Design of a Railroad Bridge.
Baltimore Type.
216' 0" Span.

Thesis for B.S.
Class 1900.

L. R. Holmes.

I have taken the design of a single-track through railroad bridge, having pin connected trusses of the Baltimore type. The span is 216 feet and there will be 18 panels each 12 feet long. The depth of the truss, center to center of chords, will be 32 feet. The distance center to center of trusses will be 16 feet. The design is to be made in accordance with Cooper's General Specifications for Iron and Steel Railroad Bridges and Viaducts, the required portions of which are given in "Roofs and Bridges" Part III pages 134-143. The material to be used is wrought iron, excepting for the pins in the trusses.

In working up the design of this bridge the following books have been consulted:

"Roofs and Bridges" Parts I & III. Merriman & Jacoby.

"Theory and Practice of Modern Framed Structures"
Johnson, Bryan, Turneaure.

"The Strains in Framed Structures"
Du Bois.

Pocket Companion of 1893
The Carnegie Steel Company.

Pocket Book of Pencoyd Iron Works
1892

Floor Timbers and Stringers.

Cross Ties.

The loading used to determine the size of the cross ties will be the alternate live load in § 23, consisting of 80000 pounds equally distributed on two pairs of drivers seven feet apart. The distance between the stringers will be taken at 6 feet 6 inches between centers. Each of the above wheels is supposed to cover three ties thus making the load on each rail for one tie equal to $20\,000 \div 3 = 6667$ pounds. The distance between centers of rails is 4 feet 10½ inches. The bending moment is $6667 \times 9.75 = 65000$ pound-inches, omitting as comparatively small the weight of the rail, spikes and cross ties. For a unit stress of 800 pounds per square inch the resisting moment equals $800\,bd^2 \div 6$. Therefore $800\,bd^2 \div 6 = 65000$. Assuming b to be 8 inches, d is found to be 7.8 inches. Therefore ties 8 x 8 inches will be used and will be spaced 4 inches in the clear. Every fourth tie will be bolted to the stringers and the guard rail will be notched over the ties and bolted to them as specified in § 13.

Stringers.

The loading for this will be the two typical passenger locomotives of the Pennsylvania R.R. The length of the stringer is twelve feet and the position of the wheel loads to produce maximum moment is when the two drivers of the first engine are on the stringer in such a position that the line between them is bisected at the center of the stringer. The bending moment due to the wheels is equal to $20000 \times 2 = 40000$ foot-pounds. The weight of the stringer is $6.3l^2 = 6.3 \times (12)^2 = 907$ pounds. The weight of the track for a panel length is 4800 pounds and half of this is 2400 pounds. Therefore the total dead load is $2400 + 907 = 3307$ pounds. The live load transferred to the leeward stringer by the wind pressure on the train is $300 \times 12 \times 8 \div 6.5 = 4431$ pounds. The maximum moment due to the dead load and to the load transferred by the wind is $(3307 + 4430)12 \div 8 = 11607$ foot-pounds. Therefore the total maximum moment is $11607 + 40000 = 51607$ foot-pounds. The depth of the stringer will be one-ninth of the span or 15 inches. We have the expression $\frac{I}{c} = \frac{M}{S}$ and therefore $I = 7\frac{1}{2} \times \frac{51607 \times 12}{8000} =$ 580.5. Therefore use for the stringer an I beam of depth 15 inches weighing 60 pounds per foot. (Carnegie, page 99, B 5)

Floor Beam.

The floor beam will be in the form of a plate girder. The weight of one stringer and the track it supports is equal to 720 + 2400 = 3120 pounds. The live load transferred by the wind equals 300 × 12 × 9.25 ÷ 6.5 = 5123 pounds. The maximum floor beam reaction due to live loads is 26667 pounds. Therefore the total concentrated load on the leeward side is equal to 34910 pounds and that on the windward side to 24664 pounds. The maximum moment due to these loads is under the leeward load and is equal to the reaction multiplied by 4.75 feet. This reaction equals 34148 pounds and the moment is equal to 162204 foot-pounds. The depth of the floor beam is 15 + 4½ + 3½ + an allowance for clearance between top of stringer and the upper flange of the floor beam. The depth will be assumed as 24 inches. Assume the weight of the floor beam to be 2000 pounds. The moment due to the weight of the beam, at the leeward side is 3346 foot-pounds. Therefore the maximum bending moment is 162204 + 3346 = 165550 foot-pounds. The effective depth is 23 inches and the flange stress is 165550 × 12 ÷ 23 = 86374 pounds. The net flange area is hence 86374 ÷ 8000 = 10.797 square inches. The composition of both flanges will be as follows:

2 angles, 5" x $3\frac{1}{2}$" x $\frac{7}{16}$", $2(3.53 - 0.44) = 6.18$ square inches
1 cover plate, 12" x $\frac{1}{2}$", $(12 - 2)\frac{1}{2}$ $= 5.00$ " "
11.18 " " .

The bending moment due to all external loads varies between the end and the stringer connection as the ordinates to a straight line and the curve of moments due to the weight of the floor beam is very nearly a straight line between the same points. The rivet spacing will hence be uniform for this division. If the thickness of the web be taken as 3/8" the number of rivets required will be $86374 \div 3940 = 21.92$ and the pitch is $57 \div 21.92 = 2.''6$ which is below the minimum limit. Let the thickness of the web be increased to 7/16" changing the bearing value of a 7/8" rivet to 4590. The number of rivets required is equal to $86374 \div 4590 = 18.81$ and the pitch is equal to $57 \div 18.81 = 3.''03$. Since the entire load is transferred to the floor beam in a width of 11," the pitch will be made 3" and extended to the inner side of the stringer connection.

The difference in flange stresses between the stringers is so small that the pitch of the rivets derived from it exceeds the maximum limit, hence the pitch will be made 6".

The cover plate of the lower flange may terminate theoretically at the point where the moment equals $7.00 \times 8000 \times 22.5 \div 12 =$ 105000 foot-pounds. This moment is at a distance from the end equal to $105000 \times 57 \div 165550 = 36.15$ inches or $57 - 36.15 = 21.85$ inches from the stringer. The length of the lower cover plate is 11 feet 2 inches after extending at each end to include two additional rivets in each row.

The bearing value of a 7/8" rivet in a 7/16" web is $4590 \times 2/3 =$ 3060 pounds. If field rivets alone are used to connect the end angles of the stringer to the floor beam web, 12 rivets will be needed since $34910 \div 3060 = 11.4$. It is not possible to put 6 rivets in each connecting angle without using a pitch below the minimum limit. Filler plates 3/8" thick will therefore be used on each side of the web of the floor beam. The strength of a field rivet in single shear is $4510 \times 2/3 =$ 3010 pounds, hence 11 rivets must be put in the two connecting angles since $\left(26667 + \frac{3120 + 5123}{2}\right) \div 3010 = 10.2$. Strictly the resultants of these shears and the horizontal reaction of the floor beam due to the direct pressure of wind on train and stringer should be found. As the wind pressure, however, mainly affects the upper rivets, and the deflection of the stringer has a similar effect, an excess of rivets should be provided

They will be arranged as follows: 6 rivets are to unite each angle to the floor beam and 5 to connect the angles to the stringer web. 5 will be placed in each row of the fillers on the floor beam. Of the 22 in the floor beam web, the 10 in the fillers can be shop rivets whose combined strength is 10 × 4590 = 45900 pounds, or more than the full load. This gives the field rivets a bearing strength greater than the double shear. Whenever the bracket angles are placed above the flange angles the rivets in them may also be counted as part of the stringer support.

