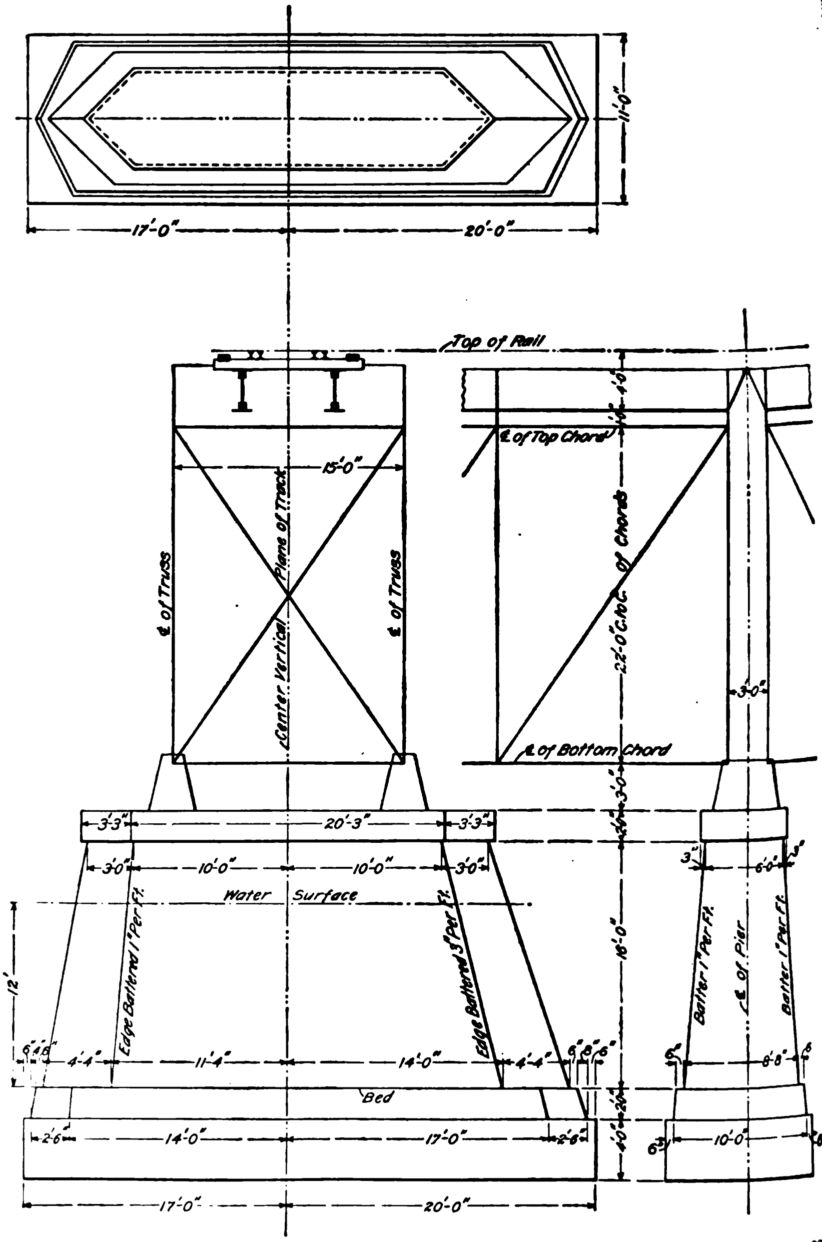

11'-0"
17'-0"
20'-0"
Top of Rail
15'-0"
℄ of Truss
Plane of Track
Center Vertical
℄ of Truss
℄ of Top Chord
22'-0" C. to C. of Chords
℄ of Bottom Chord
3'-0"
3'-3"
20'-3"
3'-3"
3'-0"
10'-0"
10'-0"
Water Surface
Edge Battered 1" Per Ft.
Edge Battered 3" Per Ft.
16'-0"
12'
4'-4"
11'-4"
14'-0"
Bed
2'-6"
14'-0"
17'-0"
Batter 1" Per Ft.
℄ of Pier
Batter 1" Per Ft.
6'-0"
6'-8"
10'-0"
17'-0"
20'-0"

FIG. 16

Surface of each truss exposed to the wind, 8 square feet per linear foot.

Surface of bridge floor exposed to the wind, 4 square feet per linear foot.

Minimum weight of train, 800 pounds per linear foot.

Maximum weight of train, 4,000 pounds per linear foot.

Velocity of the current at the surface of the water, 6 miles per hour.

Ice and drift collected about the up-stream end of the pier for a depth of 6 feet below high-water level, and for an average width of 18 feet.

It is required to determine the stability of the pier in regard to overturning and sliding, and also to find the maximum intensity of pressure on the foundation.

53. Overturning.—In investigating the factor of safety against overturning, it is necessary to consider overturning moments in both directions of the pier; that is, at right angles and parallel to the axis of the bridge. It is also necessary to consider the two conditions of a loaded and an unloaded bridge.

In discussing this subject, three cases will be considered. In Case I, the wind pressure is taken as 30 pounds per square foot (see Art. 46); in Case II, it is taken as 50 pounds per square foot. In Case I, it is customary to use the weight of an empty train, since the overturning moment is then the same as though a full train were considered, and the resisting moment will be less. The axis of moments will be taken along one edge of the bottom of the footing. Case III will deal with the overturning tendency of the forces parallel to the axis of the bridge.

54. Overturning Moments of Wind Pressure: Case I.—The wind pressure on the train is equal to 300 pounds per linear foot, applied 7 feet above the top of the rail (Art. 46). The pressure on one-half of each span may be assumed to be transmitted to the pier; this makes the total wind pressure on the train $300 \times (75 + 3 + 75) = 45{,}900$ pounds, acting at a distance (see Fig. 16, side elevation) of

$7 + 4 + 1 + 22 + 3 + 2 + 16 + 2 + 4 = 61$ feet above the bottom of the footing. The overturning moment of this pressure is, then,

$$45{,}900 \times 61 = 2{,}799{,}900 \text{ foot-pounds}$$

The wind pressure on the floor is $30 \times 4 = 120$ pounds per linear foot, applied 2 feet below the top of the rail. Then, the total wind pressure on the floor is

$$120 \times (75 + 3 + 75) = 18{,}360 \text{ pounds}$$

(The distance of 3 feet between the centers of bearings of the trusses is usually counted in computing wind pressure, because the floor and truss members project beyond the theoretical ends.) The distance from the center of this pressure to the bottom of the pier is $61 - 7 - \frac{1}{2} = 52$ feet. Then, the overturning moment of this pressure is

$$18{,}360 \times 52 = 954{,}700 \text{ foot-pounds}$$

The wind pressure on each truss is $30 \times 8 = 240$ pounds per linear foot, and where, as in this case, there is a train on the bridge, it is assumed that there is no shelter for the leeward truss; hence, the total wind pressure on the trusses transmitted to the pier is $2 \times 240 \times (75 + 3 + 75) = 73{,}440$ pounds. This pressure will be assumed to be concentrated half way between the chords of the trusses; that is, 38 feet above the bottom of the pier. Then, the overturning moment of this pressure is

$$73{,}440 \times 38 = 2{,}790{,}700 \text{ foot-pounds}$$

In calculating the pressure of the wind on the pier, and that of the water current, it is inadvisable to waste time in any unnecessary refinement, as the values of the wind and water pressures are but roughly approximate. The width of the pier subjected to the pressure of the wind is 6 feet 6 inches at the bridge seat, 6 feet at the bottom of the bridge-seat course, and 6 feet 8 inches at high-water level; then, for purposes of calculation, a uniform width of 6 feet 6 inches, or 6.5 feet, may be used, and the center of wind pressure taken 3 feet below the bridge seat. Then, the exposed area is $6 \times 6.5 = 39$ square feet, and the wind pressure is $30 \times 39 = 1{,}170$ pounds, the center of which may be taken as $4 + 2 + 16 + 2 - 3 = 21$ feet above the bottom of the pier. The overturning moment of this pressure is, then,

$$1{,}170 \times 21 = 24{,}600 \text{ foot-pounds}$$

55. Overturning Moment of Pressure Due to Current: Case I.—The intensity of pressure due to the current is given by formula **1**, Art. **47**. In the present case, since the velocity of the water at the surface is 6 miles per hour, the pressure at the surface is $2.96 \times 6^2 = 106.56$ pounds per square foot.

The intensity decreases uniformly with the depth, being $\frac{106.56}{2} = 53.28$ pounds per square foot at the bottom of the ice and drift, half way between the surface and the bottom. Then, since the exposed area of the ice and drift is given in the data as $6 \times 18 = 108$ square feet, the total pressure is

$$108 \times \left(\frac{106.56 + 53.28}{2}\right) = 8{,}631 \text{ pounds}$$

If p represents the intensity of pressure at the surface, p_1 the intensity of pressure at the bottom of the ice, and d_1 the distance from the surface to the bottom of the ice, the distance x_o from the surface to the center of pressure is given by the formula

$$x_o = \frac{(2p_1 + p)\,d_1}{3\,(p_1 + p)}$$

In the present case, $p = 106.56$, $p_1 = 53.28$, and $d_1 = 6$ feet. Substituting in the formula gives

$$x_o = \frac{2 \times 53.28 + 106.56}{3\,(53.28 + 106.56)} \times 6 = 2\tfrac{2}{3} \text{ feet}$$

Then, the distance from the center of pressure to the bottom of the pier is $4 + 2 + 12 - 2\tfrac{2}{3} = 15\tfrac{1}{3}$ feet, and the overturning moment of the pressure on the ice and drift is

$$8{,}631 \times 15\tfrac{1}{3} = 132{,}300 \text{ foot-pounds}$$

The pressure of the water on the pier below the ice and drift remains to be considered. It has already been found that the intensity of this pressure at the bottom of the ice is 53.28 pounds per square foot; this value may be substituted for p_o in formula **3**, Art. **47**, which gives

$$P = \tfrac{1}{2} \times 53.28\,A$$

The exposed area is 6 feet high and from 7 feet 8 inches to 8 feet 8 inches in width. The area is, then, $6 \times 8\frac{1}{6} = 49$ square feet; and, therefore,

$$P = \frac{1}{2} \times 53.28 \times 49 = 1{,}305 \text{ pounds}$$

This pressure may be assumed to be concentrated at one-third the distance from the bottom of the ice to the bottom of the river; that is, 4 feet above the latter, or 10 feet above the bottom of the footing. Then, the overturning moment due to this part of the pressure is

$$1{,}305 \times 10 = 13{,}100 \text{ foot-pounds}$$

56. Total Overturning Moment: Case I.—The total overturning moment can now be found by adding the six products just found. It is as follows:

	Foot-Pounds
Wind on train	2,799,900
Wind on floor	954,700
Wind on trusses	2,790,700
Wind on pier	24,600
Water on ice and drift	132,300
Water on pier below	13,100
Total overturning moment	6,715,300

If the track were on a curve, the centrifugal force of the train would be computed, and its moment about the base of the footing added to the moment already found.

57. Resistance to Overturning: Case I.—The resistance to overturning is provided by the weight of the train, bridge, and pier. The resisting moment of each is found by multiplying the weight by the distance of a vertical line through its center of gravity from the down-stream lower edge of the footing.

58. Resisting Moment Due to Weight of Bridge and Cars: Case I.—It will be assumed that, when both spans supported by the pier are fully loaded, one-half the load on each span, together with 3 feet between the spans, is supported by the pier, which makes the total weight of unloaded or empty cars $800 \times (75 + 3 + 75) = 122{,}400$

pounds. The horizontal distance from the center of the track to the lower edge of the footing is seen in Fig. 16 to be 17 feet; then, the resisting moment of the empty train is

$$122{,}400 \times 17 = 2{,}080{,}800 \text{ foot-pounds}$$

In a similar manner, the resisting moment of the bridge is found to be

$$2{,}500 \times (75 + 3 + 75) \times 17 = 382{,}500 \times 17$$
$$= 6{,}502{,}500 \text{ foot-pounds}$$

59. Resisting Moment Due to Weight of Pier: Case I.—The weight of the pier must be computed in two parts. The part above the water level is taken at its actual weight. The part immersed in water is decreased in weight on account of the buoyant effort of the water; deduction is made by decreasing the weight per cubic foot of the masonry by 62.5 pounds, the weight of a cubic foot of water.

The accurate calculation of the location of the center of gravity for each part of the pier is very tedious, and it is sufficiently accurate in practice to make use of an approximate method that requires less work and gives close enough results. In the present case, the pier may be assumed to be composed of horizontal layers 2 feet thick, each layer having a uniform horizontal cross-section equal to that at the center of its height, and the center of gravity being taken at the center of symmetry. The horizontal cross-section of each layer with pointed ends will then have the appearance shown in Fig. 17, the area A being given by the formula:

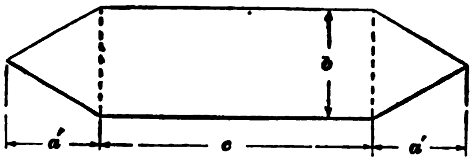


FIG. 17

$$A = \left(c + \frac{a + a'}{2}\right) \times b$$

The values of c, a, a', and b for the different layers can be calculated from the dimensions of the pier given in Fig. 16.

Commencing with the bridge-seat course, and continuing downwards, the volumes of the layers 2 feet thick are as follows:

Bridge-seat course,

$$2 \times \left[\left(20'\ 3'' + \frac{3'\ 3'' + 3'\ 3''}{2}\right) \times 6'\ 6''\right] = 305.5 \text{ cu. ft.}$$

First 2-foot layer,

$$2 \times \left[\left(20'\ 4'' + \frac{3'\ 1'' + 3'\ 1''}{2}\right) \times 6'\ 2''\right] = 288.81 \text{ cu. ft.}$$

Second 2-foot layer,

$$2 \times \left[\left(21' + \frac{3'\ 3'' + 3'\ 3''}{2}\right) \times 6'\ 6''\right] = 315.25 \text{ cu. ft.}$$

Third 2-foot layer,

$$2 \times \left[\left(21'\ 8'' + \frac{3'\ 5'' + 3'\ 5''}{2}\right) \times 6'\ 10''\right] = 342.81 \text{ cu. ft.}$$

Fourth 2-foot layer,

$$2 \times \left[\left(22'\ 4'' + \frac{3'\ 7'' + 3'\ 7''}{2}\right) \times 7'\ 2''\right] = 371.47 \text{ cu. ft.}$$

Fifth 2-foot layer,

$$2 \times \left[\left(23' + \frac{3'\ 9'' + 3'\ 9''}{2}\right) \times 7'\ 6''\right] = 401.25 \text{ cu. ft.}$$

Sixth 2-foot layer,

$$2 \times \left[\left(23'\ 8'' + \frac{3'\ 11'' + 3'\ 11''}{2}\right) \times 7'\ 10''\right] = 432.14 \text{ cu. ft.}$$

Seventh 2-foot layer,

$$2 \times \left[\left(24'\ 4'' + \frac{4'\ 1'' + 4'\ 1''}{2}\right) \times 8'\ 2''\right] = 464.14 \text{ cu. ft.}$$

Eighth 2-foot layer,

$$2 \times \left[\left(25' + \frac{4'\ 3'' + 4'\ 3''}{2}\right) \times 8'\ 6''\right] = 497.25 \text{ cu. ft.}$$

Ninth 2-foot layer,

$$2 \times \left[\left(30'\ 8'' + \frac{2'\ 5'' + 2'\ 5''}{2}\right) \times 9'\ 10''\right] = 650.64 \text{ cu. ft.}$$

The base, 4 feet deep, may be considered as one layer:

$$4 \times 37 \times 11 = 1{,}628 \text{ cubic feet}$$

The resisting moment of the portion above the water will now be found by multiplying the volume of each of the layers by the assumed weight of the masonry, 150 pounds per cubic foot, and then multiplying each of these products by the lever arm of the respective layer. The resisting moment f the portion in the water will be found by multiplying the olume of each layer by 150 − 62.5, or 87.5, pounds per ıbic foot, and then multiplying each product by the lever rm of that layer. The resisting moments are as follows:

		Foot-Pounds
Bridge-seat course,		
$(305.5 \times 150) \times 17' = 45{,}825 \times 17'$	=	779,025
First 2-foot layer,		
$(288.81 \times 150) \times 17'1'' = 43{,}322 \times 17'1''$	=	740,084
Second 2-foot layer,		
$(315.25 \times 150) \times 17'3'' = 47{,}288 \times 17'3''$	=	815,718
Third 2-foot layer,		
$(342.81 \times 150) \times 17'5'' = 51{,}422 \times 17'5''$	=	895,600
Fourth 2-foot layer,		
$(371.47 \times 87.5) \times 17'7'' = 32{,}504 \times 17'7''$	=	571,530
Fifth 2-foot layer,		
$(401.25 \times 87.5) \times 17'9'' = 35{,}109 \times 17'9''$	=	623,185
Sixth 2-foot layer,		
$432.14 \times 87.5) \times 17'11'' = 37{,}812 \times 17'11''$	=	677,465
Seventh 2-foot layer,		
$(464.14 \times 87.5) \times 18'1'' = 40{,}612 \times 18'1''$	=	734,400
Eighth 2-foot layer,		
$(497.25 \times 87.5) \times 18'3'' = 43{,}509 \times 18'3''$	=	794,039
Ninth 2-foot layer,		
$(650.64 \times 150) \times 18'6'' = 97{,}596 \times 18'6''$	=	1,805,526
Base layer,		
$(1{,}628 \times 150) \times 18'6'' = 244{,}200 \times 18'6''$	=	4,517,700
	Total,	12,954,300

The total moment is given to six significant figures, as a loser value would be an unnecessary refinement.

