THE DESIGN OF A RIVETED PLATE GIRDER RAILROAD BRIDGE

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The Design of a Riveted Plate Girder Railroad Bridge

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THE DESIGN OF A PLATE GIRDER RAILROAD BRIDGE

Data and Specifications:

The design is to be a through plate girder bridge for a single track railroad, the span being 60 ft.-0 in. center to center of end bearings. The governing specifications are those of the A.R.E.A. revised to 1948. The live load is Cooper's Standard E-72 loading and an alternate loading consisting of two 90,000-lb. axles spaced 7 ft.-0 in. center to center as given in specification 203.

In order to satisfy the clearance requirements of specification 105, the main girders are spaced 17 ft.-2 in. center to center, thus allowing for 14-in. cover plates on the upper flanges of the girders. The stringers are spaced 6 ft. -6 in. center to center (Spec. 103). Three equal panels of 20 ft.-0 in. each will be used, since this arrangement gives a reasonable slope to the diagonals of the lateral system. An odd number of panels is preferable to an even number because the maximum moment in the main girders occurs at some distance from the center of the span in the former case, and the resulting maximum moment is smaller.

Structural grade, open-hearth steel is to be used for all parts except the shoes and pedestals of the end bearings which are to be of cast steel. Rivets with a nominal diameter of $\frac{7}{8}$ in. will be used throughout.

The order of design which will be followed is;

1. Ties.

2. Stringers.

3. Intermediate floor-beams.

4. End floor-beams.

5. Main girders, including web and flange splices.

6. Lateral bracing.

7. End bearings.

DESIGN OF TIES.

The specifications affecting the design of the ties are numbers 103, 109, 202, 203, 204 and 301. The ties rest directly on the stringers, the live load being applied at the base of the rails, which, for standard gage railroads, are approximately 5 ft.-0 in. center to center. The dead load of the ties and rails is considered as concentrated at the rails in order to simplify the computations. The maximum axle load of 90,000 lb. is assumed to be distributed over 3 ties (Spec. 204). The total live load concentration at each rail on each tie, including 100 per cent allowance for impact, is

$$\frac{1}{3}\left(\frac{90,000}{2}\right) = 30,000 \text{ lb.}$$

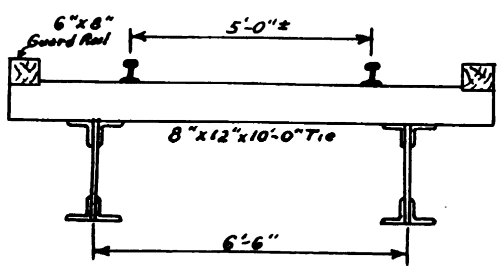

Fig. 1

In estimating the dead load, provision must be made for the weights of the ties, the wooden guard rails at each end of the ties, and the steel rails and fastenings. The rails, guard rails and fastenings are assumed to weigh 200 lb. per lin. ft. of track and the weight of timber is assumed to be 60 lb./ft.^{3} The total weight per tie is assumed as 600 lb. The total bending moment on one tie is

$$30{,}300\left(\frac{6.5-5.0}{2}\right)\times 12 = 272{,}500 \text{ in.-lb.}$$

The allowable extreme fiber stress is 2000 lb. per sq. in. Using the equation for flexure, $M = f\left(\frac{I}{e}\right)$, in which for rectangular sections $\frac{I}{e} = \frac{1}{6}\left(bh^2\right)$, the required product of $b \times h^2$ is $bh^2 = \frac{6 \times 272{,}500}{2000} = 818$ in.^{3}

An 8 x 12-in. tie, laid with the long dimension vertical in accordance with standard practice for bridge ties, is satisfactory. ($bh^2 = 1152$ in.^{3}). The length of ties is 10 ft.-0 in. (Spec. 109).

The weight of the floor per tie is then as follows:

Tie = $\frac{8 \times 12}{144} \times 10 \times 60$ =	400 lb.
Rails, guard rails & Fastenings =	200 lb.
Total =	600 lb.

With a clear spacing of 4 in. between ties, this is also the weight per lineal foot of the floor which will be used in the following computations.

DESIGN OF STRINGERS.

The dead weight of each stringer per lineal foot may be estimated from the equation;

$$w = \frac{k}{2}\left(12.5\,\ell + 100\right)$$

$w = \frac{1.22}{2}\left(12.5 \times 20 + 100\right) = 213.5$ lb.