In the angles connecting the floor beam to the posts the number of rivets attaching them to each post is [34910 + 1000] ÷ 3010 = 12 on the basis of shear. With a pitch of 3", 6 can be put in each angle and when the posts are designed it must be arranged so that the bearing strength shall be sufficient.

The number of rivets connecting the angles to the web of the floor beam must be 35910 ÷ 4590 = 8. The number of rivets in the angles alone must be at least 35910 ÷ (2 × 4590) = 4. As these must stagger with the preceeding rows of 6 rivets, 5 will be put in the angles, excluding the two in the flanges, and 4 in the fillers. The angles are $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ inch and the fillers are $7 \times \frac{1}{2}$ inch.

Below is found the weight of one floor beam.

4 angles $5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, $15' 2''$ long	@ 12.0 lbs	=	760.0
1 Cover plate $12'' \times \frac{1}{2}''$, $15' 2''$ long 1 " " $12'' \times \frac{1}{2}''$, $11' 2''$ "	@ 20.0 lbs	=	530.0
1 web " $24'' \times \frac{7}{16}''$, $15' 1\frac{7}{8}''$ long	@ 35.7 "	=	541.0
4 fillers $15'' \times \frac{3}{8}'' \times 17''$ long	@ 18.75 "	=	106.0
4 bracket angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, $10''$ long	@ 10.9 "	=	37.0
4 Connecting " $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, $23''$ "	@ 10.9 "	=	84.0
8 " " $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, $22''$ "	@ 10.9 "	=	81.0
4 fillers $7'' \times \frac{1}{2}''$, $17''$ long	@ 11.7 "	=	66.0
302 pairs of rivet heads @ 0.444 lbs		=	134.0
	Total		2339.0 lbs

As the above weight does not differ materially from that assumed for the floor beam, and as good allowance has been made in the designing of the above pieces, this will be taken as the composition of the floor beam.

Stresses in Trusses.

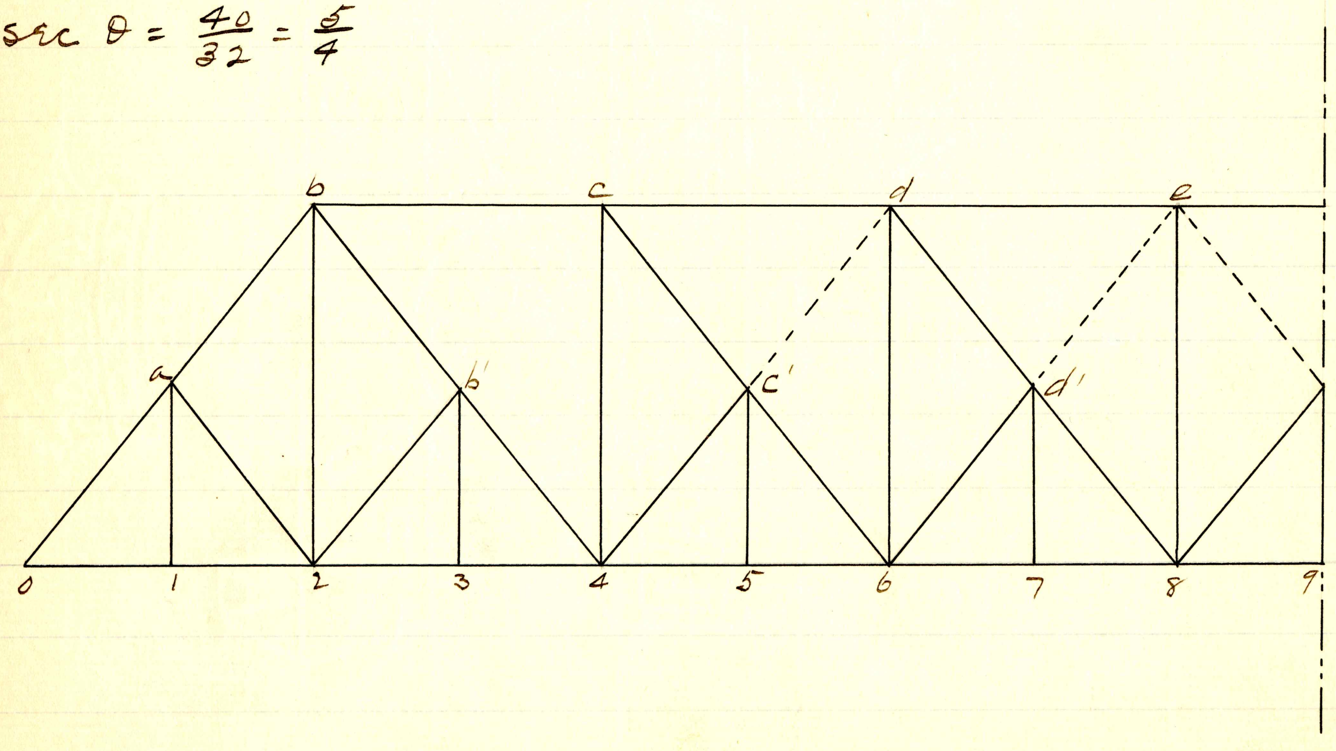
The following data and dimensions are tabulated below for convenient reference:

Span, center to center of end pins	216' 0"
Depth between centers of chords	32' 0"
Width, between centers of trusses	16' 0"
Number of panels	18
Panel length	12' 0"
Length of end post, center to center of pins	40' 0"

$\tan\theta = \frac{12}{16} = 0.75$

$\theta = 36^\circ\ 52'$

$\sec\theta = \frac{40}{32} = \frac{5}{4}$



Dead Load Stresses.

The total dead load per linear foot for one truss will be taken equal to 600+6l, l being the span in feet. This gives a dead load per truss of 204768 pounds. Assume 70000 pounds of this on the upper chord giving an apex load of 8750 pounds. Assume the panel load on the lower chord to be 7490. This gives the total dead load as 204820 pounds.

The manner in which these stresses were computed was taken from "Theory and Practice of Modern Framed Structures" page 60 and "Roofs and Bridges, Part I" page 87. Below are the stresses:

Upper Chord.

bc = −124583 lbs   cd = −160178 lbs

de = −177975 "   ef = −177975 "

Lower Chord.

0-1 = +73999 lbs   1-2 = +73999 lbs

2-3 = +73999 "   3-4 = +73999 "

4-5 = +124583 "   5-6 = +124583 "

6-7 = +160178 "   7-8 = +160178 "

8-9 = +177975 "

End Post.

$oa = -123331$ pounds. $ab = -118650$ pounds

Sub-Verticals

$a1 = b'3 = c'5 = d'7 = e'9 = +7490$ pounds

Verticals.

$b2 = +6230$ pounds $c4 = -56210$ pounds

$d6 = -32480$ " $e8 = -8750$ "

Web-Members.