60. Total Resisting Moment: Case I.—The total esisting moment is equal to the sum of those just found, nd is as follows:

	Foot-Pounds
Weight of empty train	2,080,800
Weight of bridge	6,502,500
Weight of pier	12,954,300
Total resisting moment	21,537,600

61. Factor of Safety Against Overturning: Case I. Dividing the total resisting moment found in the preceding article by the total overturning moment found in Art. 56 gives for the factor of safety against overturning

$$\frac{21{,}537{,}600}{6{,}715{,}300} = 3.2$$

62. Overturning Moments: Case II.—In the second case, it is assumed that there is no train on the bridge, and so there is no overturning moment due to the wind pressure on the train. The wind pressure in this case is assumed to be 50 pounds per square foot (Art. 46); therefore, the overturning moments due to this pressure on the bridge and on the pier can be found by multiplying those found in Art. 54 by $\frac{50}{30}$. This gives the moments as follows:

Wind on floor,

$$954{,}700 \times \tfrac{50}{30} = 1{,}591{,}200 \text{ foot-pounds}$$

Wind on trusses,

$$2{,}790{,}700 \times \tfrac{50}{30} = 4{,}651{,}200 \text{ foot-pounds}$$

Wind on pier,

$$24{,}600 \times \tfrac{50}{30} = 41{,}000 \text{ foot-pounds}$$

The overturning moments due to the current are the same as found in Art. 55. Then, the total overturning moment is

$$1{,}591{,}200 + 4{,}651{,}200 + 41{,}000 + 132{,}300 + 13{,}100 = 6{,}428{,}800 \text{ foot-pounds}$$

63. Resisting Moments: Case II.—The resisting moments in this case are the same as before, with the exception of that due to the weight of the train, which is here omitted. The total resisting moment is, then:

	Foot-Pounds
Weight of bridge	6,502,500
Weight of pier	12,954,300
Total resisting moment	19,456,800

64. Factor of Safety Against Overturning: Case II. The factor of safety against overturning, in this case, is

$$\frac{19{,}456{,}800}{6{,}428{,}800} = 3$$

65. Overturning Parallel to Axis of Bridge: Case III.—The principal force tending to overturn the pier in the direction of the bridge is the longitudinal thrust of the train. Under some conditions, wind may blow approximately in the direction of the bridge, but the effect of this force is so small compared with the longitudinal thrust that it can be neglected with safety. In order to get the greatest force, the maximum weight of the train (4,000 pounds per linear foot, Art. **52**) must be used. Since the pier supports two fixed ends (Art. **52**), the weight of 306 feet of train, or 1,224,000 pounds, may be assumed to affect one pier. Then (Art. **51**), the longitudinal thrust is $.20 \times 1,224,000 = 244,800$ pounds, acting horizontally at the bridge seat, or 24 feet above the bottom of the footing. Half of this is usually assumed to be resisted by the rails. The other half causes an overturning moment about the lower side edge of the footing, whose value is

$$122,400 \times 24 = 2,937,600 \text{ foot-pounds}$$

66. Resisting Moment: Case III.—The resisting moment is provided by the weight of the bridge and trains for a distance of $75 + 3 + 75$ feet, and by the weight of the pier. The resisting moment offered by the bridge and train is

$$(2,500 + 4,000) \times 153 \times 5.5 = 5,469,750 \text{ foot-pounds}$$

The weight of the pier, making allowance for the buoyant effort of the water, can be found from Art. **59** to be 719,200 pounds. The resisting moment due to this weight is $719,200 \times 5.5 = 3,955,600$ foot-pounds. The total resisting moment is

$$5,469,750 + 3,955,600 = 9,425,400 \text{ foot-pounds}$$

67. Factor of Safety Against Overturning: Case III. Dividing the resisting moment by the overturning moment gives the factor of safety against overturning:

$$\frac{9,425,400}{2,937,600} = 3.2$$

68. Sliding.—The horizontal forces, such as wind pressure and longitudinal thrust, tend to make the upper part of the pier slide on the lower part. The resistance to sliding is equal to the vertical load multiplied by the coefficient of friction. The coefficient of friction for granite may be taken as .65. Three cases must be considered, in the same way as in the case of overturning.

69. Factor of Safety Against Sliding: Case I.—The horizontal forces in this case have been found in the preceding pages. They are: wind on train, 45,900 pounds; wind on floor, 18,360 pounds; wind on trusses, 73,440 pounds; wind on pier, 1,170 pounds; water on ice and drift, 8,631 pounds; water on pier, 1,305 pounds; total, 148,800 pounds.

The vertical loads are as follows: weight of empty train, 122,400 pounds; weight of bridge, 382,500 pounds; weight of pier, 719,200 pounds; total, 1,224,100 pounds. Then, the resistance to sliding is $1,224,100 \times .65 = 795,665$ pounds, and the factor of safety against sliding is

$$\frac{795,665}{148,800} = 5.3$$

70. Factor of Safety Against Sliding: Case II.—The horizontal forces in this case can be found in the same way as in the preceding pages. They are: wind on floor, $\frac{50}{30} \times 18,360 = 30,600$ pounds; wind on trusses, $\frac{50}{30} \times 73,440 = 122,400$ pounds; wind on pier, $\frac{50}{30} \times 1,170 = 1,950$ pounds; water pressure, 9,936 pounds; total, 164,900 pounds.

The vertical loads are as follows: weight of bridge, 382,500 pounds; weight of pier, 719,200 pounds; total, 1,101,700 pounds. Then, the resistance to sliding is $1,101,700 \times .65 = 716,100$ pounds, and the factor of safety against sliding is

$$\frac{716,100}{164,900} = 4.3$$

71. Factor of Safety Against Sliding: Case III. The only horizontal force that need be considered in the present case is the longitudinal thrust of 122,400 pounds (Art. **65**). The vertical forces are as follows: weight of train and bridge, $6,500 \times 153 = 994,500$ pounds; weight of pier, 719,200 pounds; total, 1,713,700 pounds. Then, the resistance to sliding is 1,713,700 × .65 = 1,113,900 pounds, and the factor of safety against sliding is

$$\frac{1,113,900}{122,400} = 9.1$$

72. Sliding on Foundation.—In the three preceding articles, the entire weight of pier was used, so that the factor of safety relates to a surface at or near the bottom of the pier. The tendency to slide is greater at the bottom than at any other height, and it has been assumed that the pier rested on rock. Piers of the size under consideration usually extend down to rock or very hard strata, and so the method outlined will give the proper factor of safety if the foundations are first class.

73. Pressure on Subfoundation.—When the load on a pier is vertical and the center of gravity of the base is vertically under the center of the load, the pressure on the subfoundation is evenly distributed, and the intensity is found by dividing the load by the area of the base. When the center of gravity of the base is not directly under the center of the load, or when horizontal forces act on a pier, the pressure is unevenly distributed over the base. The effect of horizontal forces is shown in Figs. 18 and 19. When the only load on a pier is a vertical load, the line of action of the pressure on the subfoundation may be represented by the vertical line $l\,m$. When a horizontal force also acts on the pier, it is represented by the line $l\,n$; the line $l\,o$, the resultant of $l\,m$ and $l\,n$, represents the line of action of the pressure on the base. Fig. 18 shows the condition when the horizontal force is at right angles to the pier, and Fig. 19 shows the condition when the horizontal force is parallel with the pier.

Under these conditions, the line of pressure does not pass through the center of the base, and the intensity of pressure varies. The formulas for finding the maximum and minimum intensities of pressure are:

$$p_1 = \frac{W}{A} + \frac{6\,W d}{A\,L} + \frac{6\,M}{A\,L} = \frac{W}{A} + \frac{6\,(W d + M)}{A\,L} \qquad (1)$$

$$p_2 = \frac{W}{A} - \frac{6\,W d}{A\,L} - \frac{6\,M}{A\,L} = \frac{W}{A} - \frac{6\,(W d + M)}{A\,L} \qquad (2)$$

in which p_1 = maximum intensity of pressure;
p_2 = minimum intensity of pressure;
W = total vertical load;
A = area of base of pier;
d = eccentricity, or distance from center of gravity of vertical load to center of base;
L = length of base in direction of horizontal force;
M = moment of horizontal forces about base.

In order to find the maximum intensity of pressure on the foundation, two cases must be considered: Case I, when

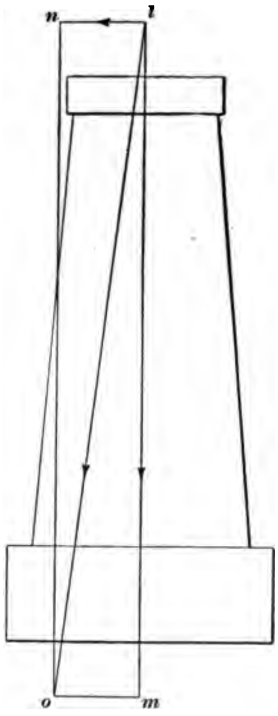


Fig. 18

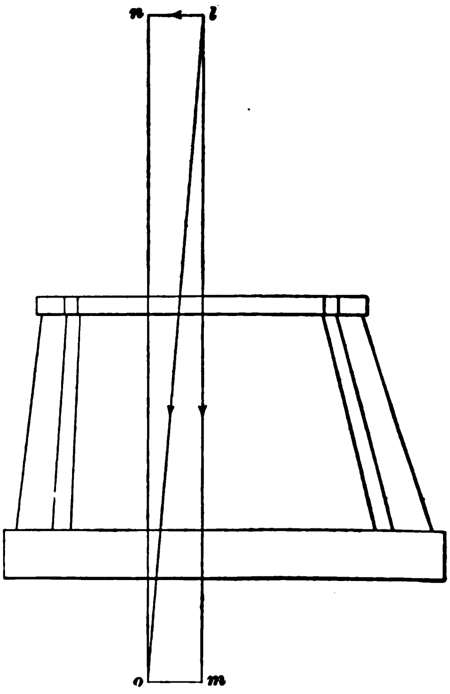


Fig. 19

there is a full load on the bridge and the horizontal forces are greatest and acting in the direction of the pier; Case II, when there is a full load on the bridge and the horizontal forces acting at right angles to the pier are greatest.

74. Formula **1** of the preceding article is used to find the maximum intensity of pressure, in order to see if it exceeds the allowable. Formula **2** is of interest only when the maximum intensity of pressure is greater than twice the average pressure; in this case, there is a tendency for one end of the pier to rise, and under this condition water may get under it and disturb the subfoundation.

In calculating the value Wd it is not necessary to get the location of the center of gravity of the vertical loads. It is invariably easier to multiply each vertical load by the distance from its line of action to the center of the base; the algebraic sum of all these products is equal to Wd.

75. Maximum Intensity of Pressure: Case I. The value of L is given in Fig. 16 as 37 feet, and, since the width is 11 feet, the value of A is $37 \times 11 = 407$ square feet. The value of W is found by adding the weight of the train, that of the bridge, and that of the pier, the last being decreased to allow for the buoyant effort of the water. Since the maximum weight of the train is 4,000 pounds and that of the bridge is 2,500 pounds per linear foot (Art. **52**), their combined weight is

$$(4{,}000 + 2{,}500) \times (75 + 3 + 75) = 994{,}500 \text{ pounds}$$

The weight of the pier, decreased by the buoyant effort of the water, is found from Art. **69** to be 719,200 pounds. The value of W is, then, $994{,}500 + 719{,}200 = 1{,}713{,}700$ pounds.

76. Since the vertical loads do not act through the center of the base, it is also necessary to find the value of Wd. The total weight of bridge and train that comes on the pier was found above to be 994,500 pounds. This acts 17 feet from the down-stream end of the pier (see Fig. 16); that is, 1.5 feet from the center, making the value of Wd for this load equal to

$$994{,}500 \times 1.5 = 1{,}491{,}800 \text{ foot-pounds}$$

The value of Wd due to the weight of the pier can most easily be found by calculating the position of the center of gravity of the pier. Since the moment of the weight of the pier about the down-stream end (Art. **59**) is 12,954,300 foot-pounds, and the weight of the pier (Art. **66**) is 719,200 pounds, the distance of the center of gravity from the down-stream end is

$$\frac{12,954,300}{719,200} = 18.0121 \text{ feet}$$

This gives 18.5 − 18.0121 = .4879 foot for the distance from the center of gravity of the pier to the center of the foundation, and the value of Wd for this load is

$$719,200 \times .4879 = 350,900 \text{ foot-pounds}$$

The total value of Wd is, therefore,

$$1,491,800 + 350,900 = 1,842,700 \text{ foot-pounds}$$

The moment of all the horizontal forces about the base is the same as the overturning moment found in Art. **56**; namely, 6,715,300 foot-pounds. Substituting the proper values in formula **1**, Art. **73**, we have

$$p_1 = \frac{1,713,700}{407} + \frac{1,842,700 \times 6}{407 \times 37} + \frac{6,715,300 \times 6}{407 \times 37}$$
$$= 7,620 \text{ pounds per square foot}$$

Since this result is much less than twice the average value $\left(\frac{W}{A} = 4,211 \text{ pounds per square foot}\right)$, it is not necessary to consider the minimum intensity of stress

77. Maximum Intensity of Pressure: Case II.—In this case, the resultant of all the vertical loads passes through the center of the base. Then, since the eccentricity is zero, the third term in formulas **1** and **2**, Art. **73**, becomes zero, and need not be considered. The values of W and A are the same as in Case I; but since the horizontal forces in this case are assumed to act at right angles to the length of the pier, L = 11 feet. The moment of the horizontal forces about the base is the same as that found in Art. **65**, or 2,937,600 foot-pounds. Substituting the proper values in formula **1**, Art. **73**, gives

$$p_1 = \frac{1,713,700}{407} + \frac{2,937,600 \times 6}{407 \times 11}$$
$$= 8,148 \text{ pounds per square foot}$$

Since this is less than twice the average value, it is not necessary to consider the minimum intensity.

78. Horizontal Forces Acting in Both Directions. The maximum intensities found in Cases I and II are those that occur when the horizontal forces act in but one direction. Since it is possible for them to act in both directions at the same time, this case must also be considered. Under this condition, however, since it will probably be of such rare occurrence, the intensity of pressure is allowed to exceed the working pressure by 25 per cent. In Case I, the maximum intensity of pressure extends for the full width of the pier at one end; in Case II, the maximum intensity of pressure extends for the full length of the pier at one side. The maximum intensity then occurs at the corner where these two edges meet, and is equal to the sum of the different terms obtained in Arts. 76 and 77, each term being used but once. For example, although the term $\frac{W}{A}$ occurs in each case, it is used but once in finding the sum. Then, the total intensity at one corner is

$$4{,}211 + 733 + 2{,}676 + 3{,}937 = 11{,}557 \text{ pounds per square foot}$$

If the intensity of pressure found in Cases I and II does not exceed the safe intensity, and if that just found does not exceed the safe intensity by more than 25 per cent., the design is assumed to be safe. Otherwise, the dimensions of the pier are increased and the foregoing computations repeated. Although the maximum intensity found by combining the two cases is greater than twice the average, no further attention need be paid to it. This high pressure will simply affect a very small corner of the pier.

ABUTMENT DESIGN

79. The method of arriving at the dimensions of an abutment are much the same as those used for piers. In the first place, the dimensions necessary to satisfy practical conditions are found; then, calculations are made to ascertain whether the abutment satisfies the theoretical conditions.

PRACTICAL CONSIDERATIONS

GENERAL FEATURES

80. Classes of Abutments.—There are three general classes, or forms, of abutments: the *wing abutment;* the T *abutment;* and the U *abutment.* The **wing abutment** consists of a mass of masonry extending the full width of the bank it restrains. It usually decreases in height from the edges of the bridge to the foot of the bank. Several forms of wing abutments are shown in the following pages. In the **T abutment** there is a face wall, called the **head,** on which the bridge rests, and a wall running back under the track or road to the top of the bank, as shown in Figs. 20 and 21. This wall is called the **stem,** or **tail.** The head usually extends beyond the stem at both sides.

In the **U abutment,** there is a head similar to that in the T abutment; instead of one stem, there are two walls running back as far as the top of the slope, as shown in Fig. 22. These walls are even with the outer edges of the roadway. The space between the walls may be filled with earth or loose rock.

81. Bridge Seat.—As in the case of piers, the first thing to be considered is the size of the top of the abutment. An abutment usually supports one end only of a span, so the width of the bridge seat for this purpose need only be about

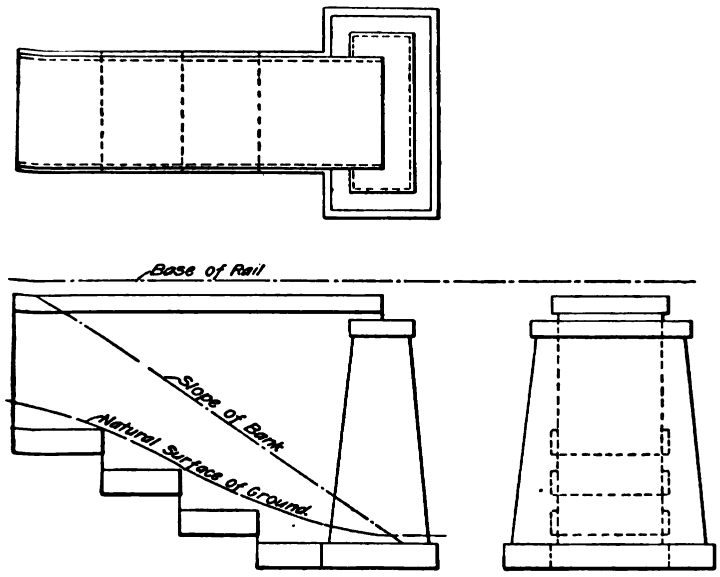

Base of Rail
Slope of Bank
Natural Surface of Ground

FIG. 20

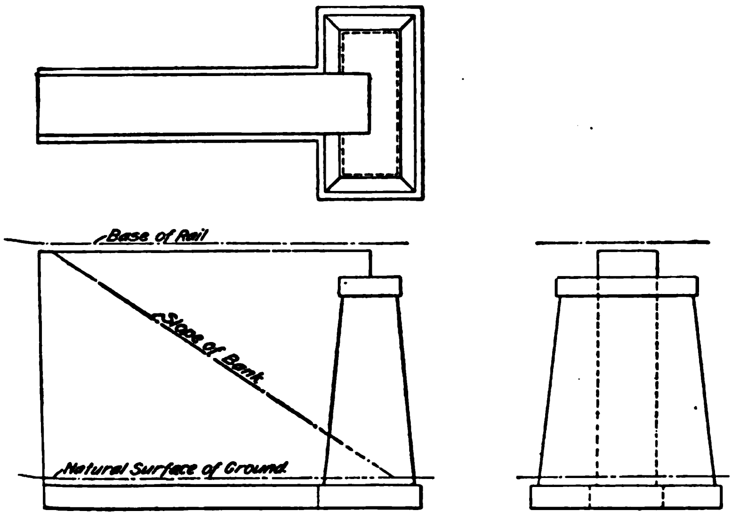

Base of Rail
Slope of Bank
Natural Surface of Ground

FIG. 21

one-half as wide as that of a pier. On account of the parapet, the bridge-seat course is usually continued back under it. The required width of bridge seat and the width of the parapet at the elevation of the bridge seat control the width of the top of the abutment. The ends of the bridge seat are carried a few feet beyond the trusses, or girders, or to the edge of the roadway.