The total dead load per ft. per stringer is $\frac{600}{2} + 214 = 514$ lb.

Maximum Moment and Shear.

The dead load moment $= \frac{1}{8} \times 514 \times (20)^2 \times 12 = 308{,}000$ in.-lb.

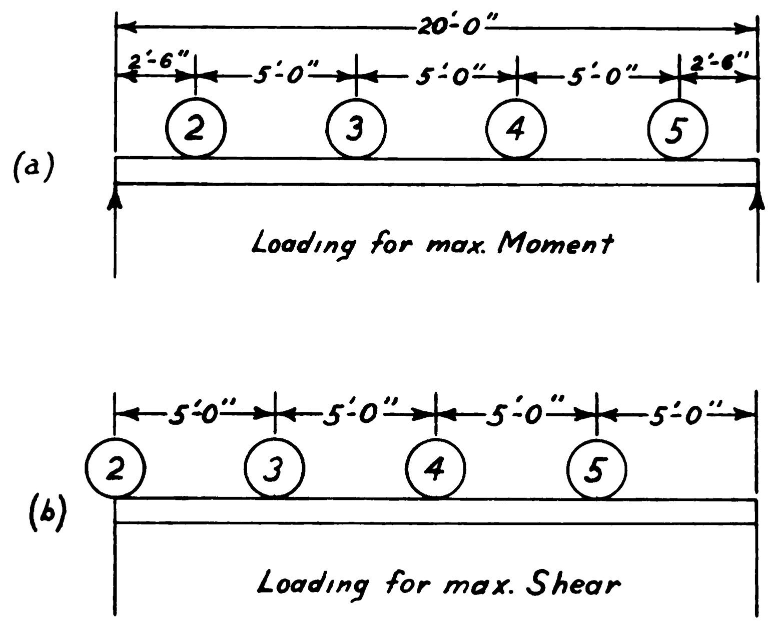

Fig. 3

The absolute maximum live load moment occurs when wheels 2, 3, 4 and 5 are on the stringer as shown in Fig. 3a.

The live load moment $= (72{,}000 \times 10 - 36{,}000 \times 7.5 - 36{,}000 \times 2.5)\ 12$

$= 4{,}320{,}000$ in.-lb.

The allowance for impact =

$\left[60 - \frac{(20)^2}{500}\right] 4{,}320{,}000 \qquad = \underline{2{,}557{,}440}$ in.-lb.

The total moment $\qquad = 6{,}877{,}440$ in.-lb.

The maximum dead load shear $= 10 \times 514 = 5140$ lb.

The maximum live load shear occurs with wheel 2 at the

Moment Diagram - Cooper E-72 Loading

	Loads in Thousands of pounds per axle	Sum of Loads in Thousands of pounds	Distances in feet (to next)	Sum of Distances in feet	Moments in Thousands of foot-pounds
1	36	36	8	0	0
2	72	108	5	8	288
3	72	180	5	13	828
4	72	252	5	18	1728
5	72	324	9	23	2988
6	46.8	370.8	5	32	5904
7	46.8	417.6	6	37	7758
8	46.8	464.4	5	43	10263.6
9	46.8	511.2	8	48	12585.6
10	36	547.2	8	56	16675.2
11	72	619.2	5	64	21052.8
12	72	691.2	5	69	24148.8
13	72	763.2	5	74	27604.8
14	72	835.2	9	79	31420.8
15	46.8	882.0	5	88	38937.8
16	46.8	928.8	6	93	43347.6
17	46.8	975.6	5	99	48920.4
18	46.8	1022.4	5	104	53798.4
Train Load	7200 lbs./ft.			109	58910.4

MOMENT DIAGRAM

Fig. 2

end of the span, (Fig. 3b) and is equal to

$$36{,}000\ \frac{(20 + 15 + 10 + 5)}{20} = 90{,}000 \text{ lb.}$$

The allowance for impact =

$$\left[60 - \frac{(20)^2}{500}\right] 90{,}000 \qquad = \underline{53{,}280} \text{ lb.}$$

$$\text{The total shear} \qquad = 143{,}280 \text{ lb.}$$

Design of Web. The depth of the web will be made equal to $\frac{1}{7}$ of the span, or 34 in.

According to specification 301, the allowable unit shearing stress on the gross section of plate girder webs is 11,000 lb. per sq. in. The thickness required is

$$t = \frac{143{,}280}{34 \times 11{,}000} = 0.383 \text{ in.}$$

A 34 x -in. web plate will be used.