$a2 = b'2 = c'4 = d'6 = e'8 = -4681$ pounds

$b'4 = +84306$ pounds $c'6 = +54644$ pounds

$d'8 = +25519$ " $bb' = +88988$ "

$cc' = +59325$ " $dd' = +29663$ "

Live Load Stresses.

The loading used in the calculation of these stresses was that of the two typical passenger locomotives of the Pennsylvania Railroad of which a tabulation of the loads are given in "Roofs and Bridges" Part I, Art 59. The loads there given are for both rails so, in order to get the stresses for one truss, these were divided by two.

The principles employed in the computation of these stresses were those of "Roofs and Bridges" Part I, Arts. 60 + 61.

Below are the stresses:

Upper Chord.

bc = -200100 pounds　　cd = -255620 pounds

de = -282100 "　　ef = -282100 "

Lower Chord.

0-1 = +115540 pounds　　1-2 = +115540 pounds

2-3 = +115540 "　　3-4 = +115540 "

4-5 = +200100 "　　5-6 = +200100 "

6-7 = +255620 "　　7-8 = +255620 "

8-9 = +282100 "

End Post

0a = -207760 pounds　　ab = -192520 pounds.

Sub-Verticals

a1 = b'3 = c'5 = d'7 = e'9 = +28667 pounds.

Verticals.

b2 = +57334 pounds　　c4 = -85170 pounds

d6 = -58467 "　　e8 = -35827 "

Web Members.

a2 = b'2 = c'4 = d'6 = e'8 = -17920 pounds.

$b'4 = +142820$ pounds. $c'6 = +107020$ pounds

$d'8 = +76220$ " $bb' = +152109$ "

$cc' = +106462$ " $dd' = +73084$ "

$ee' = +44584$ " (counter).

Wind Load Stresses.

In working out the live load stresses a uniform load of 3000 pounds per linear foot for both rails: this would make 1500 pounds per linear foot for one truss. For the wind on the train a load of 300 pounds per linear foot will be assumed and supposed to be all transferred to the leeward side. The center of pressure is seven feet above the rail. This would make the wind load on the truss equal to $\frac{9 \times 300}{16} = 170$ pounds per foot. $\frac{170}{1500} = 11\frac{1}{3}$ per cent. This will produce stresses in the truss members of the same kind as those produced by the live load and $11\frac{1}{3}\%$ as large. Since in this discussion the wind load was supposed to be transferred to the leeward truss, whereas, in reality, it would be divided between the two trusses, I think that the stresses thus calculated will be sufficient to cover all of the wind stresses in the truss members. It is also not very likely that a train will be on the bridge during a wind storm which would cause such stresses as I have calculated. Furthermore, the factors of safety to be used and the allowances that will be made in the designing of truss members render it unnecessary to consider the different kinds of wind stresses mentioned in "Roofs and Bridges" Part III, Art 51.

The wind load stresses are hence as follows:

Upper Chord.

bc = − 22678 pounds. cd = − 28970 pounds.
de = − 31971 " ef = − 31971 "

Lower Chord.

0-1 = + 13095 pounds 1-2 = + 13095 pounds
2-3 = + 13095 " 3-4 = + 13095 "
4-5 = + 22678 " 5-6 = + 22678 "
6-7 = + 28970 " 7-8 = + 28970 "
8-9 = + 31971 "

End Post.

0a = − 23546 pounds ab = − 21819 pounds.

Sub-Verticals

a-1 = b'3 = c'5 = d'7 = e'9 = + 3249 pounds

Verticals.

$b2 = +6498$ pounds. $c4 = -9753$ pounds.

$d6 = -6626$ " $e8 = -4060$ "

Web-Members.

$a2 = b'2 = c'4 = d'6 = e'8 = -2031$ pounds

$b'4 = +16186$ pounds $c'6 = +12129$ pounds.

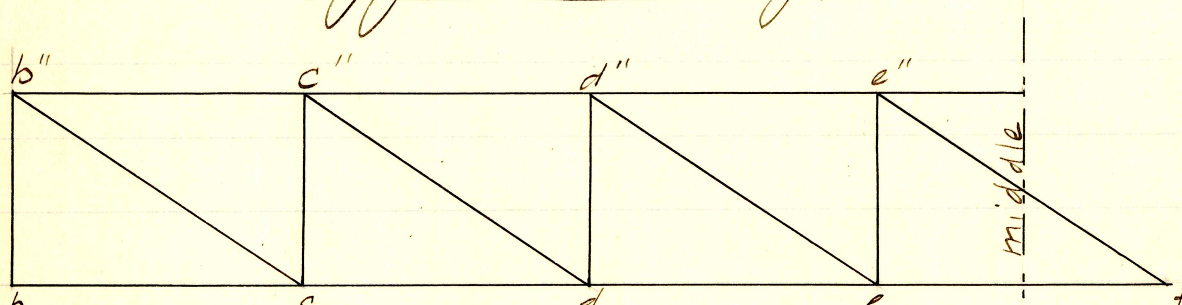
$d'8 = +8638$ " $bb' = +17239$ "

$cc' = +12066$ " $dd' = +8283$ "

$ee' = +5053$ " (counters)

Lateral Systems.

Upper Lateral System.



$\tan\theta = \frac{24}{16} = 1.5$ $\theta = 56°19'$ $\sec\theta = 1.8031$

The loading for the upper lateral system is taken at 150 pounds per foot and supposed to be equally divided between the windward and leeward sides. This gives a panel load on each chord of 1800 pounds

The stresses are computed by supposing bb" and bc removed and considering the remaining part to be a deck bridge with the given loading. Of course the compression in bb" is equal to the net reaction of the loads.

Below are the stresses:

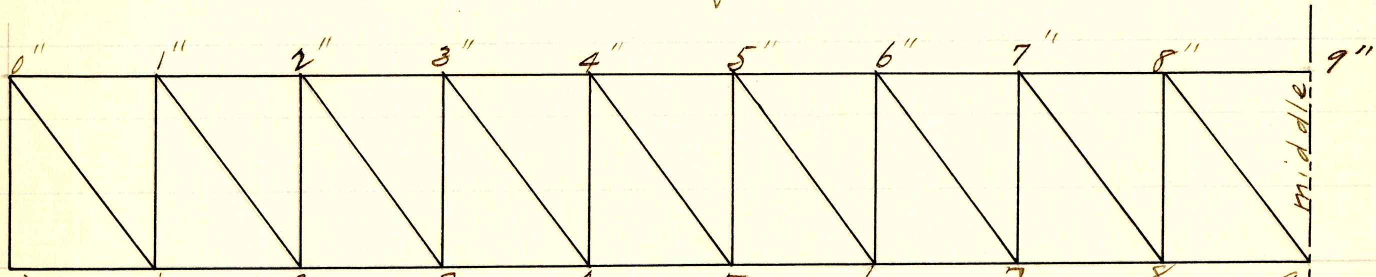
Diagonals.

b"c = +19472 pounds. c"d = +12982 pounds
d"e = + 6510 " e"f = 0

Struts.

bb" = −10800 pounds. cc" = − 9000 pounds
dd" = − 5400 " ee" = − 1800 "

Lower Lateral System.