82. Parapet.—The parapet, or back wall, of an abutment is simply a retaining wall to prevent the earth above the bridge seat from falling down. In order to keep it from

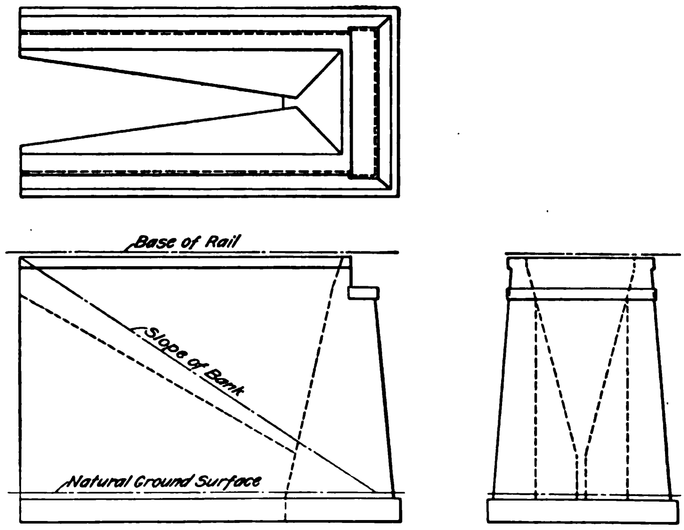


Fig. 22

tipping over or sliding forwards on account of the pressure of the earth and of the traffic, the parapet should have a width at the bottom of from .4 to .5 the height.

For a distance of about 3 feet down from the top, the back of the parapet should be given a smooth batter, usually called the **frost batter**, so that the action of the frost cannot move it. The face of the parapet should be set back far enough from the end of the bridge so that that end will not hit the parapet when the bridge expands. A distance of 3 inches is usually specified as the closest the bridge must ever come to the parapet.

83. Pedestal Blocks.—The remarks given in Art. 23 with regard to pedestal blocks for piers apply to those required for abutments. When used on abutments, it is advisable to have one surface of the block in contact with the parapet.

84. Pockets in Bridge Seats.—Great care should be taken in the arrangement of blocks and parapets on bridge seats that no pockets are formed. The word "pocket" is usually applied to portions of the work where dirt may collect and from which it is difficult to remove it. Since the earth behind an abutment usually extends up to nearly the top, winds usually blow some of it over the parapet on the bridge seat. If the steelwork is near the pocket, the dirt will pile up against it, absorb moisture from the air, and cause the steel to corrode rapidly. Pockets can be avoided by the use of end floorbeams, which do away with the pedestal blocks, and by continuing the parapet straight from end to end of the abutment so the wind can have a clean sweep across. For this purpose it is advisable to have the main trusses or girders at least 1 foot above the bridge seat.

85. Batter.—The front faces of many abutments are made plumb, because the appearance is not marred here as in the case of a pier with both faces plumb. It is somewhat better, however, to give the face a batter of about $\frac{1}{2}$ inch per foot. It should be borne in mind that the batter of the front face increases the length of the span, and, therefore, that batter should not be made unnecessarily large. No standard batter is used at the back, the slope being controlled by the top width and the required width of base.

86. Width of Base.—The width of base should be determined by means of theoretical considerations, allowing for the pressure at the back due to the earth filling increased by the weight and jar of the traffic. Generally, it is not usual in practice to make a theoretical determination of the required width of base, but to make the width a certain fraction of the height; in the case of high or unusual abutments, however, calculations should be made. It has been found by practical experience that the width of base should be from three-eighths to one-half the distance from the top of the fill to the level of the base. For first-class stone masonry or solid concrete, three-eighths of the height is sufficient; for second-class masonry, one-half the height should be used.

87. Footing.—The footing course of an abutment does not have to extend beyond the back of the masonry, but it should extend beyond the front face. This is due to the fact that the overturning tendency causes higher pressure at the face of the abutment, and the footing must be extended here in order to distribute the pressure over a greater area. In some cases, several footing courses, each projecting beyond the one above, are used.

T ABUTMENTS

88. Since the head of a T abutment has no bank to restrain, it may be made just wide enough at the top to accommodate the bearing plates of the bridge, and the front and back faces may be continued down to the base with a slight batter. The base of the head is at the foot of the slope, and the bank slopes up to the end of the stem. T abutments were formerly the most common, because of their simplicity and of the fact that low-grade masonry could be used. On the other hand, the T abutment requires much more masonry, on account of the stem, than abutments of other forms. For double-track railroads or wide streets, the amount of masonry is excessive.

The most serious objection to this form of abutment, when used for railroad bridges, is that it gives a support to the bridge that is too rigid, making riding uncomfortable to passengers and injurious to the rolling stock and track. In some cases, 1 or 2 feet of ballast is inserted between the track and the masonry, but this does not wholly remove the bad effects of rigidity.

U ABUTMENTS

89. The **U** abutment is practically a wing abutment in which the wings are parallel to the track, although it has more the appearance of a **T** abutment. The slope of the bank is sometimes carried down outside the wings, but more frequently the space between the wings is filled with earth or broken stone. In the latter case, the two wings must be designed in the same way as a retaining wall. When the earth is allowed to run down outside the wings, they may be made about three-quarters the width required by the other condition; this is permissible because the earth outside the wings helps to support them.

WING ABUTMENTS

90. Types of Wings.—There are three common types of wing abutments; namely, the **straight-wing abutment,**

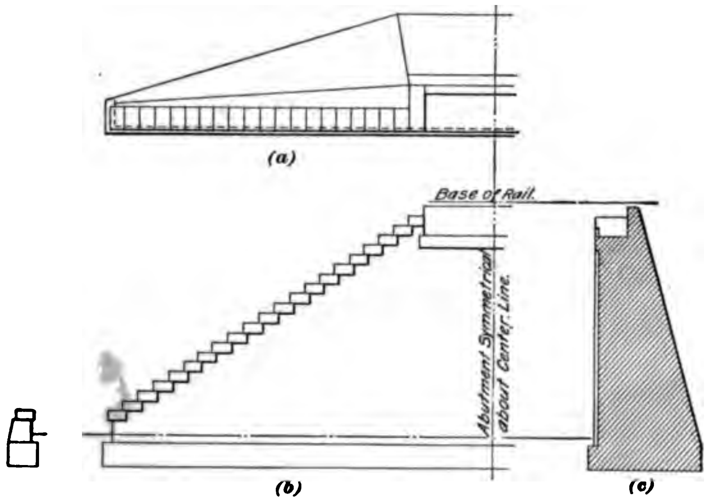
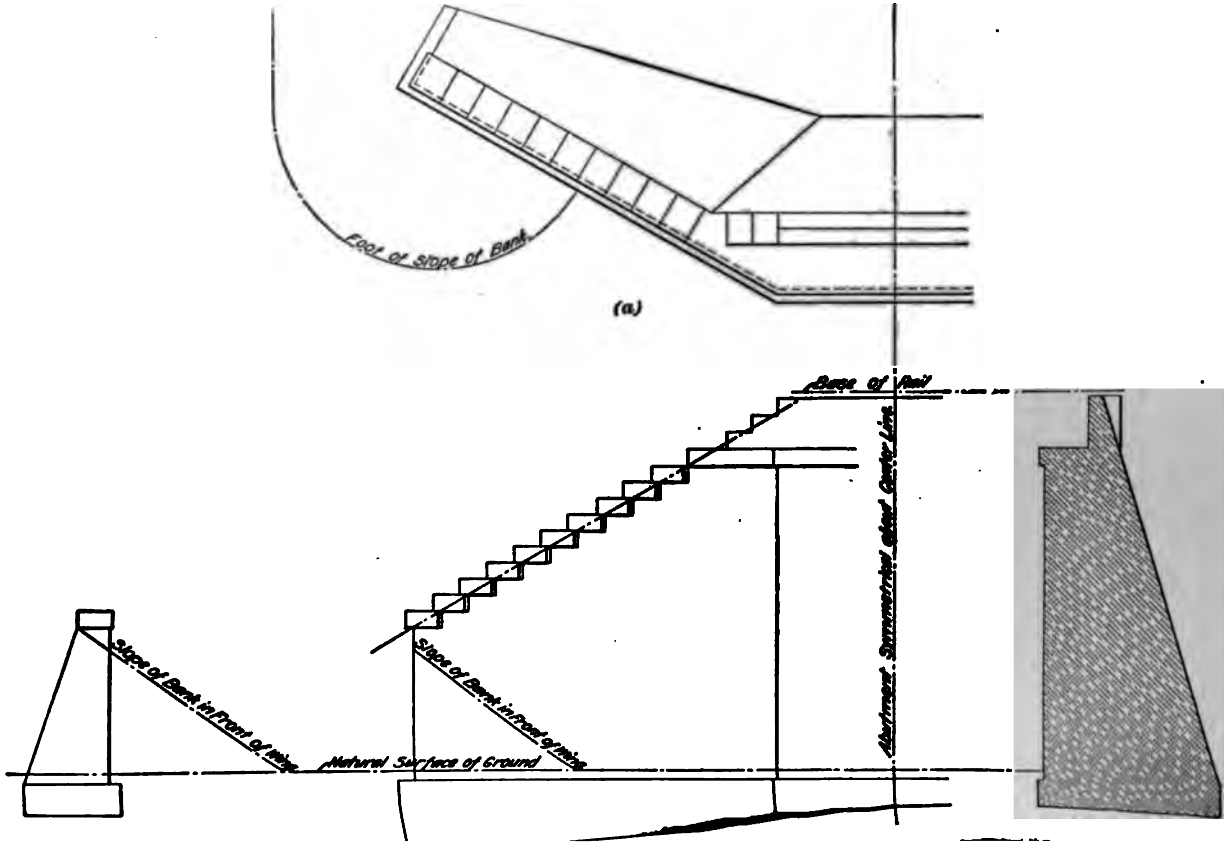


FIG. 23

Fig. 23, in which the face of the wings is in the same plane as the face of the abutment proper; the **flaring-wing**


Foot of Slope of Bank
Base of Rail
Abutment Symmetrical about Center Line
Slope of Bank in Front of Wing
Slope of Bank in Front of Wing
Natural Surface of Ground

(a)

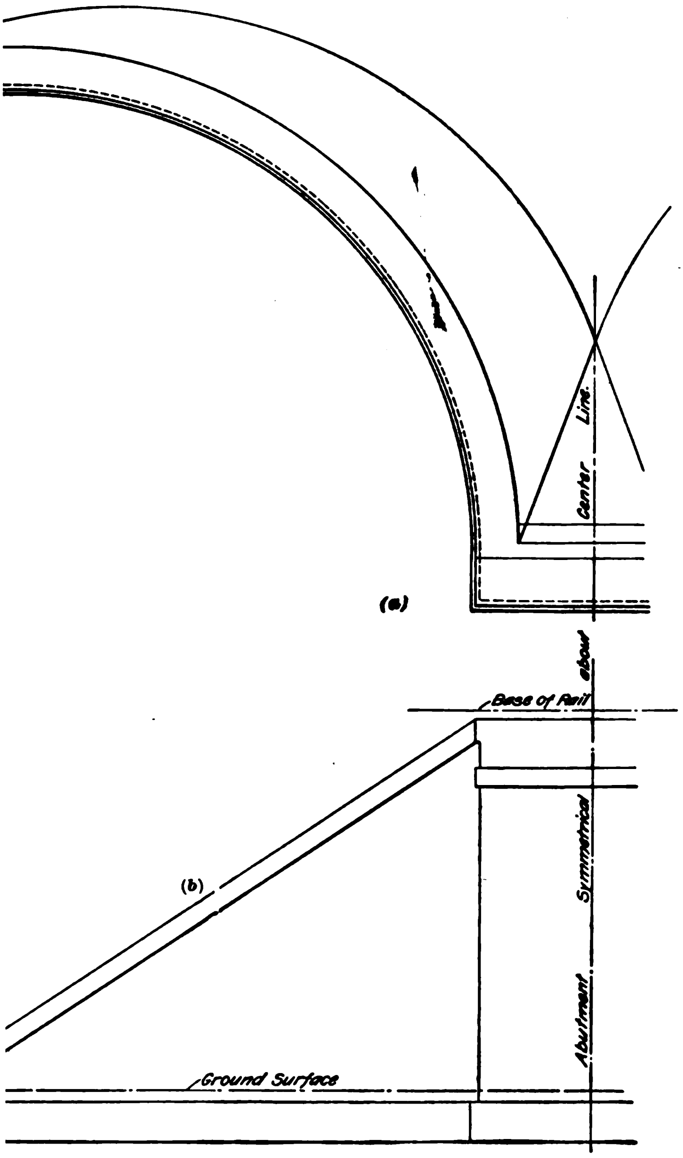

Center Line
(a)
Base of Rail
Abutment Symmetrical about
(b)
Ground Surface

FIG. 25

abutment, Fig. 24, in which the faces of the wings make an angle with the face of the abutment proper; and the **curved-wing abutment,** Fig. 25, in which the wings are curved. In general, the curved surfaces of the wings start almost normal with the face of the abutment and turn through about 90° until they are at right angles to the track.

91. Size of Wings.—Each wing starts at the top of the abutment and decreases in height until it reaches the foot of the bank, where the height reaches zero. The top surface of the wing follows approximately the intersection of the plane of the wing with the plane of the slope of the bank. In Fig. 26, (*a*) represents the form of wing that might be used with concrete, while (*b*) and (*c*) indicate the forms for stone masonry. The inclined line in (*a*) is smooth, because it is easier to finish concrete in this manner. The forms (*b*) and (*c*) are stepped, the height of the steps being the same

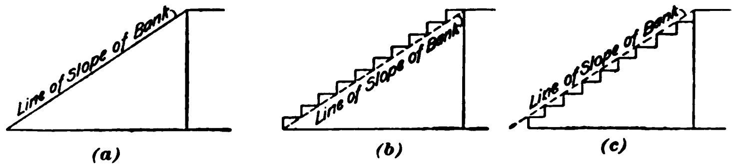


FIG. 26

as the thickness of the courses of the stone. In order to keep the earth from covering the wing, the steps should be carried beyond the slope, as shown in (*b*). To save masonry, the steps are sometimes made shorter and the earth allowed to cover the wing, as at (*c*).

In the design of wings, it is assumed that the bank has a slope of $1\frac{1}{2}$ horizontal to 1 vertical. When the earth is allowed to run over on the steps, as at (*c*), the abutment presents a very untidy appearance, and the extra expense of the form (*b*) is warranted by the improved appearance. In the form shown in Fig. 26 (*a*), the top of the wing should be from 6 to 12 inches above the earth.

92. Toe of Wing.—When it is necessary to keep the earth from running around in front of the wing, the latter is continued down to the foot of the slope. When flaring wings are used, however, it is allowable for the earth to run around somewhat, unless the abutment is on the shore of the stream. A sketch of a wing in which this was done is shown in Fig. 24, the broken line at the left in (*a*) showing the foot of slope. Some masonry is saved by adopting this method.

The thickness of a wing is usually the same at the top as that of the abutment proper; the width or thickness gradually decreases until the toe is reached, where the width is usually about 2 feet.

93. Straight-Wing Abutments.—The straight-wing form of abutment is particularly fitted for bridges over city streets, and is probably the most economical for this purpose. The face of the abutment and wings is placed at the street line, and gives a good appearance. Since in this case it is impossible to have any earth in front of the abutment, the wings must be carried to the ends of the slopes.

The principal advantage in the use of straight-wing abutments for railroad bridges is that in case an increase in the number of tracks is made, the courses can be so arranged that there will be little difficulty in extending the abutment. An outline of a straight-wing abutment is shown in Fig. 23; (*a*) is a plan of one end of the abutment, (*b*) is an elevation of the portion shown in (*a*), and (*c*) is a cross-section through the bridge seat.

In some cases, this form of abutment is slightly modified by making the back straight, and thus causing the wings to make a slight angle with the face of the abutment. This is not desirable, as it mars the appearance somewhat, and also because it involves extra expense in stone cutting in order to form the angle.

94. Flaring-Wing Abutments.—For abutments at the edge of a river, flaring wings are usually preferable to other wings. They give a greater opening for the stream above the bridge, and help to direct the current into the proper channel, thus preventing the water from injuring the foundation by getting behind the abutment. Where the greatest economy is desired, the wings may be made shorter and the earth allowed to run around in front, as previously shown in Fig. 24. This reduces the amount of masonry.

If the masonry is first class and well bonded, the flaring-wing abutment gives greater stability, for overturning or sliding can occur only when one part of the abutment tears away from the other part. In order to use the least amount of masonry in the wings when the earth is allowed to run

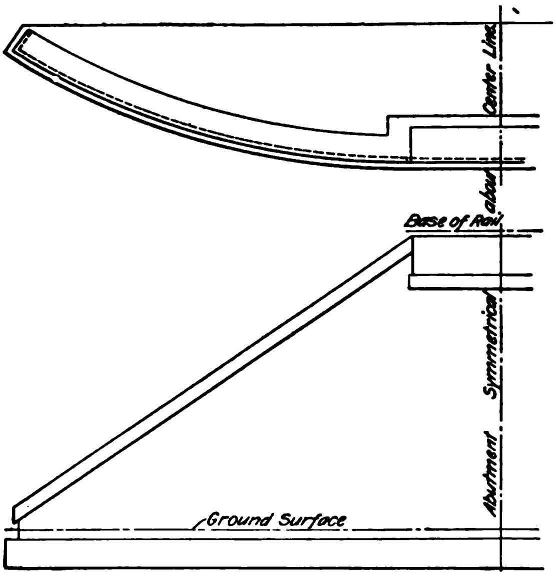


FIG. 27

around in front, they should make an angle of about 30° with the plane of the face of the abutment produced. This is the angle generally used; however, for greatest efficiency at a stream in directing the flow toward the opening, the angle should be about 10°, and the wings should be carried to the extreme foot of the bank, in order to keep the current from washing away the earth and thereby undermining the abutment.

95. Curved-Wing Abutments.—The principal reason for the use of abutments with curved wings is the improved appearance. With very high abutments, where filling material cannot be easily obtained, there is the additional advantage that they save filling. Fig. 25 shows one end of an abutment with curved wings. The front elevation, shown at (*b*), indicates that the top of the curved wing follows the slope of the bank.

Another form of curved wing is shown in Fig. 27. This form has the advantage that, like flaring wings, it easily deflects the current. It also has a pleasing appearance, and is not so expensive as some other forms of curved wings.