Design of Flanges. The flanges of the stringers, floorbeams, and main girders will be designed assuming the total effective flange area as being concentrated at the centers of gravity of the flanges. The backs of the top flange angles of the stringers will be made flush with the upper edge of the web plate, so as to prevent the formation of a water pocket between the web and angles; the bottom flange angles will project $\frac{1}{4}$ in. beyond the lower edge of the web plate to secure the greatest possible effective depth. In order to provide for the flange rivets, a vertical leg which will permit the use of two rows of rivets will undoubtedly be necessary. A 6-in. leg is the smallest standard size which may conveniently be riveted this way. A 6-in. outstanding leg will also be used in order to secure greater resistance to lateral deflection in the compression flange than would be obtainable with a smaller angle. According

to specification 406, the minimum thickness of a 6 x 6-in. flange angle that may be used is $\frac{1}{10}$ x 6, or 0.6 in. If allowance is made for the thickness of the angle, this value may be reduced to $\frac{9}{16}$ in. Angles 6 x 6 x $\frac{9}{16}$-in. will be assumed.

The center of gravity of each flange is 1.71 in. from the backs of the angles. The effective depth is then 34.25 - 2 x 1.71 = 30.8 in. The maximum flange stress is 6,877,440 ÷ 30.8 = 223,500 lb., and the required net flange area is 223,500 ÷ 18,000 = 12.4 sq. in. Since $\frac{1}{8}$ of the gross section of the web ($\frac{1}{8}$ x 34 x $\frac{7}{16}$ = 1.86 sq. in.) is considered effective in resisting bending stresses, the lower flange angles must furnish a net area of 12.4 - 1.86 = 10.54 sq. in.

Assuming that the rivet pitch near the point of maximum moment is equal to or greater than 4 in., according to spec. 409 only one rivet hole need be deducted from the gross section of each angle. The net section furnished by the assumed angles is 2(6.43 - 1 x $\frac{9}{16}$) = 11.74 sq. in.

The allowable unit stress in the compression flange (Spec. 301) is 18,000 - $\frac{5(20 \times 12)^2}{(12.44)^2}$ = 16,140 lb. per sq. in. The gross area required in the angles is

$$\frac{6,877,440}{30.8 \times 16,140} - 1.86 = 11.97 \text{ sq. in.}$$

The gross area furnished is 12.86 sq. in., and the same angles are satisfactory for both flanges.

Rivet Spacing. The smallest rivet pitch is required at the ends of the stringers, at which points the shear is a maximum. Since the top flange supports a direct vertical load, the stress in the rivets due to this load must be con-

sidered in addition to that caused by the horizontal shear. In order to simplify the shop work, the rivets in the lower flange will be spaced the same as those in the upper flange.

In determining the vertical load on the rivets, the weight of one of the heavy drivers, plus 100 per cent allowance for impact, is assumed to be distributed over 3 ft. of flange (Spec. 428). The vertical load per lin. ft. , per stringer is as follows:

$$\text{Dead load} = 514$$

$$\text{Live load} = \frac{36{,}000}{3} = 12{,}000$$

$$\text{Impact} = \underline{12{,}000}$$

$$\text{Total} = 24{,}514 \text{ lb.}$$

The vertical load per linear inch, is

$$w = \frac{24{,}514}{12} = 2040 \text{ lb.}$$

The total horizontal increment of flange stress per linear inch is equal to the shear, 143,280 lb., divided by the effective depth, 30.8 in. Since part of this stress is resisted by the web itself, the amount that must be transmitted to the rivets is reduced in proportion to the ratio of areas involved; therefore

$$\text{H.I.} = \frac{143{,}280}{30.8} \times \frac{12.86}{12.86 - 1.86} = 4060 \text{ lb.}$$

The resultant increment is

$$\text{R.I.} = \sqrt{(4060)^2 - (2040)^2} = 4540 \text{ lb. per lin. in.}$$

The strength of one rivet is governed by the bearing value on the $\frac{7}{16}$-in. web plate, and is equal to 11,800 lb.

with the 27,000 lb. unit stress allowed by spec. 301. The maximum allowable pitch at the ends of the stringer is then 11,800/4540 = 2.6 in. $2\frac{1}{2}$ in. will be used. Since the two gage lines in the 6-in. leg are $2\frac{1}{4}$ in. apart, the actual distance center to center of the rivets is 3.36 in., which satisfies the requirements of spec. 414.