$\tan\theta = \frac{12}{16} = \frac{3}{4} = 0.75$ $\theta = 30°\ 52'$ $\sec\theta = 5/4$

Fixed Load

The fixed load is the same as that for the upper lateral system giving a panel load on each chord of 900 pounds. The method of computing the stresses is the same as for the upper system. It is not necessary to record the stresses in the struts for this system, which, however, were computed, since the floor beam acts as a strut.

Below are the stresses:

0''-1 = + 19125 pounds	1''-2 = + 16875 pounds.
2''-3 = + 14625 "	3''-4 = + 12375 "
4''-5 = + 10125 "	5''-6 = + 7875 "
6''-7 = + 5625 "	7''-8 = + 3375 "
8''-9 = + 1125 "	

Moving Load.

The moving load is taken as 300 pounds per foot and divided as above giving a panel load on each chord of 1800 pounds. The computation of these stresses was carried out according to "Roofs and Bridges" Part I, Art 32. As above, the stresses in the struts, although computed, will not be recorded.

Below are the stresses:

0''-1 = +38250 pounds   1''-2 = +34000 pounds
2''-3 = +30000 "   3''-4 = +26250 "
4''-5 = +22750 "   5''-6 = +19500 "
6''-7 = +17000 "   7''-8 = +13750 "
8''-9 = +11250 "

Tabulation of Stresses.

	oa	ab	bc	cd	de	ef	b2
Dead Load	-123331	-118650	-124583	-160178	-177975	-177975	+6230
Live Load	-207760	-192520	-200100	-255620	-282100	-282100	+57334
Wind Load	-23546	-21819	-22678	-28970	-31971	-31971	+6498
Maximum	-354637	-332989	-347361	-444768	-492046	-492046	+70062
Minimum	-123331	-118650	-124583	-160178	-177975	-177975	+6230

	c4	d6	e8	a1 = b'3 = c'5 = d'7 = e'9	a2 = b'2 = c'4 = d'6 = e'8	b'4	c'6
Dead Load	-56210	-32480	-8750	+7490	-4681	+84306	+54644
Live Load	-85170	-58467	-35827	+28667	-17920	+142820	+107020
Wind Load	-9753	-6626	-4060	+3249	-2031	+16186	+12129
Maximum	-151133	-97573	-48637	+39406	-24632	+243312	+173793
Minimum	-56210	-32480	-8750	+7490	-4681	+84306	+54644

	d'8	bb'	cc'	dd'	0-1 = 1-2 = 2-3 = 3-4	4-5 = 5-6	6-7 = 7-8
Dead Load	+25519	+88988	+59325	+29663	+73999	+124583	+160178
Live Load	+76220	+152109	+106462	+73084	+115540	+200100	+255620
Wind Load	+8638	+17239	+12066	+8283	+13095	+22678	+28970
Maximum	+110377	+258336	+177853	+110030	+202634	+347361	+444768
Minimum	+25519	+88988	+59325	+29663	+73999	+124583	+160178

	8-9	ee'
Dead Load	+177975	0
Live Load	+282100	+44584
Wind Load	+31971	+5053
Maximum	+492046	+49637
Minimum	+177975	

Upper Lateral System.

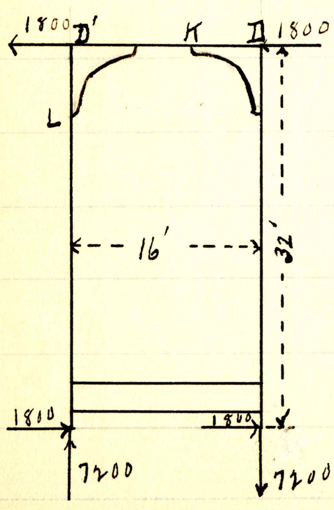
b"c	c"d	d"e	e"f	bb"	cc"	dd"	ee"
+19472	+12982	+6510	0	-10800	-9000	-5400	-1800

Lower Lateral System.

	0"-1	1"-2	2"-3	3"-4	4"-5	5"-6	6"-7
Fixed Load	+19125	+16875	+14625	+12375	+10125	+7875	+5625
Moving Load	+38250	+34000	+30000	+26250	+22750	+19500	+17000
Maximum	+57375	+50875	+44625	+38625	+32875	+27375	+22625

	7"-8	8"-9
Fixed Load	+3375	+1125
Moving Load	+13750	+11250
Maximum	+17125	+12375

The transverse or sway bracing of the trusses will consist in uniting the intermediate posts to the lateral struts by a bracket having a solid web. An outline diagram is given in the figure. As the posts are fixed at both ends so far as transverse flexure is concerned, their points of inflection are mid-way between the floor beam and the bracket or about 17.'0 from D and D'. The horizontal reactions may therefore be considered as applied



at those points. The vertical reactions equal $2 \times 1800 \times 32 \div 16 =$ 7200 pounds. The bending moment to be resisted by the bracket is $M = 1800 \times 17 = 30600$ foot-pounds. The maximum bending moment in the post is at L and equals $1800 \times 12.5 =$ 22500 foot-pounds. That in the strut II is at K, when the forces on the left are considered, and equals $1800 \times 17 -$ $7200 \times 11.5 = 52200$ foot-pounds. The direct stresses in the members are given in the preceding table. The computations just made assume that the upper lateral system is not acting. This assumption is usually made to furnish some convenient basis for ascertaining the additional stresses to be used in proportioning the members which are employed to secure the transverse stiffness of the bridge.

The portal bracket may be made so that the vertical projection of a tangent to its inner edge has a slope of about 45 degrees. It is estimated that the lower end of the portal strut is about 8.0 from the upper pin of the inclined end post. The post is 40.0 long. It is proposed to use an end floor beam and to attach it rigidly to the end post. This will extend to a point about 2.5 above the lower end of the post. If the point of inflection be taken half-way between these points it will be about 22.75 from the upper end of post. The wind load at the windward end of the portal strut is 10800 pounds and that at the leeward end is 1800 pounds. Assuming both supports to react equally each horizontal reaction is 6300 pounds and the reactions coinciding with the axes of the end posts are equal to $12600 \times 40 \div 16 = 31500$ pounds. The maximum bending moment in the end post is then $6300 \times (22.75 - 8.0) = 92925$ foot-pounds. The bending moment in the portal strut is a maximum at the end of the portal bracket, which may be taken at 3.5 from the middle of the strut. At this point its value is $6300 \times 22.75 - 31500 \times 14.75 = 321300$ foot-pounds.

Sections of Intermediate Posts.

The column formulas given in the regulations are to each other as 1 to 2 for live and dead loads. It will therefore be most convenient to reduce the stresses to the equivalent live-load stress and then use the corresponding formula. The complete design of one post will be given and only the compositions of the others as the same method is employed in the design of all.

The data for the channels used in these posts was obtained from the Handbook of the Pencoyd Iron Works, page 154.

<u>Post c4</u>.

$$\text{Stress} = 151133 - \frac{56210}{2} = 133028 \text{ pounds.}$$

Length of post = 32' = 384". Let a section be tried consisting of two 15" channels with their webs parallel to the plane of the truss. The pocket book gives the following data concerning a form which is rolled to any weight between the limits given.