SKEW ABUTMENTS

96. It is frequently necessary to build abutments at some other angle with the bridge than 90°. The bridge is then a skew bridge, and the abutments are called **skew abutments.** For such construction, straight wings may be used, as shown in outline by AD in Fig. 28. One end of

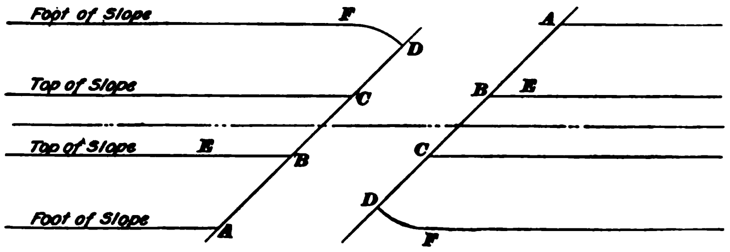
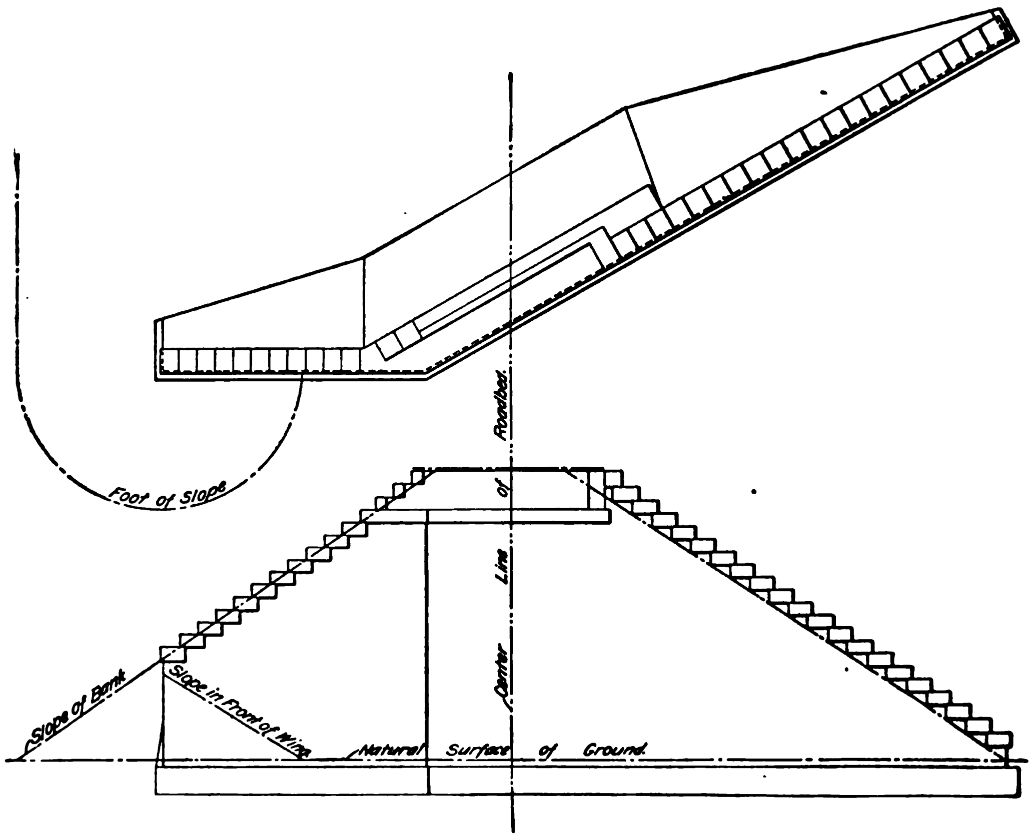


FIG. 28

each abutment can be made shorter than the other; this can be seen in the figure at F and D, both of these points being at the proper distance from the top of the slope C. In many cases, the wing is made to extend from C to F at right angles to the bridge. This really makes a flaring wing for this end of the abutment, as shown in Fig. 29. In case there is no water at the face of the abutment, masonry may be saved by allowing the earth to run around the end of the wing.


Foot of Slope
Center Line of Roadbed.
Slope of Bank
Slope in Front of Wing
Natural Surface of Ground.

In the design of skew abutments, great care should be taken that the bridge seat is wide enough. As the bearing plates are set on a skew, they take up more width on the bridge seat than when there is no skew.

PIER ABUTMENTS

97. When a trestle is built across a valley, the greater part of the distance is spanned by girders or trusses. Near the ends, however, it is usually found more economical to fill for a short distance. It is then necessary to build an

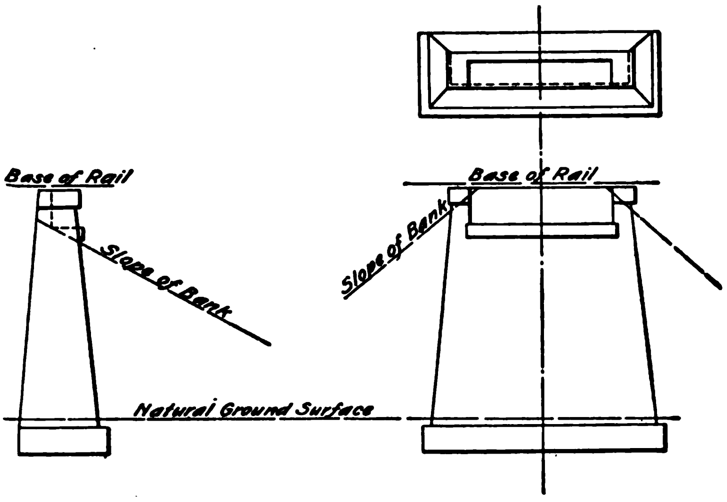


FIG. 30

abutment in this new ground for the end of the trestle. This is usually accomplished as shown in Fig. 30. A pier is built on the natural surface of the ground, and the fill allowed to run around in front of it. This form of structure is called a **pier abutment** (see Art. 2).

THEORETICAL CONSIDERATIONS

98. Forces to be Resisted.—On account of the fact that an abutment is in contact with an earth fill, it can overturn or slide in but one direction; namely, away from the bank. It is prevented from moving sidewise by the friction of the earth behind it and also by the wings. The forces that act to overturn the abutment are the pressure of the earth and the longitudinal thrust of the train.

99. Calculations.—The pressure of the earth is found in the manner explained in *Retaining Walls*, the weight of the train or other loads being added to the weight of the earth as the surcharge. The longitudinal thrust is found in the manner given for piers in Art. **51.** The methods of determining the factors of safety against overturning and sliding, and the intensity of pressure on the subfoundation, are the same as for piers.

CONSTRUCTION

MATERIALS

100. The materials most used for piers and abutments are stone, concrete, and timber. In addition to these, large cylinders having steel or cast-iron outside surfaces and concrete interiors are sometimes used.

101. Stone.—The most common material is stone. The very best stone available is generally selected. There is little economy in using poor masonry, as more is required than when good masonry is used. Granite, when available, is the best stone, because it is hard, strong, and durable, and can be brought to required shapes more easily than other stones of equal strength. Its only weakness is that it disintegrates easily when exposed to fire; but this danger seldom occurs in piers and abutments.

Syenite, gneiss, quartz, trap, and porphyry are also excellent stones, but are hard to work, and so are seldom used for bridge masonry. Quartzite also is a good stone. When these stones are not available, some grades of sandstone or limestone may take their place.

Certain kinds of stone are not suited for bridge masonry, on account of their tendency to disintegrate from the action of frost, acids, attrition, or fire. As a general rule, there is less danger from frost in those stones that are compact and non-absorptive than in porous stones, which permit water to enter and freeze inside. A compact stone is also less liable to absorb acids from the air or the water than a porous stone.

Most sandstones are very absorptive, and the cementing material is easily dissolved if it consists of lime, iron oxide, clay, magnesia, or feldspar. Quartzite, or sandstone in which the cementing material is of quartz, is less permeable and contains less matter that can easily dissolve. A stratified stone is also more liable to disintegration than an unstratified stone, because it offers straight paths between the strata for moisture to reach the interior, and also because it offers easy cleavage lines. In case stratified stones are used, they should be placed so that the strata will be horizontal.

Limestone consists chiefly of carbonate of lime, and is easily injured by nitric and by hydrochloric acid. A magnesian limestone is much more durable than a pure limestone. Exposure to fire reduces the limestone to almost pure lime, which is soluble in water.

102. Concrete.—Concrete, both plain and reinforced, is much used for bridge masonry. It is more durable than many limestones and sandstones, and, when properly made and laid, is sometimes as strong and durable as granite. The principal advantages of concrete are its comparatively low cost and the fact that it makes a monolithic structure. A Portland-cement concrete mixed in the proportions of 1:2:4 with hard, angular, well-graded stones for the aggregate, is a sufficiently good material for all bridge masonry. In many cases stone is used for the face of an abutment or pier, and the interior is made of concrete.

103. Timber.—In temporary construction, and in some cases where time does not permit the erection of masonry supports, bridges are supported by timber structures. These usually consist of several framed or pile bents set parallel and close to one another. The abutments in this case are similar to the end bents described in *Trestles*, except that more than one bent may be used. In permanent structures, wood should never be used except for the foundations, and when so used the top of the timber construction should be below the lowest level the water ever reaches.

104. Joints.—The joints in stone masonry should be made as thin as possible, and rich Portland-cement mortar used. There is no economy in using a first-class stone and then binding the whole together with a poor mortar.

105. Steel Cylinders.—Sometimes, piers are made of steel or cast-iron cylinders filled with concrete. These are easy to place and are well adapted to soft soils. Short sections are placed and bolted one on top of another, and the whole is forced into the ground. The interior is usually excavated while the cylinder is being driven, and then filled with concrete. In this form of construction, a pier usually consists of two cylinders with a set of heavy girders extending across their tops.

DETAILS

106. Specifications.—The specifications for bridge masonry may be substantially the same as for retaining walls, the principal difference being in the bridge seat. All the masonry above the footing should be first-class stone laid in Portland-cement mortar. The stones should be laid with horizontal and vertical joints, the courses being from 18 to 24 inches in height. The faces can be left rough, provided no point projects more than 3 inches beyond the edge of the joint. All joints should be perfectly straight.

The bridge-seat course should be bush-hammered on top, and brought to a level surface at the proper elevation for the bearing plates. All joints above the footing should be pointed with neat cement with a ¼-inch semicircular bead, except at the top of the bridge seat, where they should be level with the stone. For bonding purposes, not less than one-quarter of the face should be composed of headers. All stones should be at least 4 feet long, but no stone should be longer than five times its height. On account of the impact of ice and drift, the masonry near the water-line of a pier in a stream may be of a better class than the remainder. The footing may be of coarse rubble, with stones about ½ cubic yard in size.

107. Bridge-Seat Course.—The bridge-seat course should be so designed that the stones will have a good bond. Except where the bearing is very large, there should be but one stone under the bearing plate. In general, the joints should be kept as far as possible from the edges of the bearing plates. The top of each stone under a bearing plate should be dressed smooth. Pedestal blocks should not be used for a height less than 8 inches, and the height should not be less than one-fifth the length of the block. For a small difference in elevation of the bearings, castings may be used, or the bridge-seat stones may be made of different thicknesses, as previously explained in connection with Fig. 5 (*b*).

108. Parapets.—The parapet of an abutment helps to bond the bridge-seat stones. The latter should extend at least 9 inches behind the face of the parapet. When practicable, it is advisable to have the bridge-seat stones extend entirely across the top of the abutment. There is no necessity for dressing the face of a parapet. In railroad bridges, it is customary to have the top of the parapet from 8 to 12 inches below the base of rail, and to rest a tie on its top. In highway bridges, it was customary to have the top of the parapet even with the street surface. This gave rise to an uncomfortable jar in passing over the masonry. At present, the top of the parapet is finished off below the street surface, and the flooring is continued across.

109. Pedestals and Piers for Trestles.—The small depth of a trestle pier requires great care in properly bonding the separate stones, so that the loads shall be properly distributed over the base. Fig. 7 shows an outline of a pier suitable for the support of a high trestle bent. When separate pedestals are used, the top of each should be a single stone. Even when concrete is used, the cap should be of a good hard stone. Fig. 31 shows a stepped pedestal of stone masonry. When the lower part is concrete, the sides are sloped, as shown in dotted lines.

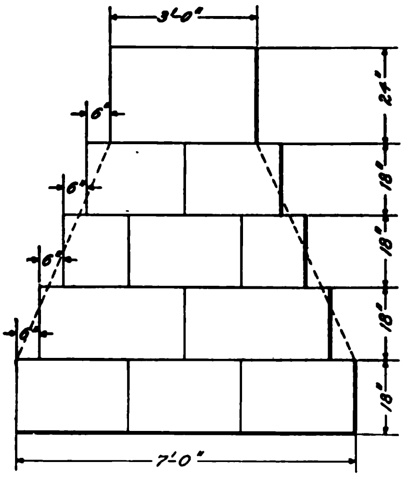


FIG. 31

110. Batter.—The batter of the face of a pier or of an **abutment** is obtained by cutting the face of the stone so that it will slope the proper amount. The batter at the back of an abutment, where it is hidden by the earth, is obtained by allowing the ends of the stones to project beyond those above; the back ends are usually left in the shape in which they come from the quarry.

111. Undermining.—Unless piers or abutments rest on solid rock, water is likely to wash away the material under them. This is called **undermining**, and is especially likely to take place in piers placed in running streams. The danger from this is not apparent, since the damage is done under water. In order to avoid undermining, the pier should be carried to a great depth, and broken stone piled up around it. This broken stone, when used for this purpose, is called **riprap.**

BRIDGE DRAWING

INTRODUCTION

1. The general principles that govern the preparation of all bridge drawings are the same as those that have been explained and illustrated in *Introduction to Construction Drawing*. In the present Section, the additional special information required for bridge drawing will be given, the application of the principles being illustrated by four detail drawing plates. The student is required to send the tracings of the drawing plates, one at a time, to the Schools for correction. He should retain the pencil drawings for future reference; it may be necessary for him to trace them again.

2. Structural Shapes and Standards.—Almost all the parts used in structural and bridge work have been standardized to a certain extent, and their dimensions and properties have been tabulated and printed in handbooks by the manufacturers of rolled steel. Bridge draftsmen should have a copy of the handbook used by the designer. All the information of use to bridge draftsmen that is usually contained in structural-steel handbooks is given in *Bridge Tables*. The use of these tables is explained in *Bridge Members and Details*, Parts 1 and 2. In preparing the drawing plates, however, it will not be necessary to consult tables to any great extent, as all the dimensions required for the laying out of the work are shown on the plates that he will receive, or are given in the following pages.

3. Center Lines.—In making a drawing of a part of a bridge, it is customary to draw first the center line of the part, and then refer all points to that line. In addition to the first or main center line, other lines are drawn to represent the centers of details. For example, the line on which the centers of a number of rivets in a row are located is drawn before locating the rivets. That line is sometimes called the **center line of the rivets**; more frequently, however, it is called the **gauge line**. Strictly speaking, only symmetrical objects have center lines; but, in bridge work, lines that serve as bases of reference in locating details on non-symmetrical objects are spoken of as center lines. Center lines on a drawing are very important, and should be laid out very accurately, as they are used as standards of reference from which to lay out distances and locate points.

4. Dimensions.—A frequent source of trouble in making a bridge drawing is the location of the dimension lines. The drawing is usually so filled with views of details that it is difficult to draw the dimension lines without interfering with other lines. Draftsmen should be very careful so to locate the dimension lines and write the dimensions that no ambiguity can arise. When there are several consecutive dimensions, such as a number of rivet spaces that are unequal, it is well to show them in a line, so that they can be added readily when the drawing is being checked. In such a case, it is customary to give both the separate distances between rivets and the distance between the end points, or between two easily distinguished points near the ends. This distance is the sum of a number of smaller distances, and, by giving it, the necessity is avoided of adding the smaller distances whenever the total distance is wanted. The method is illustrated in Fig. 1, which represents the connection angle at the end of a stringer. The lines *aa* and *bb* are the center lines of the flange rivets, and are $2\frac{1}{4}$ inches from the top and bottom, respectively, of the flange angles. The spacing of the rivets in the connection

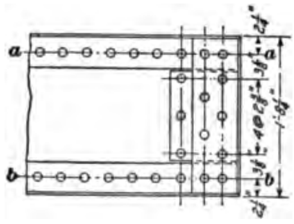


Fig. 1

angle is given by the left-hand line of dimensions. For convenience of reference, the sum of all the dimensions contained in the left-hand line is given at the right. This sum is the distance from the top of the top flange angle to the bottom of the bottom flange angle, and is usually spoken of as the vertical distance back to back of the flange angles.

5. When a dimension line is too short to permit the dimension to be written legibly between its ends, the number indicating the dimension is placed outside the line and close to it, as the dimension of 2¼ inches at the top and bottom of the stringer in Fig. 1. Dimension numbers should, as a rule, be placed as close as possible to the objects or parts to which they refer, except where, when so located, they interfere with other lines or numbers. In all cases, they are located with respect to the other lines or numbers in such a way that there will be no doubt as to their meaning.

6. Shortened Views.—When a part of a bridge has the same form, dimensions, and parts for a considerable distance, as from *a a* to *b b*, Fig. 2, it is unnecessary to show the whole of it to scale if the space on the drawing is limited. In such a case, a portion of the member may be left out, and the ends moved closer together. The parts that are then shown will be drawn to scale just as though the entire member were drawn, but the dimensions referring to the omitted or broken portion are not shown to scale. In Fig. 3 is shown the member shown in Fig. 2, but shortened so as to occupy about one-half the space. An opening is usually left, as at *c*, Fig. 3, and lines *d*, *e* are drawn to indicate that part of the member has been omitted. The cutting lines are sometimes straight, as *d* and *e*, Fig. 3 (*a*), and sometimes irregular, as *f* and *g*, Fig. 3 (*b*). When a member is cut and shortened, as just explained, its total length (13 feet 3 inches in this case) is written on a dimension line between its ends, although this line does not represent that length to scale.

The total length of a member is frequently called the **over-all dimension**, or the **length over all** of the member. This dimension should always be given.