In a similar manner the maximum allowable pitches at the quarter point and at the middle point of the span are computed. The maximum live load shears at the center occurs with the alternate load.

Quarter point.

$$\text{Total shear} = \left(\frac{15}{20} + \frac{10}{20} + \frac{5}{20}\right)\frac{72{,}000}{2} - 514 \times 5 = 51{,}430 \text{ lb. per stringer}$$

$$\text{Load per ft.} = 514 + \left(\frac{36{,}000}{3}\right)2 = 24{,}514 \text{ lb.}$$

$$w = \frac{24{,}514}{12} = 2040 \text{ lb./in.}$$

$$\text{H.I.} = \frac{51{,}430}{30.8} \times \frac{12.86}{12.86 - 1.86} = 1457 \text{ lb.}$$

$$\text{R.I.} = \sqrt{(2040)^2 + (1457)^2} = 2500 \text{ lb. per lin. in.}$$

$$\text{Max. pitch} = \frac{11{,}800}{2500} = 4.7 \text{ in.}$$

Middle point.

$$\text{Total shear} = \left(\frac{10}{20} + \frac{3}{20}\right) \times \frac{90{,}000}{2} - 514 \times 10 = 24{,}110 \text{ lb. per stringer}$$

$$\text{Load per ft.} = 514 + \left(\frac{45{,}000}{3}\right)2 = 30{,}514 \text{ lb.}$$

$$w = \frac{30{,}514}{12} = 2540 \text{ lb./in.}$$

$$\text{H.I.} = \frac{24{,}110}{30.8} \times \frac{12.86}{12.86 - 1.86} = 683 \text{ lb.}$$

$$\text{R.I.} = \sqrt{(2540)^2 - (683)^2} = 2630 \text{ lb.}$$

Max. pitch = $\frac{11,800}{2630}$ = 4.5 in.

From the above, the pitch is selected as 4.5 in. over the center half of the stringer span.

Connection to floor-beam. The stringers will be connected to the floor-beams at either end by means of a pair of angles, one leg of each connection angle being riveted to the web of the stringer and the other leg riveted to the web of the floor-beam. Shop rivets are used in the web of the stringer and field rivets in that of the floor-beam. The latter rivets are power driven, however, so that the same unit values are used as for the shop rivets. The minimum size connection angle, according to spec. 425 is 4 x 4 x $\frac{1}{2}$ in.

The number of rivets which must pass through the stringer web is governed by the bearing value on the web.

$\frac{143,280}{11,800}$ = 13 rivets are required

Of these, a sufficient number must be placed through the angles to prevent failure of the rivets in double shear.

$\frac{143,280}{16,200}$ = 9 rivets

Theoretically, those rivets which pass through the flange angles should not be considered effective in the end connection as they are already stressed in transferring the horizontal shear from the flange to the web. Since the entire end connection serves to stiffen the stringer at this point, however, it may be considered as safe practice to count one-half of these rivets as available for the end connection.

With this number of rivets required, it will be necessary to place them in two rows. Thus the connection angle size will be increased to 6 x 6 x $\frac{1}{2}$ in. to satisfy spec. 413. The detail shown in Fig. 4 furnishes 9 rivets in double shear and 13 in bearing on the stringer web, 4 of the latter being in the filler plates outside of the angles. The actual spacing of the double-shear rivets is 3 -in. center to center, which is permitted by spec. 413.

The number of rivets required in the outstanding legs of the connection angles must be sufficient to provide for the maximum stringer reaction in single shear, or for the maximum floor-beam reaction (which provides for the maximum simultaneous loads on two adjacent stringers) in bearing on the floor-beam web. The latter number cannot be determined until the floor-beam is designed. The number required in single shear is

143,280/8,100 = 18 rivets

Nine rivets will be placed in each angle, unless the number required for bearing on the floor-beam web necessitates an increase. This will be investigated at the end of the floor-beam design. These rivets cannot be spaced finally until the position of the angles with respect to the floor-beam is fixed. The complete stringer detail is shown in Fig. 4.