	Weight	Area	Moment of Inertia	Radius of gyration
	68.93	20.68	564.78	5.22
	47.83	14.35	428.74	5.47
Diff.	21.10	6.33	136.04	0.25

Let the value of r = 5.4 be tried. This gives an average equivalent live-load unit stress for the post of

$$7000 - 40\frac{384}{5.4} = 4155 \text{ pounds}$$ and the area of one channel of ½(133028 ÷ 4155) = 16.01 square inches. By interpolation the elements corresponding to an area of 16.01 square inches are, weight, 53.36; I = 464.4; and r = 5.4. As this value of r agrees with that assumed no modification

is necessary. The flanges of the channels will be turned inward as this gives the most economical section. The moment of inertia, I', of one channel with respect to its neutral axis parallel to the web is 18.35, the center of gravity being 0."9736 from the outside of the web. Let h be the distance from this neutral axis to a parallel axis through the center of the post, with respect to which the channel is to have a moment of inertia equal to I, and since $I = I' + Ah^2$ in which A is the area of the channel,

$$464.4 = 18.35 + 16.01 h^2$$

$$\therefore \quad h = 5.''28$$

The post would be equally strong in both directions under the given direct stresses when the distance back to back of channels is $d = 2(5.28 + 0.9736) = 12.''51$ provided the length of the column were the same for flexure in both directions. To make allowance for this we will make $d = 14''$. The arrangement of the lattice bars etc will be treated of later.

Post d6.

Two 12" channels; weight = 43.04; area = 12.33; $I = 221.16$; $r = 4.15$; and $d = 11''$

Post e8.

Two 10" channels: weight = 27.33; area = 8.20; $I = 103.46$; $r = 3.57$; $d = 10''$

Posts a2, b'2, c'4, d'6, e'8.

Two 9" channels: weight = 18.80; area = 5.64; $I = 57.9$; $r = 3.20$ $d = 8.''5$.

Sections of Ties and of Lower Chord.

The unit stresses used in the following designs will be
Dead Load = 16000 pounds per square inch
Live Load = 8000 " " " " assuming the wind load as tabulated to be a live load. Most of the tension members are made in the form of eyebars. The ratio of the thickness to the width of the body of the bar varies from about $\frac{1}{3}$ to $\frac{1}{8}$, the latter ratio being approached as the number of bars in the same panel increases. As in the design of the posts the equivalent live-load stress will be found and then the unit stress for live-load used to find the area required. The complete design of one member will be given and then the composition of the others as the design of all tension

members was made according to the same principles.

Lower Chord

Pieces 0-1, 1-2, 2-3, 3-4

$$\text{Stress} = 202634 - \frac{73999}{2} = 165634 \text{ pounds}$$

$$\text{Area required} = \frac{165.634}{8000} = 20.7 \text{ square inches.}$$

2 bars 6" x $1\frac{3}{4}$" = 21 square inches and these will be used.

Pieces 4-5 + 5-6

2 bars 8" x $2\frac{1}{4}$".

Pieces 6-7 + 7-8

2 bars 8" x $2\frac{5}{8}$".

Piece 8-9

2 bars 9" x $2\frac{7}{8}$".

Tension Vertical

b 2

2 bars 4" x $1\frac{1}{8}$".

Sub-Verticals

a-1, b'-3, c'-5, d'-7, e'-9.

2 bars 2" x $1\frac{1}{8}$".

Web - Members.

bb'

2 bars 7" x 2".

cc'

2 bars 6" x $1\frac{5}{8}$"

dd'

2 bars 5" x $1\frac{1}{4}$"

b'4

2 bars 7" x $1\frac{5}{8}$"

c'6

2 bars 6" x $1\frac{5}{8}$"

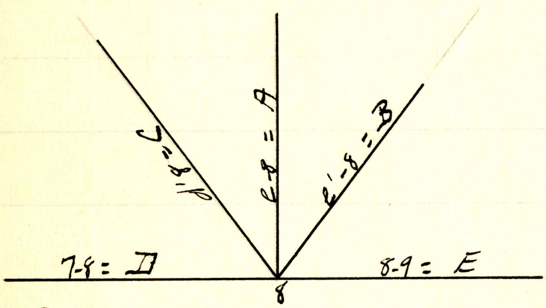
d'8

2 bars 5" x $1\frac{1}{4}$"

Pins and Eyebar Heads.

All of the pins will be made the same size and for the design of this size the stresses at the point 8 will be considered. Of course these stresses must be simultaneous. The method of design will be that laid down in Du Bois, page 307. The condition of the loading will be the second wheel of the first engine at the point 9. This loading gives the following stresses.

	Live Load	Dead Load	Maximum
e 8	− 35827	− 8750	− 44577
d'8	+ 44584	+ 25519	+ 70103
8-9	+ 153060	+ 177975	+ 331035
7-8	+ 114795	+ 160178	+ 274973
e'8	− 11875	− 4681	− 16556



In the following design the forces on one-half of the pin will be considered.

Horizontal Components.

The horizontal components on the left of 8 are $\frac{D}{2} + \frac{C}{2}\sin\theta$ and those on the right of 8 are $\frac{E}{2} - \frac{B}{2}\sin\theta$. We have $\frac{E}{2} - \frac{B}{2}\sin\theta = 160551$ pounds. The distance between the center of pressure of $\frac{E}{2}$ and $\frac{B}{2}\sin\theta$ and the center of pressure of $\frac{D}{2}$ and $\frac{C}{2}\sin\theta$ was found to be 2".4. In the calculation of this distance clearance was allowed between the

pieces as specified in "Roofs and Bridges" Part III, Art 54.
Therefore the horizontal moment is

$$M_H = 160551 \times 2.4$$
$$= 385322.4 \text{ inch-pounds.}$$

Vertical Components

The vertical components downward are $\frac{A}{2} + \frac{B}{2} \cos\theta$ and that upward is $\frac{C}{2} \cos\theta$. $\frac{A}{2} + \frac{B}{2} \cos\theta = 28910$ pounds and the distance between the centers of pressure of the upward and downward forces is 3."31. Therefore the vertical moment is

$$M_V = 28910 \times 3.31$$
$$= 95692 \text{ inch-pounds}$$

The resultant moment hence is

$$M = \sqrt{(M_V)^2 + (M_H)^2}$$
$$= 397100 \text{ inch-pounds}$$

From the table in Du Bois, p 309, it is seen that a 6" pin will do for this moment. Therefore use a 6" pin all through the bridge.

The specifications for the Eye Bar Heads are taken from Carnegie p 167.

Section of Upper Chord and End Post.