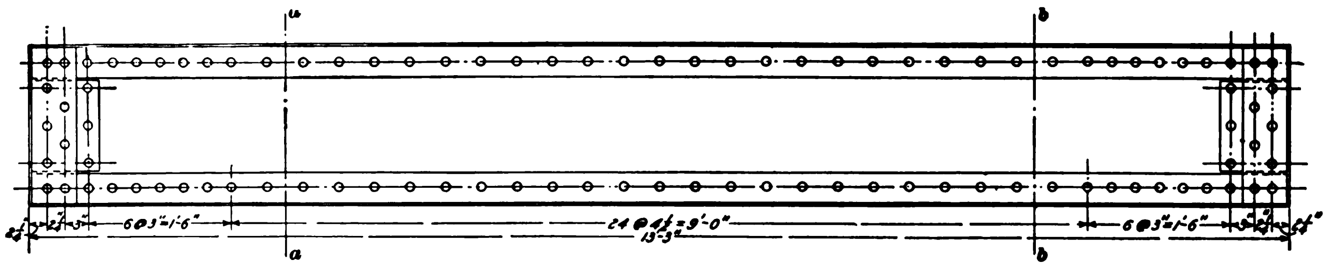


FIG. 2

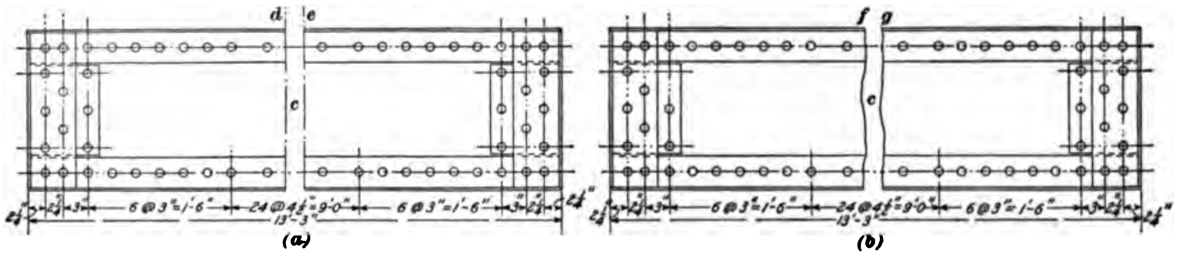
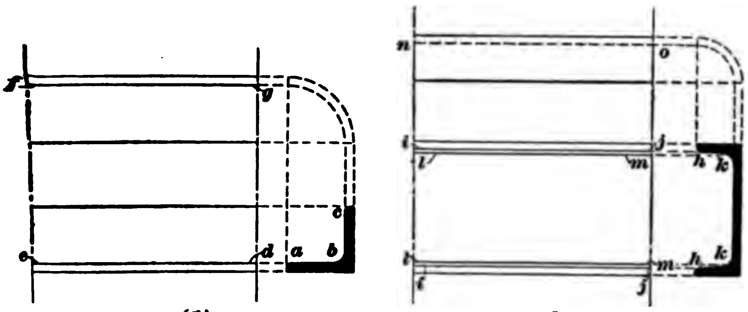


FIG. 3

7. Representation of Structural Shapes by Lines. The usual methods of representing structural shapes in cross-section were explained and illustrated in *Introduction to Construction Drawing*. These shapes are represented in plan and elevation as shown in Fig. 4, the curved corners



(a) (b)

FIG. 4

being represented by single lines at the points where the surfaces joined by the curves would intersect. For example, the curved corners at *a* and *b*, Fig. 4 (*a*), are represented in the elevation by the line *de*, and the curved surfaces *b* and *c* by the line *fg* in the plan. Similarly, in Fig. 4 (*b*), the curved corners at *h h* are represented in the elevation by the lines *i j*; those at *k k*, by the lines *l m* in the elevation and the line *n o* in the plan. When channels and I beams are drawn to a small scale, the lines *i j* and *l m* are very close together, and it is customary to draw but one line to represent both corners, this line being about midway between the two, as shown in Fig. 5.

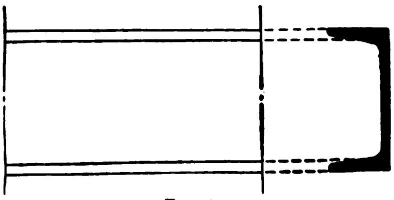


FIG. 5

8. Cross-Sections of Thin Shapes.—When shapes appear very thin on drawings, they are not section-lined or cross-latched as explained in *Introduction to Construction Drawing*, but are filled in solid black, as shown in Figs. 4 and 5. When two surfaces are in contact, a narrow space is

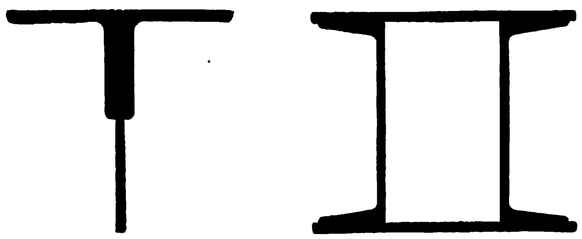


FIG. 6

CONVENTIONAL SIGNS FOR STRUCTURAL RIVETS

SHOP FIELD

Two full heads . . . .

Countersunk inside and chipped . . . . .

Countersunk outside and chipped . . . .

Countersunk both sides and chipped . . .

INSIDE OUTSIDE BOTH SIDES

Flatten to $\frac{1}{8}$ inch high or countersunk and not chipped . . . . .

Flatten to $\frac{1}{4}$ inch high . . . . . .

Flatten to $\frac{3}{8}$ inch high . . . . .

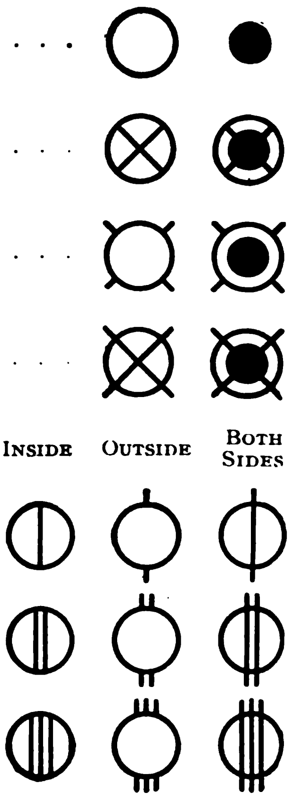


FIG. 7

left between them, as shown in Fig. 6. This method of showing cross-sections is employed on almost all drawings made to a scale of 1 inch to the foot or to a smaller scale.

9. Conventional Signs for Rivets.—The two conventional signs for rivets are explained in *Bridge Members and Details*, Part 1, and illustrated in *Bridge Tables*. The Osborne standard is believed to be the simpler and plainer, and will be used in the drawing plates of this Section. For convenience, the conventional signs are repeated in Fig. 7. In riveted members, rivets are shown only in those views in which they appear in end elevation, except where but one view of the member is shown, in which case some of the rivets may be shown in side elevation, as at *a*, *a*, Fig. 8.

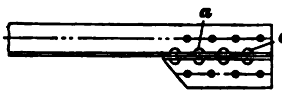


FIG. 8

10. Eyebar Heads.—Usually, the sizes of heads of eyebars are not given on bridge drawings, but they are drawn to scale. For this purpose, the diameter of the circular head is taken from a table, and the head is laid out as explained in *Introduction to Construction Drawing*. The radius of the curves that join the outline of the head with the sides of the bar is usually equal to twice that of the circular head, as illustrated in Fig. 9.

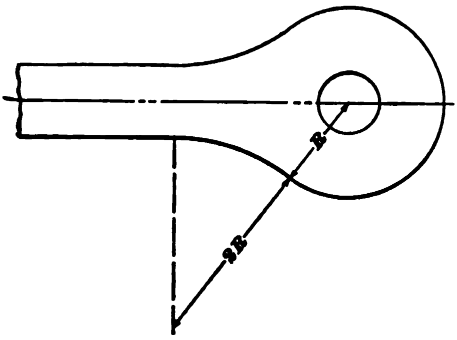


FIG. 9

11. Pin Nuts. The long diameter of each pin nut can be taken from a table. The outlines of these nuts are drawn by inscribing hexagons in circles having diameters equal to the long diameters of the respective nuts. The thicknesses and other necessary dimensions are also taken from tables.

12. Numbering Drawings.—There are various systems in use for numbering detail and working drawings. In some offices, each drawing is given a separate number, as was done in *Introduction to Construction Drawing* and *Construction Drawing*. In other offices, each "job" or contract is numbered, and all drawings relating to that contract are given the same number. When this system is followed, there is usually given a secondary set of numbers indicating the number of each drawing and the number of drawings that relate to the contract. The form used in some offices is as follows:

Contract 976
Sheet 3 of 6 sheets

and in others, as follows:

Contract 976: (3) *of* (6)

In each case, the notation indicates that the drawing is one of a set of drawings that relate to contract 976, that it is drawing number 3 of such set, and that the complete set consists of six sheets or drawings. The former system will be used in the plates in this Section.

DRAWING PLATES

GENERAL CONSIDERATIONS

13. The drawing in this Section consists of four bridge plates showing all the joints on one side of the center of the pin-connected truss treated in *Design of a Highway Truss Bridge*, Parts 1 and 2. The details of the ends of all the members that connect at the joints are also given, together with the side elevation of an intermediate floorbeam and sidewalk bracket. The student is advised not to start the drawing of these plates until after he has completed the two Sections mentioned.

14. In making a drawing of a bridge of this kind, it is the usual practice to lay out on a large sheet the skeleton drawing of one-half of the truss to a scale of $\frac{1}{4}$, $\frac{3}{8}$, or $\frac{1}{2}$ inch to the foot. This gives the directions of the members that meet at each joint, and shows the joints and members in their relative positions. The appearance of the drawing when the skeleton outline has been drawn is shown in Fig. 10, in which $fghi$ is the border of the sheet, and $aBEe$ the outline of one-half of the truss. The details of the ends of the members are then drawn at the joints, and the members broken between the joints to indicate that portions are omitted. The details of the members are usually drawn to a scale of $\frac{3}{4}$ or 1 inch to the foot. Each office or drafting room has a rule as to the scale to be used. In the following drawing plates, the distances between the joints are drawn to a scale of $\frac{3}{8}$ inch to the foot, and the details of the ends of the members are drawn to a scale of 1 inch to the foot.

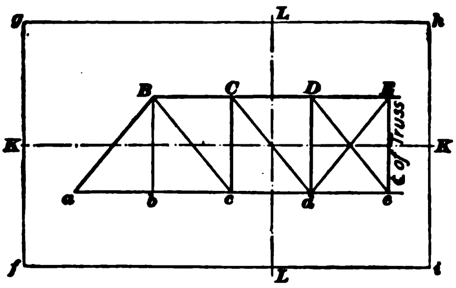


Fig. 10

In explaining the layouts of the following plates, some of the dimensions given in the text refer to sizes of parts and distances on the members, which are to be laid out to the scale of 1 inch to the foot. Other dimensions, such as those referring to the location of views on the plates are to be laid out full size on the drawings. There will be no difficulty in recognizing which dimensions are to be scaled and which are to be laid out full size.

15. On account of the difficulty in handling and mailing, the Schools have found it advisable to limit the size of drawing plates to 13 in. × 17 in. inside the border lines. It is impossible to draw an entire half truss on a sheet of this size without using an inconveniently small scale; hence, to conform to the size of the plate and the usual scale, the truss is shown on four plates. The sheet shown in Fig. 10

may be considered to be divided by the lines *KK* and *LL*, and the part of the truss in each of the four corners drawn on a separate sheet.

16. Erection Diagram.—When a truss is too large to be conveniently shown on a sheet, as just described, each member is drawn separately, and in the lower right-hand corner of the sheet is placed a skeleton drawing of the truss

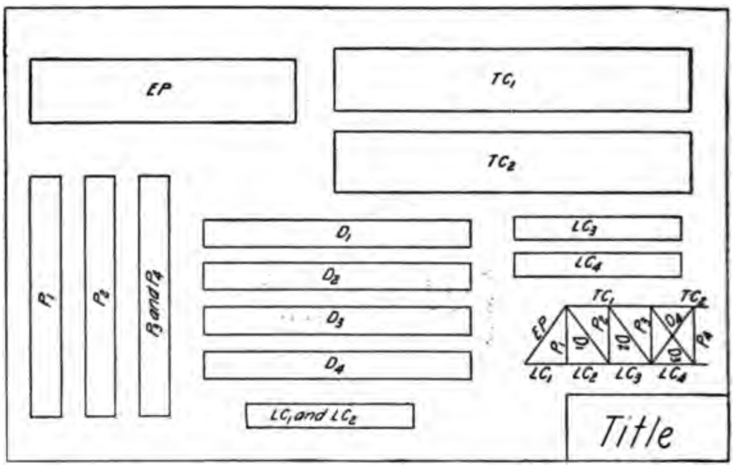
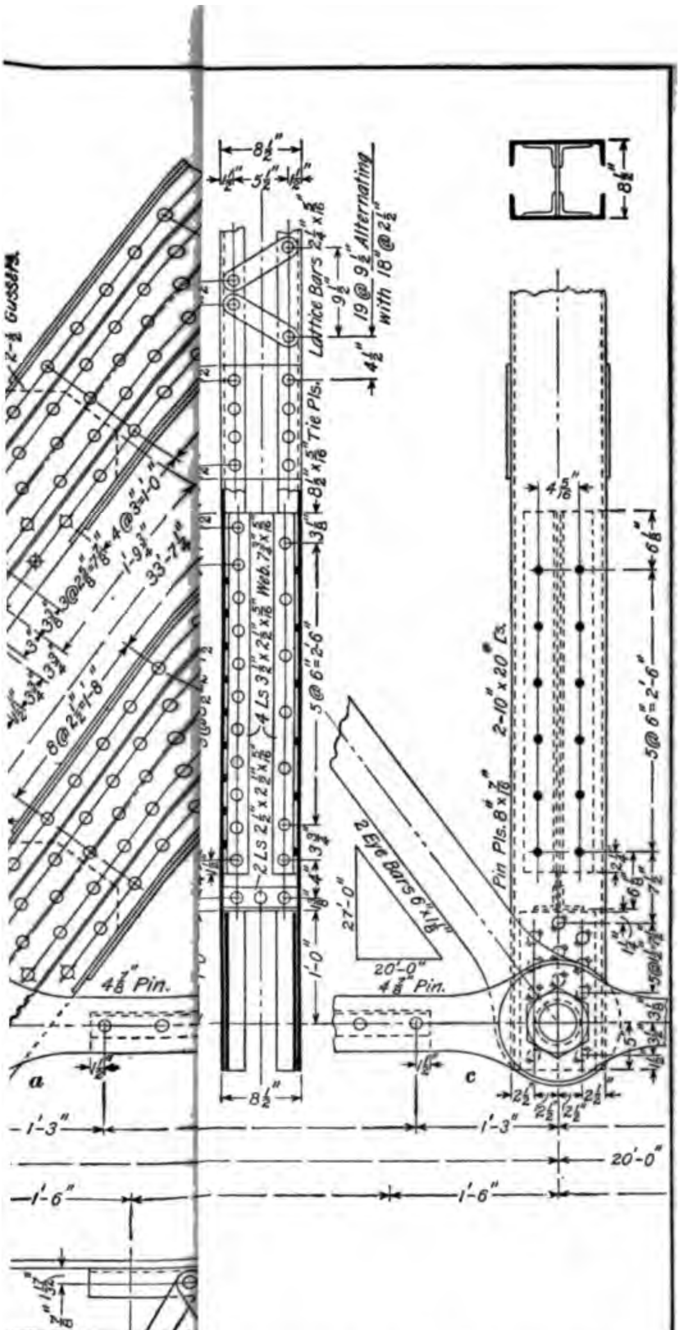


FIG. 11

to a very small scale. Each member is given a letter and number so as to show its position in the assembled truss. The general arrangement of the views on such a drawing is illustrated in Fig. 11. In most cases, however, more than one sheet is required in order to show all the members.

17. General Directions.—The plates will all be 13 in. × 17 in. inside the border lines, with the views arranged as explained in detail in the following articles. A space of $1\frac{1}{2}$ in. × 4 in. is reserved in each plate for the title, but it is not always possible to locate the titles at the lower right-hand corners of the sheets. They are located either on the lower or on the right-hand border line.

All the information necessary for the drawing of the plates is given in detail in the following articles. When the distance



HIGHWAY BRIDGE DETAILS

SHEET I OF 4 SHEETS

DRAWN BY----------------

DATE----------------

107

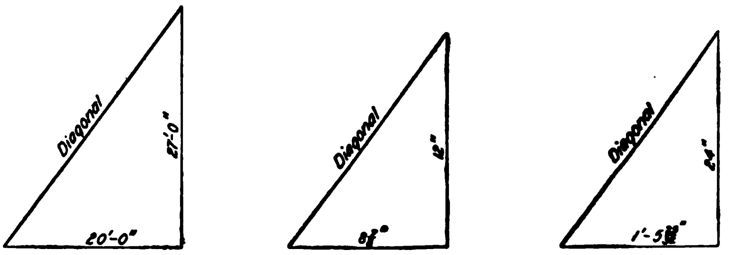
from the edge of a piece to the last rivet is not given on the plate, it may in general be taken as $1\frac{1}{2}$ inches. In case any difficulty is experienced in finding the dimensions of a section in the text or on the plate, they can be found in *Bridge Tables*.

The heads of the shop rivets on the plates are $1\frac{1}{4}$ inches in diameter, and the holes for the field rivets are $\frac{13}{16}$ inch in diameter. The letters in the title are $\frac{1}{4}$ inch in height, and all the other lettering on the sheets is $\frac{3}{32}$ inch in height, made as described in *Introduction to Construction Drawing*.

DRAWING PLATE 107, TITLE: HIGHWAY-BRIDGE DETAILS

18. This plate shows the elevation of the joint at the left end of the bottom chord and two joints to the right of it. In addition, it shows side elevations of the two verticals, cross-sections of the two verticals and end post, and a plan of the bottom chord members. The border lines enclose an area of 13 in. × 17 in.