Computed Weight. Before proceeding with the design of the remaining parts of the bridge, the actual weight of one stringer will be determined in order to check the value which was previously assumed. In determining the finished

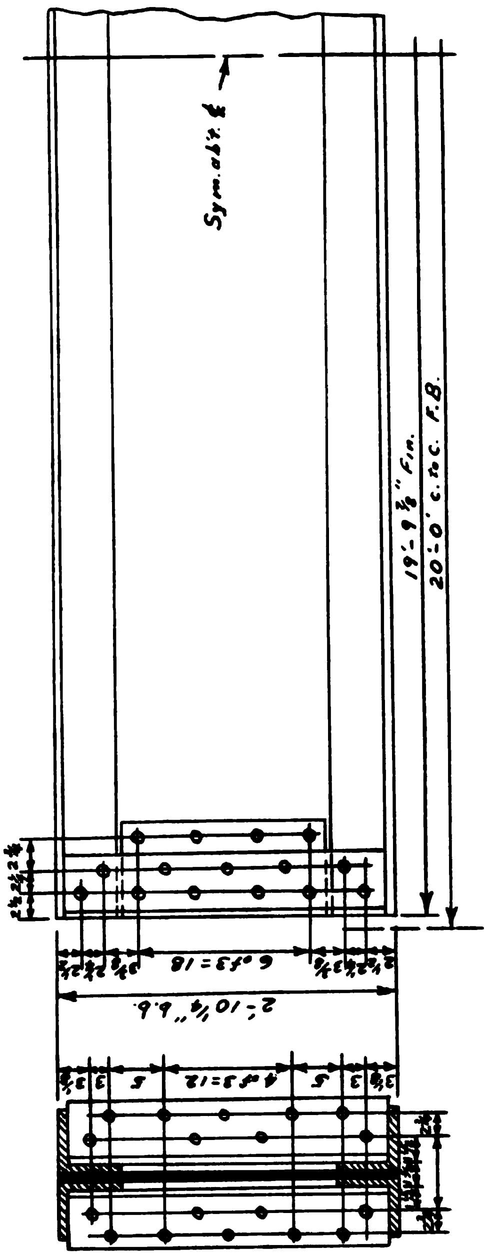

Fig. 4

length of the stringer flange angles and web, an erection clearance of $\frac{1}{16}$ in. is provided at aech end; the thickness of the floor-beam web is assumed as $\frac{1}{2}$ in. and that of the floor-beam flange angles as $\frac{3}{4}$ in.

The resulting weights are as follows:

1 web plate 34 x $\frac{7}{16}$ x 19 ft.-9$\frac{7}{8}$ in. @ 51.2 #/ft.	=	1014 #
4 flange angles 6 x 6 x $\frac{9}{16}$ x 19 ft.-9$\frac{7}{8}$ in. @ 21.9#/ft.	=	1736
4 filler plates 9 x $\frac{9}{16}$ x 1 ft.-10 in. @ 31.5 #/ft.	=	128
4 conn. angles 6 x 6 x $\frac{1}{2}$ x 2 ft.-9$\frac{7}{8}$ in. @ 19.6#/ft.	=	216
300 rivet heads @ 21.3 #/100	=	70
Total	=	3164 #

The weight assumed was 214 x 20 = 4280 lb., which is acceptable.

DESIGN OF INTERMEDIATE FLOOR-BEAM.

Each intermediate floor-beam is a built-up girder, the span of which is equal to the distance center to center of the main girders, 17 ft.-2 in., and which supports two symmetrical concentrated loads (Fig. 5) spaced 6 ft.-6 in. apart, in addition to its own weight. Each of the concentrated loads is equal to the sum of the maximum simultaneous end reactions at the abutting ends of the two adjacent stringers, including the dead load, live load, and the

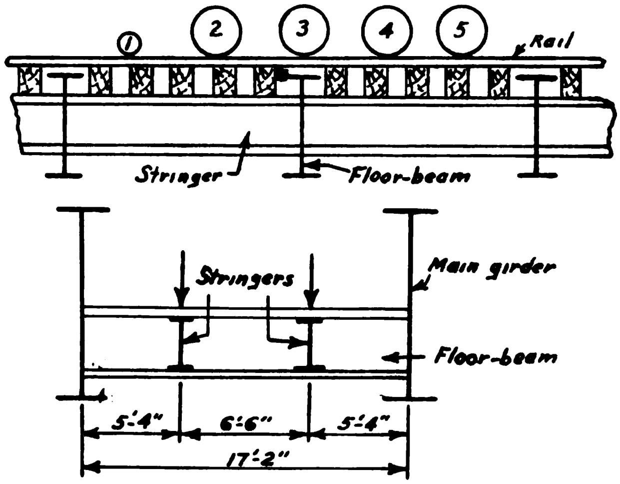

Fig. 5

allowance for impact. The weight of the floor-beam varies with too many factors to permit of an accurate preliminary estimate, other than may be obtained by comparison with previous similar designs. Some of the influencing factors are the spacing of floor-beams, the spacing of girders, the depth of the floor-beam, the maximum thickness of metal which may be used, which effects the composition of the flanges, the class of loading, etc. A sufficiently approximate criterion of the weight, W, of one floor-beam is given by the formula