For ease of construction and to avoid having to make so many joints between members of different sizes, I will make the upper chord and end post of the same size and composition, taking the stresses as those in the member Ef. Of course this is on the side of safety since the stresses in all of the other members are less than those in this one. The upper chord usually consists of two vertical web plates with an angle iron at the outer top and bottom of each web, the whole united by a cover plate above and by latticing below. The web must be deep enough so that the head of the eye bar bb', which is placed on the outside of the chord, will clear the under side of the angle and also its rivets. Adding the radius of the pin, the half width of section of eyebar head, height of rivet head and thickness of angle, the result is $3.0 + 5.25 + 0.64 + 0.625 = 9.52''$. It is assumed that angles $5/8''$ thick will have to be used. The web then ought to be 20" deep. The clearance between the webs will depend upon the members at the panel point C. The post C4 measures 14" back to back of channels and its webs need pin plates whose thickness for moderate spans should rarely vary much from 3/8". An allowance for the space between shapes with countersunk rivets may be taken at 1/8". The tie cc' is 1 5/8" thick. The half clearance is then 7.0 + 0.375 + 0.125 + 1.625 + 0.125 = 9 1/4". If the distance between webs be made 19 1/4" there will be sufficient room. As pin plates should be placed outside of the vertical legs of the angles; no smaller size can be used than 4" x 3" the shorter side being placed horizontal. As some allowance must be made for the thickness of webs, the cover plate will be made 25 1/4" wide. The pins are placed in the neutral axis of the chord section or else secondary stresses will be caused. Convenience in manufacture also makes it desirable that the pins shall be in the center line of the pin plates. If the upper and lower angles have the same size, the pin plates and their fillers will fit well and the pins will be brought to the mid depth of the webs. To bring the center of gravity to this position flats are rivted to the horizontal leg of the lower angles. In this case two flats are rivetted to each lower angle of a thickness equal to that of the cover plate.

The clear width between the rows of rivets in a web

20" deep with 4" vertical legs of angles is

$20 - 2(2.25) - 1\frac{7}{16} = 14\frac{1}{16}$ and therefore the thickness must be at least $\frac{1}{2}$". The allowable clear distance between the rivet lines of a $\frac{1}{2}$" cover plate is $40 \times \frac{1}{2} =$ 20" and this will be used. The allowable unit stress involves the radius of gyration of the section to be designed. An approximate value may be found by taking four tenths of the depth overall. In this case $r = 0.4[0.5+20+1.5] = 8.8$". The length of the chord is 24' = 288" and the unit live load stress is $8000 - 30\frac{288}{8.8} = 7018$ pounds per square inch. The equivalent live load stress in ef is 492046 − $\frac{1}{2}$(177975) = 403059 pounds and the area required is $403059 \div 7018 = 57.43$ square inches.

The composition will be as follows:

1 cover plate	$25\frac{1}{4}$" × $\frac{1}{2}$"	=	12.625	square inches
4 angles	4" × 3" × $\frac{5}{8}$"	=	15.92	" "
2 webs	20" × $\frac{1}{2}$"	=	20.00	" "
4 plates	4" × $\frac{1}{2}$"	=	8.00	
Total		=	56.545	" "

Although the area is a little smaller than that required by the stresses nevertheless, considering the factor of safety assumed in taking this stress and the great margin that was allowed in computing the stresses, I think that this composition will be sufficient for the upper chord and end post.

Hangers

Where the floor beam is to be attached at the lower end of a tension vertical to fasten the floor beam to. These hangers will be made of plates. The size of the pin used at both ends of these suspenders will be of the same size as those in the rest of the bridge. From Carnegie, page 174, the bearing value of a 6" pin is 72000 lbs. for 1" thickness of plate. The maximum stress in the sub-verticals is 39 406 pounds. We will make the plate 1" thick, 12" wide, and 5'6" long. Its lower end is 9" below the center of the lower pin and the distance, center to center of pins, is 4' 1½". The bearing value of this plate will be more than enough for the floor beam attachment. In like manner the size of the hanger for b2 was computed. In this case in addition to the tension in b2 the downward components of the thrusts in a2 and b'2 were taken into consideration, making the stress = 109473 pounds. This hanger is 1¾" thick, 15" wide, 5'6" long, lower end is 9" below center of lower pin, and distance, center to center of pins, is 4' 1½"

Counters.

Counters will be placed in the middle panel and in two panels on each side of the middle one. The stress in ee' is (page 18) 49637 pounds. Therefore the area required is 49637 ÷ 8000 = 6.2 square inches. Use a square bar 2½" on a side and this will be the size of all of the counters. The specifications for the eye bar heads are obtained from Carnegie, page 167; and the specifications for the sleeve nuts from the pocket book of the Pencoyd Iron Works, page 240.

Lateral and Transverse Bracing.

The upper lateral ties will consist of bars of square section. The wind stress in the end tie is 19472 pounds, and the unit stress of 15000 pounds per square inch requires an area of 1.298 square inches. A bar $1\frac{3}{16}''$ square will be used giving an area of 1.4 square inches. All of the upper lateral ties will be made of this size.

The lateral struts will be composed of four angles united by vertical latticing, the upper angles extended across and riveted to the cover plates of the chords, and the lower angles resting on top of the horizontal leg of the lower angle of the chords. The lateral ties will also be attached to the cover plates and hence the direct compression in the struts must be taken by the upper angles. The thickness of the latticing depends on the bearing strength of the rivets needed to transmit the vertical shear. This shear is equal to 7200 pounds and for a single system of lattice bars inclined 30 degrees with the vertical the stress is $7200 \times 1.155 = 8316$ pounds in each bar. As $\frac{3}{4}''$ rivets will be used in the latticing this stress requires two rivets in a $\frac{1}{2}''$ plate, the bearing strength of one rivet in a $\frac{1}{2}''$ plate being 4500 pounds. The radius of gyration for two angles $3\frac{1}{2}'' \times 2\frac{1}{2}''$ with the longer legs horizontal and the backs separated $\frac{1}{2}''$ is about 1.86 with reference to the vertical axis midway between them.

Using this value the allowable unit stress is $9000 - 50\frac{192}{1.86} = 3840$ pounds per square inch, and the area required is $9000 \div 3840 = 2.34$ square inches. Two angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ give an area of 2.88 square inches. The bending moment at 4.5 from the center line of the truss is 52200 foot-pounds (page 20). The effective depth of the entire strut is 23" if the same angles be used above and below. Taking the unit tensile stress of 15000 lbs. the net area in one flange is $\frac{52200 \times 12}{23} \div 15000 =$ 1.82 square inches and for the unit compressive stress of 10500 lbs. the gross area is 2.60 square inches. Two angles $3'' \times 2\frac{1}{2}'' \times \frac{1}{4}''$ have a net area of $2(1.31 - 0.19) = 2.24$ square inches and a gross area of 2.62 square inches and will be chosen for both flanges.

Before proceeding further let the side elevation of the truss members be drawn whose sections have been designed and also a section showing the elevation of an intermediate strut and a floor beam with the post connecting them. The bottom line of the floor beam will be placed $10\frac{1}{2}''$ above the center line of the pins. After this the computation and drawing will progress together.

By drawing the bracket connecting the lateral strut and post the lever arm of its flange is found to be about 2.25 and its stress $30600 \div 2.25 = 13600$ pounds. The net section must be (§37) $13600 \times 1.8 \div 15000 = 1.63$ square inches. The length of the flange is about 6 feet. For two angles $2\frac{1}{2}" \times 2\frac{1}{2}"$ & $\frac{5}{16}"$ web between them r is at least 1.17, since no angle thinner than $\frac{1}{4}"$ is allowed. The unit stress is $10500 - 60\frac{72}{1.17} = 6810$ pounds per square inch and the gross area $10800 \times 1.8 \div 6810 = 2.85$ square inches. The angles must therefore be $\frac{5}{16}"$ thick giving a gross area of 2.94 square inches and a net area of $2(1.47 - 0.27) = 2.4$ square inches.