19. Center Lines of Members.—The center lines of the members are drawn first. The center line of the lower chord in elevation is drawn parallel to and $8\frac{1}{2}$ inches from the top border line. The center of the end joint is then located on this center line 1 inch from the left border line, and the centers of the other joints are located from the first by laying off two distances of 20 feet each to the scale of $\frac{3}{8}$ inch to the foot. The joints will be referred to by the letters given in Fig. 10, the three joints on this plate being *a*, *b*, and *c*. Next, draw vertical lines through *a*, *b*, and *c*, to represent the centers of the chair at *a* and the verticals at *b* and *c*. Next, lay out the center line of the end post from *a*, and the center line of the diagonal from *c*. There are various methods of specifying the slope of a diagonal line on a bridge drawing; the most common is to give the coordinates of a point on the line with reference to the joint from which it starts, or to give the lengths of the sides of a right triangle of which the diagonal or part of it is the hypotenuse. See Fig. 12. The slope of a line, when given in this way, is called the **skew**. In Fig. 12 (*a*), the total distance passed over vertically and horizontally by a diagonal of the truss under consideration is given; this method is convenient and well adapted to detail drawings. For shop drawings, the method shown in Fig. 12 (*b*) is preferable; this gives the distance passed



(a) (b) (c)

Fig. 12

horizontally in 1 foot vertical. It is preferred by some to give the skew in 2 feet instead of in 1, since the workmen use steel squares 2 feet on a side. The skew in 2 feet, in this case, is $\frac{20}{27} \times 24 = 17\frac{21}{27}$ inches. Since, in this case, the skew is given in terms of the total distances, it is best to measure along $a\,b$ and $b\,c$, Fig. 13, and lay off $a\,a'$ and $c\,c'$, each equal to 20 feet, to a convenient scale, say $\frac{1}{8}$ inch to the

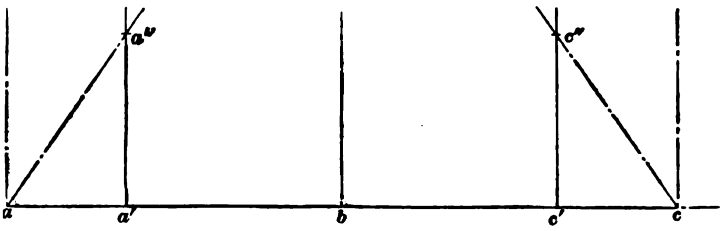


Fig. 13

foot. Then, erect perpendiculars at a' and c' and lay off $a'\,a''$ and $c'\,c''$, equal to 27 feet, to the same scale as $a\,a'$ and $c\,c'$. Then, $a\,a''$ and $c\,c''$ are the center lines of the inclined members. When, on the following plates, the right triangle giving the skew is drawn, the long leg of the right angle will in every case be made 1 inch in length.

20. Joint *a*.—Next, lay off the width of the eyebar that runs from *a* to the right, 6 inches wide, one-half above and one-half below the center line, and break it off about $4\frac{3}{8}$ inches from *a*. Now, lay off the top and bottom of the angle that shows dotted inside the eyebar; this leg is $2\frac{1}{2}$ inches wide, $1\frac{3}{8}$ inches of which is above the center line. The left end of the angle is 1 foot $1\frac{1}{2}$ inches and the first rivet is 1 foot 3 inches from *a*; the rivets are 6 inches apart. It is not necessary to make circles for the rivets on the pencil drawings, but simply to indicate their centers by short lines at right angles to the gauge lines.

The head of the eyebar can now be drawn. In *Bridge Tables*, the radius of this head is given as $6\frac{3}{4}$ inches. Then, the radius of the curves joining the head to the straight-portion is $13\frac{1}{2}$ inches. The long diameter of the nut is $7\frac{1}{2}$ inches; the diameter of the pin is $4\frac{7}{8}$ inches; and the diameter of the threaded end of the pin is 4 inches. These can now be drawn.

21. Next, lay out the outlines of the end post, breaking it off about $6\frac{1}{4}$ inches above the pin. First, lay off the top and bottom lines $7\frac{3}{4}$ and $7\frac{1}{2}$ inches, respectively, from the center line, and draw them parallel to it. Next, lay out the thicknesses of the outstanding legs, and draw the inside lines of the angles. Next, measure $7\frac{1}{4}$ inches below the center of the pin, and draw the end line of the end post to its intersection with the center line. Then, draw the other line at the end of the post, making the same angle with the center line as the first line. Measure in $3\frac{1}{2}$ inches from each side of the post to locate the edges of the flange angles, and draw them parallel to the center line. Next, draw two lines very close to the edges of the angles and parallel to them, to represent the edges of the 8-inch side plate. Now, lay off the gauge lines of the rivets, and put in the rivets according to the dimensions given in the drawing. The ends of the tie-plates that appear on the sides of the end post can be located by measuring from the nearest rivets.

22. Measure up 4 inches on a vertical through *a*, and draw the end of the end post as shown on the drawing plate, breaking it off 4 inches from the center of the pinhole. This is shown by itself to avoid confusion with the pin nut and eyebar head. The top of the chair is 1 foot 8 inches above the center of the pin, and 9 inches wide. The outside of the gusset at the left is straight from the outer corner of the flange angle of the end post to the outer corner of the chair. The angles of the diaphragm inside the gussets are $\frac{5}{16}$ inch apart, and have the 3-inch leg in view. The other dimensions can be scaled as given. Some of the rivets in the end post are countersunk on one or both sides to give room for the eyebar head on the outside and the pedestal on the inside.

23. Next, locate the center of the cross-section of the end post $8\frac{3}{8}$ inches (on the inclined center line) from the center of the pin, and lay out, according to the dimensions given, first the webs 15 in. × $\frac{7}{16}$ in., then the flange angles $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{1}{2}$ in., and then the side plates 8 in. × $\frac{1}{2}$ in.

24. Joint *b*.—Now, draw the eyebars, eyebar heads, pin and pin nut, and angles inside the eyebars at the joint *b*, in the same way as for joint *a*; the dimensions are the same. Next, draw the vertical lines representing the angles in the hip vertical, starting them $2\frac{5}{8}$ inches (to scale) above the center of the pin and breaking them off $6\frac{1}{2}$ inches above. The adjacent backs of the angles are $\frac{5}{16}$ inch apart; the outstanding legs are $7\frac{5}{16}$ inches apart. The gauge lines for the rivets and rivet holes are $4\frac{5}{16}$ inches apart, $2\frac{5}{32}$ inches on each side of the center. The rivet spacing is given at the left of the member. The pin plate at the lower end of this vertical extends 5 inches below the pin, and is 10 inches wide at the bottom. The top of the pin plate is $10\frac{5}{8}$ inches above the center of the pin. Next, put in the cross-section of this member, locating its center $7\frac{1}{2}$ inches above the center of the pin, and make the angles $3\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × $\frac{5}{16}$ in.

25. Next, draw the side view of this vertical, locating its center line $2\frac{1}{8}$ inches from that of the other view, and breaking it off about 7 inches above the center line of the bottom chord. This view is $7\frac{1}{4}$ inches wide; the legs of the angles are 2¼ inches wide; and the gauge lines of the rivets are 4½ inches apart. At the lower end of this member, the angles are connected by a web-plate, which extends up to 4 feet 10 inches above the center of the lower chord, and is connected to the angles by means of rivets spaced as shown. Near the bottom of the member, a short horizontal angle 2½ inches wide is riveted to the inside of the member. Above the web-plate, the angles are connected by latticing. This latticing is drawn by first locating the centers of the rivets in the ends of the lattice bars from the dimensions given, then drawing the center lines of the lattice bars, then drawing a semicircle 2¼ inches in diameter at the end of each bar and connecting these semicircles by lines parallel to the center lines and 1⅛ inches from them on each side.

26. The outstanding legs of the angles are blacked in at intervals in this view to indicate that there are rivet holes in them. Those at the left of the view can be located by projecting across from the front view shown directly over the pin. In the right-hand side, the lower rivet is 1 foot 7⅛ inches above the center line of the lower chord, and above this there are eight spaces of 3½ inches each. It is not customary to show rivet holes in this way wherever they occur, but only when it is desired to emphasize some special feature. In the present case, this view shows that the holes are not spaced the same on both sides of the truss.

27. Joint *c*.—The details of the members meeting at the joint *c* are next drawn. The eyebar that starts toward joint *b* is the same as the eyebars at that joint. The eyebar that extends toward the right is 5 inches in depth, and the circular portion of the head is 6¼ inches in radius. The eyebar that is shown as the diagonal is 6 inches wide, and the circular part of the head has a radius of 6¾ inches. The pin and pin nut at this joint are the same as at joints *a* and *b*.

28. The vertical shown in this view is composed of two channels 10 inches in width, extending from 5 inches (to scale) below the center of the lower chord to about 6½ inches above it. The tie-plates that show on the sides of this view near the top are located by projecting across from the side view, which will be explained presently. The inside lines of the angles that are shown in dotted lines are $\frac{5}{16}$ inch apart, and the outer edges are $7\frac{5}{16}$ inches apart. The rivet holes in these two angles are located on gauge lines $4\frac{5}{16}$ inches apart, their distances from the center of the bottom chord being shown on the right of the view. The top and bottom lines of these angles can also be located from the dimensions, and also the angles seen in end view below the angles just referred to. Next, put in the outline of the pin plate, 8 inches wide, at the bottom of the vertical, and then the gauge lines of the rivets, as given below the pin. The rivets connecting the pin plate to the channel are next located. These rivets are shown dotted where they are hidden by the eyebar heads; they are also shown flattened to a height of $\frac{3}{8}$ inch on the outside of the member, to allow the eyebars to lie closer to the vertical.

The cross-section of this member can now be drawn above the front elevation; the center of the cross-section is $7\frac{1}{2}$ inches above the center line of the lower chord. The channels are $8\frac{1}{2}$ inches back to back, and the flanges are $2\frac{3}{4}$ inches in width. On the inside of the section is shown the outline of the diaphragm that is inserted between the channels near the lower end. The web of this diaphragm is $\frac{5}{16}$ inch thick. The legs of the angles next to the web are $2\frac{1}{2}$ inches in width, and the legs in contact with the channels are $3\frac{1}{2}$ inches in width.

29. The side elevation of this vertical is next drawn, the center line being located parallel to and $2\frac{5}{8}$ inches to the left of the center line of the other view. It is customary to show side views on the right of front elevations; but, in this case, there being no available room on the right, the side view is shown on the left. The top of the side view is broken off 7 inches above the center line of the lower chord. The lattice bars near the top of the view are drawn in the same way as for the hip vertical. The tie-plate is 1 foot in height and located, as shown, by the dimension lines at the left of the side view. Below the tie-plate there are no rivets in the flanges of the channels, so they are assumed to be cut away, thus showing the webs of the channels in section. This is for the purpose of showing the detail of the diaphragm more clearly; if the flanges were not cut away, the diaphragm would be hidden behind them to such an extent as to obscure the details. The legs of the diaphragm angles that show in this view are $2\frac{1}{2}$ inches in width. The gauge lines of the rivets in the diaphragm are $2\frac{1}{2}$ inches on each side of the center line, and the rivets are located on them as shown by the dimensions at each side. At the lower end of this view, the pin plates are shown in cross-section.

30. Plan of Bottom Chord.—The plan of the bottom chord is shown below the elevation. The center line of the plan is parallel to and $1\frac{3}{4}$ inches above the lower border line. At joint *a*, only the ends of the eyebars are shown; it is assumed that the end post and pin have been removed. At joint *b*, the hip vertical is shown in cross-section. This view is broken off $3\frac{3}{8}$ inches from *a* and 4 inches from *b*. The eyebars are not parallel to the center line, but diverge from it. The ends, however, may be shown parallel as far as the breaks, and then offset, as shown on the drawing plate. This better illustrates the fact that the eyebars diverge, and shows the direction in which they diverge. The two eyebars that form each member are connected by lattice bars. When the divergence is slight, the gauge lines of the rivets that connect the lattice bars to the outstanding legs of the angles riveted to the insides of the eyebars are made parallel to each other and to the center line. This makes it easy to get the lattice bars all the same length, and is accomplished by skewing the gauge lines on the angles. In this case, the gauge line of each angle is $1\frac{17}{32}$ inches from the back of the angle at the left end and $1\frac{15}{16}$ inches from the back of the angle at the right end; the difference of $\frac{13}{32}$ inch does not practically affect the strength of the latticing.

The lattice bars can be put in as follows: First, draw the gauge lines of the rivets in the angles as shown; then, locate the rivets on these lines, and draw the center lines of the lattice bars. Then, draw semicircles $2\frac{1}{8}$ inches in diameter at each rivet, and draw two lines $2\frac{1}{8}$ inches apart and parallel to each center line to represent the sides of the lattice bars. The appearance of the two eyebars latticed in this way is shown in Fig. 14 to an exaggerated scale. This is the most satisfactory method of arranging the latticing between diverging eyebars. Other methods require the bars to be of

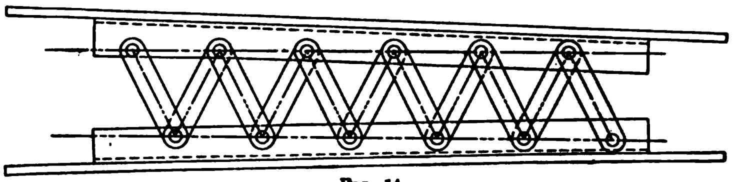


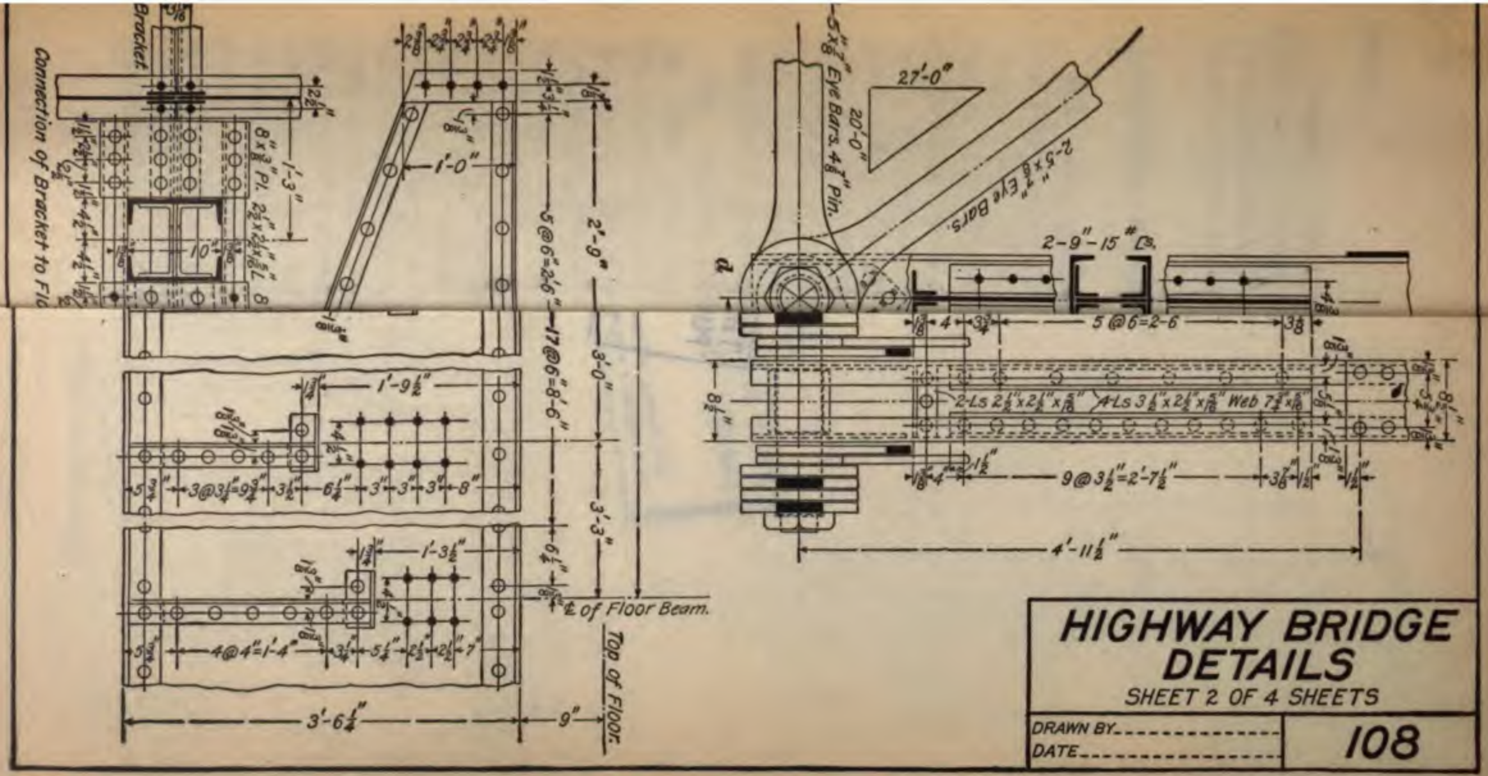
FIG. 14

different lengths, which involves much more computation on the part of the draftsman and more trouble in the shop. When the divergence is greater than shown on the drawing plate, the same method of connecting the lattice bars can be used, except that, in such a case, it is necessary to use wider angles on the eyebars.

31. The nuts shown on the pin at the joint *b* are $1\frac{1}{4}$ inches thick and are recessed $\frac{1}{2}$ inch; the threaded ends of the pin are $1\frac{7}{8}$ inches in length at each end. Between the inside eyebars and the pin plates of the hip vertical, and also between the pin plates, the pin is shown enclosed in thin rings or cylinders. These are usually made of cast iron about $\frac{1}{2}$ or $\frac{3}{4}$ inch in thickness, and of lengths just sufficient to fill the spaces between the members on the pin; they serve the purpose of preventing the members that connect to the pin from moving sidewise.

Wherever there are spaces in pins to which members do not connect, they should be filled with fillers. The outside diameter of the fillers at the joint *b* is $6\frac{1}{2}$ inches, and the inside diameter is 5 inches.

32. On all bridge drawings, it is customary to refer the center line of the chords to some plane of reference, such as the top of the floorbeam, the top of the floor, or some other



Bracket.
Connection of Bracket to Fl
8"x3/8" Pl.
2-9"-15# [s.
2-5"x7/8" Eye Bars.
Eye Bars. 4 7/8" Pin.
27'-0"
20'-0"
5@6"=2'-6"
17@6"=8'-6"
2'-9"
3'-0"
3'-3"
4'-11 1/2"
9@3 1/2"=2'-7 1/2"
3'-6 1/4"
9"
℄ of Floor Beam.
Top of Floor.
HIGHWAY BRIDGE
DETAILS
SHEET 2 OF 4 SHEETS
DRAWN BY
DATE
108

convenient elevation. In the present case, the top of the floor at the center of the bridge is used, and is shown 5 feet 3¼ inches above the center line of the lower chord. This line serves to locate different parts of the bridge.

DRAWING PLATE 108, TITLE: HIGHWAY-BRIDGE DETAILS

33. This plate shows the elevation of the center joint of the lower chord and the joint next to the left of it. Referring to Fig. 10, it is seen that these joints are lettered *d* and *e*. Between the two joints is shown the elevation of the end of each vertical (they are the same), to avoid confusion with the eyebar heads. At the right of the elevation of joint *e* is shown a cross-section through the eyebars, and a side elevation of the verticals.