$$W = 2600 + 2Ed$$

in which E = the class of Cooper's loading

d = the distance between floor-beams in feet.

$$W = 2600 + 2 \times 72 \times 20 = 5480 \text{ lb.}$$

In order to simplify the computations for moment and shear, one-half of the weight of the floor-beam will be considered as concentrated at each stringer.

Maximum Moment and Shear. The maximum live load floor-beam reaction for the given conditions occurs when wheel 4 is placed at the middle support of two adjacent panels, wheels 1, 2, and 3 being on one span and wheels 5, 6 and 7 on the other span. The live load reaction is

$$\frac{2}{20} \times 18 + \frac{10}{20} \times 36 + \frac{15}{20} \times 36 + 36 + \frac{15}{20} \times 36 + \frac{6}{20} \times 23.4$$
$$+ \frac{1}{20} \times 23.4 = 118{,}000 \text{ lb.}$$

The allowance for impact, assuming the loaded length to be 2 panel lengths is

$$\left[60 - \frac{(40)^2}{500}\right] 118{,}000 = 67{,}000 \text{ lb.}$$

The dead load concentration is 3164 + 20 x 300 = 9164 lb., and the total maximum floor-beam load from the stringers is 118,000 + 67,000 + 9164 = 194,164 lb.

Assuming the weight of the floor-beam as 5480 lb., the total concentrated load at each stringer connection is

$$194{,}164 + \frac{5480}{2} = 196{,}904 \text{ lb.}$$

The maximum moment = $196{,}904 \times \frac{17.17 - 6.5}{2} \times 12 = 12{,}600{,}000$ in.-lb.

The maximum shear = 196,904 lb.

Design of Web.

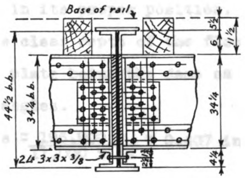

Fig. 6

The depth of web must be selected with due consideration for the connection of the stringer. The angles of both the upper and lower flanges project ¼ in. beyond the edges of the web plate, so as to provide for the proper placing of the cover plates (assuming that such are necessary).

As shown in Fig. 6, the depth of the floor-beam web is 10 in. greater than that of the stringer. The upper surface of the top flange angle of the stringer is made flush

with the lower edge of the top flange angle of the floor-beam. With the 12-in. ties notched $\frac{1}{2}$ in., a clearance of $3\frac{3}{4}$-in. ($4\frac{1}{8}$ to top of rivet heads) is provided between the base of the rail and the uppermost surface of the floor-beam. This clearance is based on the assumption that a 6-in. vertical leg will be necessary for proper riveting, and that one $\frac{3}{4}$-in. cover plate will be sufficient to provide for the flange stresses in the floor-beam. The lower surface of the bottom flange angle of the stringer is so located with the arrangement indicated above, that a 3 x 3 x -in. erection angle may be riveted to the lower gage line of the 6-in. floor-beam angle and at the same time support the stringer in its proper position. A filler plate is placed in the clear depth of the floor-beam web, the thickness of this plate being the same as that of the floor-beam flange angles.

Web thickness $= \frac{196,900}{11,000 \times 44} = 0.407$ in.

Since the commercial thicknesses are multiples of $\frac{1}{16}$ in., a $\frac{7}{16}$-in. web is selected.

Design of Flanges. In the preliminary investigation, the effective depth is assumed to be $2\frac{1}{2}$ in. less than the distance back to back of flange angles, or 42 in. The maximum flange stress is

$$\frac{12,600,000}{42} = 300,000 \text{ lb.}$$

and the total effective net flange area required is

$$\frac{300,000}{18,000} = 16.67 \text{ sq. in.}$$

Flange elements must furnish $16.67 - \frac{1}{8} \times 42 \times \frac{7}{16} = 14.37$ sq. in.