As the panel length is not large the lower lateral ties will be designed to take tension only as I think the floor beams will be strong enough to take all of the compression due to the wind. The ties, as in the case of the upper lateral ties, will consist of bars of square section. The unit stress in the pieces is taken at 15000 pounds per square inch. The method of design is the same as that used for the upper lateral ties so that only the sizes of the members will be given.

<u>0"-1</u>	1 bar	2"	square.
<u>1"-2</u>	1 "	$1\frac{7}{8}$"	"
<u>2"-3</u>	1 "	$1\frac{3}{4}$"	"
<u>3"-4</u>	1 "	$1\frac{5}{8}$"	"
<u>4"-5</u>	1 "	$1\frac{1}{2}$"	"
<u>5"-6</u>	1 "	$1\frac{3}{8}$"	"
<u>6"-7</u>	1 "	$1\frac{1}{4}$"	"
<u>7"-8</u>	1 "	$1\frac{1}{16}$"	"
<u>8"-9</u>	1 "	1"	"

The portal will consist of a small lattice girder of four panels with a double system of webbing united by brackets with solid webs to the end posts. By drawing a side elevation of the end post and upper chord it is found that there will be plenty of room over and above the clear head room required to construct a portal of a convenient depth, since the bridge is 36' deep. Assume the distance between the rivet lines in the flange angles which is the effective depth to be 50". The maximum moment will be taken as that found on page 21 and equals 321300 foot-pounds. The flange stress is hence 321300 x 12 ÷ 50 = 77112 pounds. The net section of the lower flange is 77112 ÷ 15000 = 5.14 square inches. Assume the unit stress in compression to be 8000 pounds per square inch. This requires an area of 77112 ÷ 8000 = 9.64 square inches. Two angles 5" x 4" x 5/8" (gross section = 10.46 square inches) will be used for both flanges the longer legs being placed normal to the plane of the portal.

The vertical shear is 31500 pounds and as the braces make an angle of approximately 45 degrees with a line parallel to the axis of the end post the stress in each brace is 1/2(31500) 1.414 = 22270. As this requires five 3/4" rivets in each end of the brace for the thickness of the web, a wide angle will be used so as to allow two rows of rivets thus reducing the size of the connecting plates. An angle 4 1/2 x 3" x 1/2" will allow of of two rows of rivets and has ample strength when treated as a column.

The perpendicular struts in the girder will be made of two 3" x 3" x 1/2" angles.

The flange of the bracket has a lever arm of about 40" with a center of moments about 3' below the upper pin in the end post. The flange stress hold in equilibrium the moment of the horizontal reaction of the end post which may be considered as applied at the point of inflection. Its value is $6300(22.75-3)12 \div 40 = 37330$ pounds. Treating the flange as a column about 7' long similar to that of the intermediate bracket the area is found to be 8.78 square inches. Two angles $5" \times 5" \times \frac{1}{2}"$ give 9.50 square inches and will also have an excess of net section. As this design has not taken into account the pin which is at the middle of the end post and as this would probably materially alter the above figures by bringing the point of inflection nearer the upper pin, I think that the above design is on the side of safety and will be sufficiently strong. The thickness of the web for the bracket is 3/4".

Pin Plates

Owing to the fact that the upper chord is of the same size in all of the panels, I will make the pin plates of the same size at the intermediate points c, d, + e. The allowable bearing per linear inch for the 6" pin is 72000 pounds. No plate less than 3/8" thick will be used. The linear bearing for each side of de is $\frac{1}{2} \times 492046 \div 72000 = 3.''42$. The pin plates will be the same on the outside and on the inside of the chord: this requires $1.''71$ thickness on each side. One plate will be used 5/8" thick between the lower edges of the vertical legs of the angles and one 1 1/8" thick over this and extending up to the bottom of the horizontal legs of the angles. The first one will extend 14" on each side of the center line of the pin and the second one will extend 8" on each side.

The pressure of the pin against the lower part of the post c4 is equal to the vertical component of the stress in b'4 or $243312 \times \cos\theta = 194650$ pounds. The linear bearing for each side of the post is $\frac{1}{2} \times 194650 \div 72000 = 1.''35$ and a 3/4" plate will be used inside and outside. This will have its lower edge six inches below the center of the pin and contain 2 rivets in each vertical row. The upper edge will extend 17" above the center of the pin and have 5 rivets in each vertical row.

The same size plates will be used for the upper end of the post.
The only difference that will be made in the other vertical posts will be in the thickness of the plates, the length and rivetting remaining the same as for C4. The plates for d6 are $\frac{1}{2}$" thick. The plates for E8 are $\frac{3}{8}$" thick.
For the slanting posts the plates are $\frac{1}{4}$" thick and only on outside. The lower edge is 6" below the center of the pin and contains 2 rivets in each row, and the upper end is 14" above the center of the pin and contains 4 rivets in each row. Both of the last two measurements are made on the center line of the post.
The required bearing of the upper chord and of the end post on the pin at b are 3".42 and 2".46 respectively, on each side. Two of the outside pin plates of the chord and two of the inside pin plates of the end post are to be extended past the pin for the purpose of preventing any ordinary blow from displacing these members as well as to facilitate erection. These plates are called hinge plates or jaw plates. The other plates are faced parallel to the bisector of the angle between the axes of the members and at a distance of $\frac{1}{8}$" from it. In arranging

the pin plates 1/8" clearance is allowed between each hinge plate of one member and the nearest pin plate on the other. The webs are directly opposite being of the same thickness, and the filler plates (between the angles) are made of the same thickness as the angles. The pin plates at this point will be of the same thickness on both members. These will be one plate 5/8" thick and three plates 1" thick. Three of those on the chord and two of those on the post are on the outside. The order of the plates for the chord will be as follows: commencing at the outside, there is one on the outside of web, then one on inside, then two on the outside the innermost one being the filler plate which is the longest of all; keeping the same order as above the ends of these will be distant from the center line of the pin 12", 17", 25", and 33". Two extra rows of rivets are put in the web part of the chord. The order of the plates for the post will be as follows: [the order will always be given commencing at the outside and from the shortest to the longest]. 1 outside, 2 inside, 1 outside and the lengths will be the same as the corresponding ones for the chord.

The plates for the point a are to be 2".46. They will consist of four 5/8" plates: one inside and the rest outside, the one inside being the second one in the order. The distances of the ends from the center pin are 9", 14", 19", and 24".

At the point o the bearing required for the end post is about 2.52 on each side, as an additional vertical load of 8730 pounds is transmitted by the end floor beam. This consists of a live load of 6000, half a stringer and the track it carries, and half a floor beam, the position of the live load being that which produces the maximum stress in oa. The forked end of the post will extend 15" from the center of the pin to satisfy § 89. Four 5/8" plates will be used, one being a hinge plate: one outside, hinge inside, and two outside and the distances from the center of the pin are the same as at a.

Stay Plates and Latticing.

Compression members united by latticing have plates at the end whose purpose is to aid in properly dividing the stress between the two segments of the member. These are called stay plates, or batten plates, or tie plates.

According to the specifications the plates for the upper chord and end post will have to be 3' 2" long.