In addition to the members of the truss, there is shown on this plate a side elevation of a sidewalk bracket, and a side elevation of one-half of an intermediate floorbeam.

34. Center Lines of Members.—The center line of the lower chord is first drawn parallel to and 6 inches from the top border line. Next, the center lines of the verticals are put in; that at *d* is parallel to and 3 inches from the left border line; that at *e* is parallel to that at *d* and 20 feet from it, to the scale of ⅜ inch to the foot. The center lines of the inclined members are then laid out as explained in Art. **19.** Having put in the center lines at both joints, draw the side lines of the straight portions of the eyebars at the joint *d*, and then put in the outlines of the heads. The circular portions of the heads of the bottom chord bars and of the diagonal at the left have a radius of 6¼ inches; the radius of the curves connecting the heads to the straight portions is 12½ inches. The circular portion of the head of the diagonal on the right has a radius of 4½ inches; the radius of the other curves is 9 inches. The pin and pin nut are the same as shown on the preceding plate; that is, the long diameter of the nut is 7½ inches, the diameter of the pin is 4⅞ inches, and the diameter of the threaded end is 4 inches.

35. The vertical can now be drawn. This view is 9 inches wide, and is cut off $5\frac{3}{8}$ inches above the center line of the bottom chord; a space of 1 inch is left open for the insertion of the cross-section, the center of which is $2\frac{3}{4}$ inches above the center of the pin.

In spite of this intermediate break, this part of the vertical is neither shortened nor lengthened. The spacing at the right, 8 @ $3\frac{1}{2}$ inches = 2 feet 4 inches, is laid out to the scale of 1 inch to the foot in the same way as though the member were not cut.

In the front elevation of this vertical, it is assumed that the front side has been cut away and removed, and the diaphragm between the channels is shown in cross-section. This method is frequently followed when the two faces of a member are not alike, and serves the purpose of showing the detail of the opposite side of the member. In this case, this view shows the location and spacing of the rivet holes in the opposite side of the member. The dimensions are laid out and the lines drawn in the same way as explained for the vertical at *c* in the preceding plate.

36. Joint *e*.—The members at *e*, the center joint of the truss, can next be drawn. The eyebars, eyebar heads, pin nut, and pin are the same here as at the preceding joint, except that one inclined eyebar is 3 inches wide instead of 5 inches.

The vertical is cut off $5\frac{3}{8}$ inches above the pin, and a space 1 inch wide is left open for the cross-section, the center of which is $2\frac{3}{4}$ inches above the pin. This vertical is the same as that at *d*, but a different view is shown; in this case, the front elevation is shown, and no part of the front face is assumed to be cut away. No difficulty should be experienced in drawing this view.

37. The triangular upper part of the pin plates at *d* and *e* are shown dotted, and the rivets also are shown in connection with the remainder of the members at the joints. To avoid confusion, however, a separate drawing is made, showing the rivet spacing in this pin plate. For this purpose, the center line of the vertical is located 6 inches from the left border line, the center of the pinhole is located $1\frac{1}{2}$ inches above the center line of the lower chord, and the view is broken off 4 inches above this center line. The pin plate, which is 7 inches wide, and the rivets are then located according to the dimensions given. The rivets are shown flattened to $\frac{3}{8}$ inch in height on the outside of the member.

38. Having completed the drawing of the front elevation of these two joints, the side elevation of the verticals, shown at the right end of the plate, can be drawn. The two verticals are the same, so that one view serves for both. The vertical center line of this view is $3\frac{1}{4}$ inches from the right border line. This view is similar to the side view at the joint *c* in the preceding plate, except that the flanges of the channels are not cut away, and the pin at the lower end of the vertical, together with the eyebars that connect to it, are shown. That part of the diaphragm, as well as the rivets that are hidden by the flanges of the channels, is shown in dotted lines. The vertical is broken off $5\frac{5}{8}$ inches above the center of the pin. The student should have no difficulty in drawing this view of the vertical by following the dimensions given on the drawing.

39. When the vertical has been drawn, the eyebars can be put in. First, leave a space of $\frac{7}{16}$ inch on each side of the vertical to allow for the flattened rivet heads and clearance; then, lay out a width of $\frac{13}{16}$ inch; then, one of $\frac{1}{16}$ inch, then, one of $\frac{13}{16}$ inch on each side, to allow for the thicknesses of the diagonal eyebars and the clearance between them. The bottoms of these four bars are shown $4\frac{1}{2}$ inches below the center of the chord; the tops are 1 and $1\frac{1}{2}$ inches, respectively, above the center of the chord. The two inside bars are shown in cross-section. Next, lay off four bars, $1\frac{1}{16}$ inch in width on each side of the vertical, leaving a space $\frac{1}{16}$ inch wide between each two bars. The heads of these bars are all shown $12\frac{1}{2}$ inches in height, and two of the bars on each side are shown in cross-section 5 inches in height. The pin, $4\frac{7}{8}$ inch in diameter, pin nuts, $1\frac{1}{4}$ inches thick with recesses

⅜ inch deep, and filling washer inside the vertical, 6½ inches in diameter, can now be drawn.

The line representing the top of the floor at the center of the bridge can now be put in.

40. Sidewalk Bracket and Floorbeam.—The views at the bottom of the drawing plate are side elevations of the intermediate sidewalk bracket and floorbeam shown in their relative positions. The center line of the truss is shown between the two views, and is located 8¼ inches from the left border line. The top of the bracket and floorbeam is horizontal, and is located 4⅜ inches above the bottom border line. The right line of the bracket is 4¼ inches from and parallel to the center line of the truss, and the bracket is 3 feet 6¼ inches deep at that point. At a point 7 feet to the left of the center line of the truss, the bracket is 12 inches deep. By plotting two points at these places, the bottom line of the bracket can be drawn. The left end can then be drawn parallel to the center line of the truss and 7 feet 3⅜ inches to the left of it. Next, draw the lines representing the inner surfaces of the outstanding legs of the flange angles, then the gauge lines of the rivets in the flange angles 1⅜ inches from the backs, and then the inner edges of the flange angles, 2½ inches from the backs. Then, draw the two stiffeners, making them 2½ inches wide, with the outstanding legs at the left, and the backs 1 foot 3 inches and 4 foot 3 inches, respectively, from the center line of the truss. The bottoms of these stiffeners are 1 foot 2¾ inches below the top line of the bracket.

Next, the connection angle can be drawn at the right end of the bracket. The leg that shows in this view is 3 inches wide, and the gauge line is 1¾ inches from the right side of the angle. When the outlines and gauge lines have all been drawn, the rivets and rivet holes can be located according to the dimensions given on the drawing plate. The outstanding leg of the connection angle at the right end of the bracket is blacked in at intervals, to represent the rivet holes for the rivets that connect this member to the vertical of the truss.

41. The space remaining on the drawing plate is too small to show every part of the floorbeam to scale, so it is necessary to break and shorten the beam. In this case, it is inadvisable to leave out one large portion, as then it would be necessary to omit one or more of the stringer connections, which it is very important to show. The shortening is accomplished by omitting a short section between each two consecutive stringer connections. The left end of the floor-beam is first drawn $4\frac{1}{4}$ inches from the center line of the truss; then, the bottom line of the bottom flange is drawn parallel to the top line and 3 feet $6\frac{1}{4}$ inches from it. Next, the centers of the stringer connections are put in, $1\frac{3}{4}$ inches, $3\frac{1}{8}$ inches, $4\frac{1}{2}$ inches, $5\frac{7}{8}$ inches, and $7\frac{1}{4}$ inches, respectively, from the center line of the truss. Half way between each two consecutive stringer connections, two irregular lines, about $\frac{1}{8}$ inch apart, are drawn vertically from the top to the bottom lines. An irregular line is also drawn $\frac{3}{4}$ inch from the right border line, to indicate the end of the view.

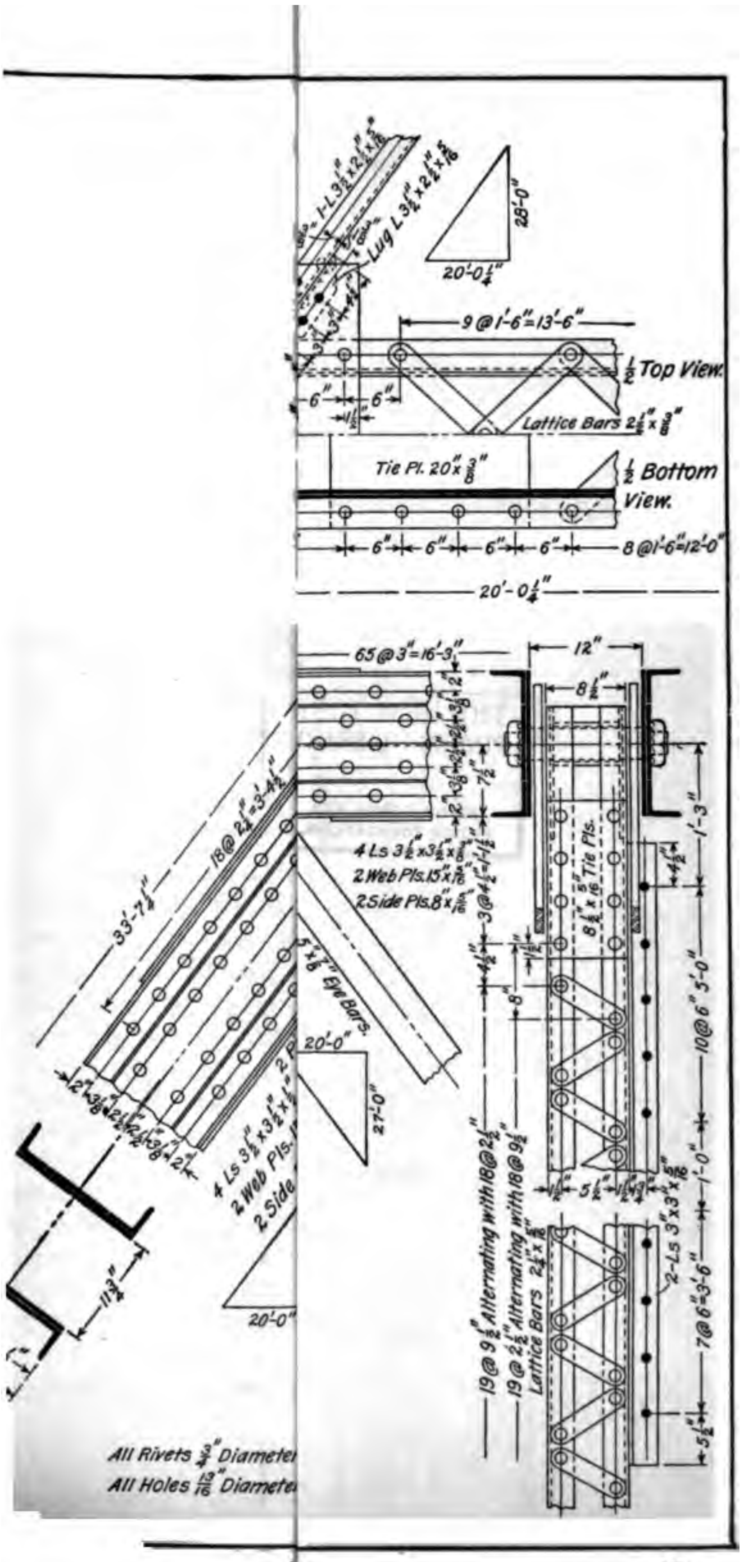
Next, the horizontal lines representing the inner surfaces, the gauge lines, and the inner edges of the top and bottom flange angles are drawn, and the flange rivets are located on the gauge lines according to the dimensions given at the top of the view. The connection angles at the left end of the floorbeam are next drawn $3\frac{1}{2}$ inches wide, with the gauge line 2 inches from the back and the top and bottom ends of the angles $\frac{1}{4}$ inch from the flange angles; the rivets are located on the gauge line according to the dimensions shown at the right of the connection angle.

Next, the open holes for the stringer connections are located, by the dimensions given, from the center lines of the connections and the top of the top flange. The shelf angles and stiffeners are next put in, the same procedure being followed for each of them. First, locate the top of the shelf angle by the given distance from the top of the top flange angles, and draw the shelf angle 3 inches deep and 6 inches long, one-half on each side of the center line. Next, draw in the stiffener, placing the back of the angle even with the center line of the connection, and measuring

$2\frac{1}{4}$ inches to the right to locate the right edge. Then, put in the gauge lines of the shelf and the stiffener angles, and locate the rivets on them according to the dimensions given in the drawing plate.

The detail of the connection of the bracket to the floorbeam, in the lower left corner, can now be drawn. The center line of the bracket and floorbeam is parallel to and $1\frac{1}{2}$ inches from the lower border line. The center line of the vertical is $2\frac{1}{2}$ inches from the left border line. The vertical is composed of two 10-inch channels with flanges $2\frac{3}{4}$ inches in width. The lines inside the vertical form a top view of the diaphragm shown in cross-section in the upper part of the plate. The top flange of the bracket is $5\frac{5}{16}$ inches in width and is broken off $\frac{5}{8}$ inch from the left border line. The top flange of the floorbeam is $8\frac{3}{8}$ inches in width and is broken off $4\frac{5}{8}$ inches from the left border line. When the top flanges have been put in, the connection plates can be drawn; they are 8 inches wide with one edge $\frac{1}{4}$ inch from the back of a channel on each side. The angles under the connection plates are $2\frac{1}{2}$ in. × $2\frac{1}{2}$ in. × $\frac{5}{16}$ in. and are placed with their vertical legs in close contact with the flanges of the channels. The longitudinal section of the sidewalk stringer is 1 foot 3 inches from the center of the vertical and is $1\frac{1}{8}$ inch above and below the center line of the bracket. The lower flange of the stringer is $4\frac{3}{8}$ inches wide. The splice plates shown in cross-section are 6 inches long. The rivets can now be located by means of the given dimensions.

42. The dimension lines and dimensions may be put in now on the whole drawing, or they may be put in on each figure as soon as the figure is done. The latter method is the better to follow on the pencil drawing, as the work is fresh in the draftsman's mind. In tracing the drawing, however, it is better to put in the dimension lines when each view is traced, and to put in the dimensions after all lines have been traced.


20'-0¼"
28'-0"
9 @ 1'-6" = 13'-6"
½ Top View.
Lattice Bars 2¼" x 3/8"
Tie Pl. 20" x 3/8"
½ Bottom View.
8 @ 1'-6" = 12'-0"
20'-0¼"
65 @ 3" = 16'-3"
12"
8½"
33'-7¼"
18 @ 2¼" = 3'-4½"
20'-0"
27'-0"
1'-3"
10 @ 6" = 5'-0"
1'-0"
7 @ 6" = 3'-6"
Lattice Bars 2¼" x 5/16"
All Rivets ¾" Diamete
All Holes 13/16" Diamete

DRAWING PLATE 109, TITLE: HIGHWAY-BRIDGE DETAILS

43. This plate shows the elevation of the hip joint and of the top chord joint next to the right of it. Cross-sections of the members and side views of the verticals are also shown. In addition, there is at the top of the plate a view, one-half plan, one-half section, of the top chord, showing the connections of the transverse strut and of the diagonals of the upper lateral truss to the top chord.

44. The elevation and side views will be explained first. Draw the center line of the top chord parallel to and $5\frac{7}{8}$ inches from the top border line. Draw the center line of the hip vertical parallel to and $4\frac{7}{8}$ inches from the left border line, and the center line of the next vertical parallel to it and 20 feet $\frac{1}{4}$ inch to the right of it, to a scale of $\frac{3}{8}$ inch to the foot. Draw the inclined center lines of the diagonals as explained in Art. **19.** By reference to Fig. 10, it is seen that the joint on the left is joint *B*, and that on the right is joint *C*. The elevation of joint *B* will be explained first.

45. When the center lines have been drawn, the top and bottom lines of the top chord can be drawn $7\frac{3}{4}$ and $7\frac{1}{2}$ inches, respectively, above and below the center line of the chord. Then, the lines representing the inner surfaces of the outstanding legs, the gauge lines of the rivets, and the inner edges of the top and bottom flange angles can be drawn. The top chord is cut off at the left end, $\frac{1}{8}$ inch from a line that bisects the angle between the center lines of the top chord and end post; it is broken off $3\frac{5}{8}$ inches to the right of the center of the pin. The intermediate gauge lines are next put in according to the dimensions at the right, and the rivets in this part of the chord put in position according to the dimensions at the top of this view. Some of the rivets are countersunk on the inside, to leave room for the pin plates on the inside of the end post, which project up into the top chord.

The pin plates can now be drawn. The longest plate fits between the edges of the flange angles, and is 8 inches wide; it extends $1\frac{1}{2}$ inches beyond the last two rivets in the web. The next two pin plates are 14 inches wide, and fit in between the outstanding legs of the flange angles. The right end of each plate can be located by counting the rivets enclosed in the plate. The left ends of the 8-inch and the inside 14-inch pin plates are even with the skew end of the chord; the outside pin plate extends out beyond the pin, the outline being as shown in Fig. 15. The upper edge of the pin plate is continued parallel to the center line BC as far as its intersection a with the bisector bc. The outer edge ad of the pin plate then continues parallel to the center line of the end post to the intersection of ad with the center gauge line of the chord. The other edge is then drawn from d to e, the latter being vertically below a. This view will be completed after the plan has been drawn.

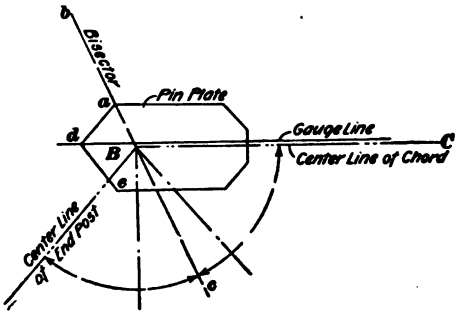


Fig. 15

46. Now, proceed in the same way for the end post as for the top chord just drawn, and break it off 4 inches from the center of the pin. First, draw the flange angles, then the 8-inch side plate, and then the gauge lines. Next, put in the rivets according to the dimensions, and then draw the pin plate, which is on the inside of the member. Some of these rivets (those shown in dotted lines) are countersunk on both the inside and the outside of the section, to allow room for the eyebar heads on the inside, and for the projecting pin plates of the top chord on the outside. The upper end of this pin plate is cut off in the same way as that on the top chord, as illustrated in Fig. 15. The portions of the tie-plates that show in this view can be located and drawn as they are shown on the drawing plate. Now, draw the cross-section of the end post, making its center line $5\frac{1}{2}$ inches from the center of the pin. In the cross-section, the clear distance between the webs is $11\frac{3}{4}$ inches, the webs are 15 in. × $\frac{7}{16}$ in., the side plates are 8 in. × $\frac{1}{2}$ in., and the flange angles are $3\frac{1}{2}$ in. × $3\frac{1}{2}$ in. × $\frac{1}{2}$ in.