A rivet pitch of $2\frac{1}{2}$ in. in the flange angles is assumed, thus requiring 1 rivet holes to be deducted from each vertical leg. A total of $4\frac{3}{4}$ holes must be deducted from the two angles and a total of two holes from each cover plate. The former number may have to be revised when the exact rivet pitch is known.

The flange section assumed is:

Section	Total gross area, sq. in.	Total No. of rivet holes	Total area of rivet holes, sq. in.	Total net area, sq. in.
2 Ls 6x4x¾	13.88	4 ¾	3.56	10.32
1 Cover Pl. 10x⅝	6.25	2	1.50	4.75
Total	21.38	....	.....	15.07

The true effective depth may now be determined. The distance from the backs of the angles to the center of gravity of the flange section, using gross areas, is

$$\bar{x} = \frac{13.88 \times 2.08 - 6.25 \times 0.312}{13.88 + 6.25} = 1.34 \text{ in.}$$

And the effective depth of the floor-beam is

$$h = 44.5 - 2 \times 1.34 = 41.82 \text{ in.}$$

The ervised required net section of the flange elements is

$$\frac{12,600,000}{18,000 \times 41.82} = 2.3 = 14.43 \text{ sq. in.}$$

The same section will be used for the compression flange.

Pitch of Rivets.

$$\text{H.I.} = \frac{196,900}{41.82} \times \frac{20.13}{20.13 + 2.3} = 4225 \text{ lb. per in.}$$

The strength of one rivet is governed by the bearing value on the -in. web plate, 11,800 lb. The maximum pitch is

$$p = \frac{11,800}{4225} = 2.79 \text{ in.}$$

A value of $2\frac{3}{4}$ in. will be used from the supports to the stringer connections. This is a greater pitch than the value assumed in computing net flange area and no revision of design is necessary. Since the shear at any section between the stringers is practically zero, the maximum pitch allowed by specifications, 7 in., will be used in this portion of the floor-beam.

The required pitch in the cover plates is

$$(H.I.)' = \frac{196,900}{41.82} \times \frac{6.25}{20.13 + 2.3} = 1310 \text{ lb. per in.}$$

$$p' = \frac{8100}{1310} = 6.2 \text{ in.}$$

Since two cover plate rivets are placed in the same cross-section, this value of p' may be doubled for the pair of rivets. According to spec. 413, however, the maximum allowable pitch is 6.5 in., which governs in this case.

Connection to Main Girders. The end connection of the floor-beam will consist of a pair of 6 x 6 x -in. angles, riveted to the floor-beam and to the main girders in a manner similar to that used in fastening the stringers to the floor-beams. The number of shop rivets required in bearing on the $\frac{7}{16}$-in. web plate is 196,900/10,350 = 19; the number required in double shear is 196,900/16,200 = 13; and the number of field rivets (power driven) required in single shear through the web of the main girder is 196,900/8100 = 25.

Computed Weight. Assuming a -in. girder web, and $\frac{3}{4}$-in. girder flange angles, and allowing -in. erection clearance at either end, the actual finished length of the floor-beam is 19ft.-10 in. The weight of one floor-beam is as follows:

1 web plate 42 x $\frac{7}{16}$ x 19 ft.-10 in. @ 63.1#/ft.	= 1250 lb.
4 flange ∠s, 6 x 4 x $\frac{3}{4}$ x 19 ft.-10 in. @ 23.6#/ft.	= 1820
2 cover plates 10 x $\frac{5}{8}$ x 19 ft.-10 in. @ 21.5#/ft.	= 850
4 filler plates 9 x $\frac{3}{4}$ x 2 ft.-10 in. @ 23.0#/ft.	= 260
4 conn. 6 x 6 x $\frac{5}{8}$ x 3 ft.-8$\frac{7}{8}$ in. @ 24.2#/ft.	= 360
4 filler plates 20 x $\frac{3}{4}$ x 2 ft.-10 in. @ 51.0#/ft.	= 580
800 rivet heads @ 21.3#/100	= 170
4 erection ∠s 5 x 3 x $\frac{3}{8}$ x 1 ft.-1 in. @ 9.8#/ft.	= 40
Total	5330 lb.

The assumed weight was 5480 lb., so no revision is necessary.

Time does not permit the completion of the design, but the procedure for designing the end floor-beams and main girders is similar to that used for the stringers and intermediate floor-beams.

ROOM USE ONLY

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