47. Next, the eyebar that forms the diagonal can be drawn. The bar is 6 inches wide, one-half on each side of the center line, and the radius of the circular part of the head is $6\frac{3}{4}$ inches. The diameter of the pin is $4\frac{3}{4}$ inches, that of the screw end $3\frac{1}{2}$ inches, and that of the nut $6\frac{5}{8}$ inches. These can now be put in.

48. The hip vertical is drawn next. The pin or hanger plate should be drawn first. This is 10 inches wide from 7 inches above the pin to 8 inches below it, and then narrows to $7\frac{5}{16}$ inches at 1 foot $9\frac{1}{2}$ inches below the pin. The angles are drawn next; their backs are $\frac{5}{16}$ inch apart, and their outer edges are $7\frac{5}{16}$ inches apart. The upper ends of the angles are $2\frac{1}{2}$ inches below the center of the pin, and the break at the lower end is $4\frac{1}{4}$ inches below. The center of the cross-section is 5 inches below the center of the pin. Of the rivets at the upper end of the hip vertical, only those are shown that appear below the other members. The rivets that are hidden are omitted, as they would confuse the drawing if shown. In this case, the upper end of the hip vertical and the spacing of the rivets are shown in an additional view below and to the right of the joint. The center line of this view is $1\frac{1}{4}$ inches from the center line of the hip vertical in the other view, and the center of the pinhole is $3\frac{3}{4}$ inches below the center line of the top chord. The three upper rivets in this view are countersunk for the purpose of allowing the eyebars in the end diagonal to lie close to the hanger plates.

49. Next, draw the side elevation of the hip vertical shown just to the right of joint *B*. The center line of this view is $4\frac{1}{4}$ inches from the center of the pin at *B*; the view extends down to 5 inches below the center of the top chord. This view can be laid out very readily by the dimensions given.

50. Joint *C*.—At the joint *C*, first draw the irregular lines at the ends of the top chord 2 inches from the center of the pin at each end, and project all the necessary lines across from joint *B*. Now, put in the rivets according to the dimensions, and show the left end of the 8-inch side plate $10\frac{5}{8}$ inches from the center of the pin. The rivets around the pin are flattened to $\frac{3}{8}$ inch high on the inside, to give room for the eyebars. Next, draw the eyebars 5 inches wide, and the eyebar heads with a radius of $5\frac{3}{4}$ inches. The pin and nut can now be drawn. The diameter of the pin is $3\frac{1}{2}$ inches, that of the threaded end is $2\frac{1}{2}$ inches, and that of the nut is $5\frac{3}{16}$ inches.

51. Next, draw the vertical, making it 10 inches wide and cutting it off $3\frac{1}{4}$, $4\frac{3}{4}$, and $6\frac{3}{4}$ inches below the center line of the top chord. Draw the angles that show in dotted lines, locating the top $\frac{7}{8}$ inch and the bottom $6\frac{3}{8}$ inches below the center of the top chord. These angles are each 3 inches wide, and their backs are $\frac{5}{16}$ inch apart. Now, put in the rivets in this view according to the dimensions. Next, draw the cross-section, locating its lower line $4\frac{1}{2}$ inches below the center of the top chord. The flanges of these channels are $2\frac{3}{4}$ inches wide; the angles are each 3 in. × 3 in. × $\frac{5}{16}$ in.

52. The outline of the pin plate at the top of the vertical is shown, but the rivets are omitted from this view to avoid confusion. They are shown in a separate view, the center line of which is $1\frac{7}{8}$ inches to the left of the center of the vertical in the elevation. The center of the pinhole is 2 inches below the center of the top chord. The rivets are shown flattened to $\frac{3}{8}$ inch in height to provide room for the eyebars.

53. The side view of the vertical at *C* is next drawn on the right end of the sheet, and the eyebars and top chord are shown in cross-section in their relative positions. The center line of this view is located parallel to and $1\frac{1}{4}$ inches from the right border line. The cross-section is first drawn in the same way as for the end post (see Art. **46**). Then, the vertical is drawn $8\frac{1}{2}$ inches in width, and the tie-plate and lattice bars are drawn according to the dimensions.

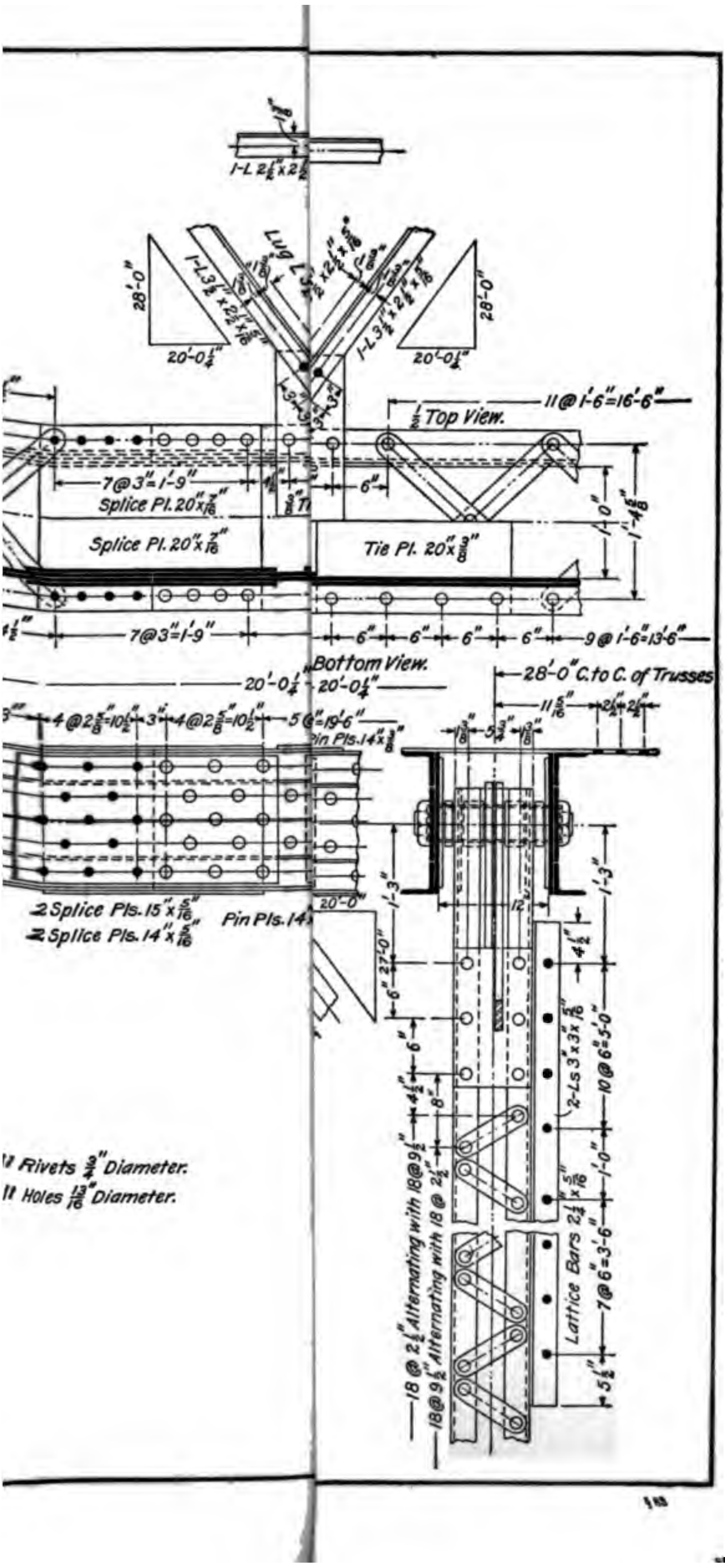
The gauge lines of the rivets in the flanges of the channels are scaled in position, and the center lines of the lattice bars are drawn as explained before. The lattice bars can then be drawn in. The view is broken $3\frac{3}{4}$, $4\frac{1}{4}$, and $6\frac{3}{4}$ inches below the center line of the top chord. The angles on the right side of this view are located by projecting the top and bottom across from the dotted lines shown in the front elevation; the rivet holes are then located according to the dimensions. These angles are for the connection of the transverse strut to the vertical.

The cross-section of the top chord can now be drawn. The pin nuts on the outside of this section are 1 inch in thickness with recesses $\frac{3}{8}$ inch deep, and the threaded ends of the pins project $\frac{1}{4}$ inch beyond them at each end. The eyebars forming the diagonal that connects at C are shown between the inside surfaces of the chord section and the outside surfaces of the vertical.

54. Plan of Top Chord.—The plan of the top chord is shown at the upper part of the plate. The center line of this view is drawn parallel to and $3\frac{1}{8}$ inches from the top border line. The centers of the joints are located by projecting up from the elevation. In the portion of the plan shown below the center line of the chord, it is assumed that the upper flange is removed. The web is first drawn in cross-section, $\frac{5}{16}$ inch thick, with the inner line 6 inches from the center line. Next, the 8-inch side plate at the joint C and the pin plates at the joint B are shown in section. Their ends are located by projecting up from the center line of the elevation. The plan is broken off at three places, each of which is $3\frac{5}{8}$ inches from the next joint. The line representing the outside surface of the vertical leg of the bottom flange angle is next drawn, and then the gauge line and the outer edge of the bottom flange angle. The rivets are located on the gauge line according to the dimensions given, and the tie-plates and lattice bars put in on this half of the plan. The ends of the tie-plates on the bottom flange in the elevation can now be located by projecting down from the plan.

55. The upper half of the plan is now drawn. It is best first to draw the gauge line of the rivets in the outstanding leg of the flange angle $8\frac{5}{16}$ inches from the center line, and to locate the rivets along this line according to the dimensions given on the plate. Then, the tie-plates and lattice bars can be drawn as shown, and afterwards the lines representing the web and vertical leg of the top flange angle can be drawn. By proceeding in this way for both joints, the proper parts of the lines representing the web can be shown full or dotted the first time they are drawn. The open holes in the plan near the joint *B* are for the connection of the portal at this joint.

56. At the joint *C* there is also shown the connection of the top flange of a transverse frame and of two diagonals of the upper lateral truss to the top flange of the top chord. For this purpose, the top tie-plate at this joint, which is 2 feet $9\frac{1}{4}$ inches in width, is extended out 1 foot $5\frac{11}{16}$ inches from the center line of the chord, and acts as a gusset or lateral connection plate. The center line of the transverse strut is located by projecting up from the elevation. The top flange angles of the transverse strut are $\frac{5}{16}$ inch apart, and the legs shown in this plan are each 3 inches wide. The angles are broken off $2\frac{3}{4}$ inches from the center line. Next, the center lines of the diagonals are put in according to the given skew, in the manner explained in Art. **19.** They are drawn from the intersection of the center lines of the chord and transverse strut. In the present case, each diagonal is composed of one angle, the leg shown in the plan being $2\frac{1}{2}$ inches in width. The angles are located with the backs on the center lines, and at the ends, small angles, called **lug angles**, help to transmit the stress from the lateral angle to the gusset. When there is no lug angle on the end of a lateral, the center of gravity of the lateral is usually made to coincide with the center line drawn on the sheet.



1-L 2½"x2½"
28'-0"
20'-0¼"
Top View.
11@1'-6"=16'-6"
7@3"=1'-9"
Splice Pl. 20"x7/16"
Splice Pl. 20"x7/16"
Tie Pl. 20"x3/8"
1'-0"
1'-4⅞"
7@3"=1'-9"
9@1'-6"=13'-6"
Bottom View.
20'-0¼"
20'-0¼"
28'-0" C. to C. of Trusses
4@2⅝"=10½"
4@2⅝"=10½"
5@1'=19'-6"
2 Splice Pls. 15"x5/16"
2 Splice Pls. 14"x5/16"
20'-0"
1'-3"
1'-3"
2-Ls 3"x3"x5/16"
10@6"=5'-0"
Lattice Bars 2¼"x5/16"
7@6"=3'-6"
18@2½" Alternating with 18@9½"
18@9½" Alternating with 18@2½"
Rivets ¾" Diameter.
Holes 13/16" Diameter.

When, as in this case, there is a lug angle at the end, the center of gravity of the lateral angle is also sometimes made to coincide with the center line as just described; but it is frequently arranged as shown in the plan, the center line passing between the lug angle and lateral angle and coinciding with the back of the latter throughout its length. The lateral angles are shown broken off about $\frac{3}{8}$ inch from the top border line.

57. The pins and top views of the verticals are frequently shown with the plan of the top chord. They are omitted here simply to avoid confusion and unnecessary work.

DRAWING PLATE 110, TITLE: HIGHWAY-BRIDGE DETAILS

58. In making a bridge drawing, it is customary to draw the elevation first and then the plan and lateral connections, as in the preceding plate. In drawing the present plate, however, since the student is familiar with the general arrangement of the parts, it will be most convenient for him to draw the plan first. In the plan on this plate, there is shown, in addition to the detail of the connection of the laterals to the top chord, the detail of the intersection of the two diagonals of the top lateral truss at the center of a panel, and also the angle that connects this intersection with the top flanges of the transverse frames near the center of the bridge.

The center line of the top chord is first drawn parallel to and $4\frac{1}{4}$ inches from the top border line. The panel point at D is then located $4\frac{3}{4}$ inches from the left border line, and that at E 20 feet $\frac{1}{4}$ inch from it, to the scale of $\frac{3}{8}$ inch to the foot. Vertical lines are drawn at these points to represent the center lines of the verticals. The cross-section of the web and pin plates and the bottom plan of the top chord are drawn in the same way as in the preceding plate. The splice plates shown in cross-section are 2 feet 3 inches long, and their centers are 2 feet $6\frac{1}{8}$ inches from the center of the pin at *D*. The pin plates on the inside of the chord at *D* and *E* are each 2 feet $3\frac{1}{4}$ inches long. The remainder of this plan can be drawn according to the dimensions. It is shown broken off $4\frac{1}{2}$ inches to the left and $3\frac{1}{2}$ inches to the right of the center of the joint at *D*, and $3\frac{1}{2}$ inches to the left and right of the center of the joint at *E*.

59. Next, the gauge line of the rivets in the top flange angle is located $8\frac{5}{16}$ inches above the center line, and the rivets are located along this gauge line according to the dimensions. Then, the tie-plates and lattice bars are put in as shown. The tie-plates are the same size as those described in Art. **56** in connection with the preceding plate. The lines representing the top flange angle and the side of the web are then put in, completing this part of the view.

60. Next, the center line of the struts that run from the intersections of the diagonals to the transverse frames is drawn parallel to and $\frac{7}{8}$ inch from the top border line. The gauge line of the angle is assumed to coincide with this center line; at the center of the panel, two lines are drawn at the proper skew to represent the center lines of the diagonals, and short lengths of these diagonals are drawn. Then, the ends of the longitudinal angles are located, and the gusset at the center of the panel is drawn according to the given dimensions. The center lines of the transverse struts have already been drawn, and the angles are spaced equally on each side of those lines, the angles being $\frac{5}{16}$ inch apart, and each being 3 inches in width. These angles can now be drawn, and broken off about $\frac{1}{8}$ inch from the top border line and about half way between the center line of the bridge and the center line of the top chord. The gussets at the centers of the transverse frames can now be drawn according to the dimensions. The laterals are then drawn, thus finishing the plan.

61. The center line of the top chord in elevation is parallel to and 7 inches from the top border line. The center lines of the verticals have already been located. The top chord can first be drawn according to the dimensions. Since this view is entirely similar to views of the top chord previously drawn, it is deemed unnecessary to explain it in detail. The views are broken $4\frac{1}{2}$ inches to the left and $3\frac{1}{4}$ inches to the right of the center of the pin at *D*, and $3\frac{1}{4}$ inches to the left and $1\frac{1}{2}$ inches to the right of the center of the pin at *E*. The rivets in the chord near the pin at *D* are shown countersunk on the inside to leave room for the eyebar heads. At *E*, both diagonals are counters and connect at the center of the pin. There is then no necessity for countersinking the rivets at the joint *E*.

62. The verticals can next be drawn in the same way as has been described several times in the preceding articles. Both verticals are broken off $3\frac{5}{8}$, $3\frac{3}{4}$, and $5\frac{5}{8}$ inches below the center line of the top chord. The top line of the angles shown dotted is $\frac{7}{8}$ inch, and the bottom line is $5\frac{1}{4}$ inches, below the center line of the top chord.

63. The eyebars can next be drawn 3 inches in width and with circular heads having a radius of 4 inches. The pins are $3\frac{1}{2}$ inches, the threaded ends are $2\frac{1}{2}$ inches, and the pin nut is $5\frac{3}{16}$ inches in diameter. These can now be drawn.

64. In order to avoid confusion in the drawing, the upper end of the verticals, which are alike, is shown by itself, half way between the two verticals and with the center of the pinhole 2 inches below the center line of the top chord. The rivets connecting the pin plate to the web of the channel are shown flattened on the outside to $\frac{3}{8}$ inch in height. This is necessary at the joint *D* to leave room for the eyebar heads, and is done at joint *E* for the purpose of having the verticals alike.

65. The side view of the verticals, showing also the cross-section of the top chord, is given on the right-hand end of the plate, with the center line parallel to and 2 inches from the right border line. This is so much like other side views that have been already explained in full that it is deemed unnecessary to explain this view in detail. The nuts are

1 inch in thickness, with recesses $\frac{3}{8}$ inch deep, and the threaded ends project $\frac{1}{4}$ inch at each end beyond the outer surfaces of the nuts. The two diagonals that connect at *E* are shown $\frac{1}{16}$ inch apart. The spaces between their outer surfaces and the inner surfaces of the verticals, and between the outer surfaces of the verticals and the inner surfaces of the chord sections, are filled with filling rings 5 inches in diameter.

INDEX

NOTE 1.—All items in this index refer first to the section (see the Preface), and then to the page of the section. Thus, "Abutment location, §82, p4," means that abutment location will be found on page 4 of section 82.

NOTE 2.—The abbreviations hb and rtb in this index stand, respectively, for "highway bridge" (or "bridges") and "railroad truss bridge" (or "bridges").

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T

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V