Panel bc. The left end of the left plate will be placed 1' 3" from the center line of the pin and the right end 4' 5" from the ℄ of pin. The left end of the right plate will be 3' 11" from ℄ of pin and the right end will be 9" from ℄ of pin. The lattice plates will be $4\frac{1}{2}" \times \frac{3}{8}"$. The pitch lines of rivets in the angles are distant apart $19\frac{1}{4} + 1 + 2(2\frac{1}{4}) = 24\frac{3}{4}$ The pitch of the lattícing in the following design will mean the horizontal distance between the two ends of a bar and each bar will be called a panel. If the pitch of the latticing be made 12", 15 panels can be used if the end rivets of the latticing be placed 4" from the ends of the stay-plates In the chord and the end post two rivets will be in the end of the bars.

Panel cd. Left plate: left end, 1' 3" from ℄ of pin: right end 4' 5" from ℄ of pin. Right plate: right end 1' from ℄ of pin; left end 4' 2" from ℄ of pin. Pitch = 12". Panels = 15. End rivets $2\frac{1}{2}"$ from end of plates.

Panels de and ef. Same as for cd and bc respectively, only in ef the left end of the left plate and the right end of the right plate are each 1' from ¢ of corresponding pin but the latticing is the same.

<u>Panel ab</u>. Upper plate: upper end 2' from ¢ of pin: lower end 5' 2" from ¢ of pin. Lower plate: lower end 4" from ¢ of pin: upper end 3' 6" from ¢ of pin. Pitch = 12". Panels = 11. End rivets 2" from ends of plates.

<u>Panel oa</u>. Upper plate: upper end 1'6" from ¢ of pin: lower end 4'8" from ¢ of pin. Lower plate: lower end 1'6" from ¢ of pin: upper end 4'8" from ¢ of pin. Pitch = 12". Panels = 10. End rivets 4" from ends of plates. The rest of the latticing will have only one rivet in the end of the bar.

<u>Posts a2, b'2, c'4, d'6, e'8</u>. Length of plate = 1'1". Upper plate: upper end 1' from ¢ of pin: lower end 2'1" from ¢ of pin. Lower plate: lower end at end of post: upper end 7" from ¢ of pin. Latticing = 3" x 1/4". Pitch = 3". Panels = 68. End rivets 2" from ends of plates.

<u>Post c4</u>. Length of plates = 1'9". Upper plate: upper end 1'8" from ¢ of pin: lower end 3'5" from ¢ of pin. Lower plate: lower end 3' from ¢ of pin: upper end 4'9" from ¢ of pin. Latticing = 3" x 3/8". Pitch = 6". Panels = 47. End rivets 3" from

ends of plates

Post d6. Length of plates = 1'5". Upper plate; upper end 1'6" from ℄ of pin; lower end 2'11" from ℄ of pin. Lower plate; lower end 3' from ℄ of pin; upper end 4'5" from ℄ of pin. Latticing = 3" x 3/8". Pitch = 5". Panels = 5 8. End rivets 3" from ends of plates.

Post e8. Length of plates = 1'3". Upper plate; upper end 1'6" from ℄ of pin; lower end 2'9" from ℄ of pin. Lower plate; lower end 3' from ℄ of pin; upper end 4'3" from ℄ of pin. Latticing = 3" x 3/8". Pitch = 4". Panels = 7 4. End rivets 2" from ends of plates.

Connections of Wind Bracing.

The lateral rods of both systems will be connected to the trusses by means of clevises of standard form.

Upper Lateral System. For a bar $1\frac{3}{16}$" square the Pocket Companion gives the size of the pin as $2\frac{1}{4}$". The thickness of the connecting plate will be 19472 ÷ 27000 = 3/4". The plate must have 19472 ÷ 4510 = 4.3 rivets connecting it to the cover plate and angles of the chord but 6 rivets will be used as shown on the drawing.

The web of the bracket of the portal is made $3/4''$ thick and extended upward through the lower flange of the strut and united not only to the flange angles but also to the web members. It is fastened to the middle of the inner web of the end post by two angles $4\frac{1}{2}'' \times 4'' \times \frac{1}{2}''$, their longer legs being in contact with the web of the end post.

Lower Lateral System. The connecting plates for the lower laterals will be connected to the bottom of the floor beam.

0''-1.	Pin = $2\frac{1}{4}''$.	Plate = $2\frac{1}{8}''$.	Rivets = 14.
1''-2.	Pin = $2\frac{1}{4}''$.	" = $1\frac{15}{16}''$.	" = 13
2''-3.	Pin = $2\frac{1}{4}''$.	" = $1\frac{3}{4}''$.	" = 11
3''-4.	" = $1\frac{7}{8}''$	" = $1\frac{1}{2}''$.	" = 10
4''-5.	" = $1\frac{7}{8}''$	" = $1\frac{1}{4}''$.	" = 9
5''-6.	" = $1\frac{7}{8}''$	" = $1''$.	" = 8
6''-7.	" = $1\frac{7}{8}''$	" = $\frac{7}{8}''$.	" = 7
7''-8.	" = $1\frac{7}{8}''$	" = $\frac{3}{4}''$.	" = 6
8''-9.	" = $1\frac{7}{8}''$	" = $\frac{1}{2}''$.	" = 5

Pedestal, Expansion Rollers, and Bed Plate.

The pedestal is constructed by riveting the vertical pin plates, which receive the entire reaction at the support, to a horizontal bearing plate by means of four angles, 4" x 4" x 5/8"; two being inside and two outside. A thick angle is taken in order to aid in distributing the pressure over the bearing plate. The bearing plate at the expansion end rests on a set of friction rollers placed in a frame so as to maintain their spacing. The rollers move on a "rail plate" which itself is supported by the masonry.

The allowable pressure per linear inch for rollers 4" in diameter is $500\sqrt{4} = 1000$ pounds and the maximum reaction of the leeward support at one end is 269005 pounds. This requires 8 rollers 34" long which is satisfactory as it places the pin plates very nearly at the quarter points of the rollers. Allowing a clearance of 3/8" between the rollers and an additional space of one inch for the middle one of three connecting rods of the frame, the size of the bearing plate will be 37" x 34". If the bed plate were no larger than this it would reduce the pressure on the support to $269005 \div (37 \times 34) = 214$ pounds per square inch, which is far within the specified limit. The frame consists of two bars 5" × ½" which receive the journals of the rollers and are bolted together by three parallel rods ⅞" in diameter. The rollers are supported on a series of 16 parallel rails rivetted to a bed plate 1" thick. The rails are of 50-pound standard section, with their heads brought to rectangular form by planing on the top and sides. One side of the base is removed almost entirely in order to space the rail head with a clearance of ½", the other side being fastened to the bed plate by 7 rivets with a pitch of 5". The bearing plate is made 1½" thick and to provide against lateral sliding the vertical bars are allowed to project ½" above the surface of the rollers and the same distance below the tops of the rails. The hinge plate on the inside of the web will be extended to have the form of the dotted lines from the right line of the web to the left line of the web produced as shown on the drawing. The outside pin plates will be cut off in the direction of the two parallel dotted lines and will abut against a plate 2" thick which will be rivted along with the web of the post and the hinge plate to the bearing plate. This 2" plate will be rivetted to the web of the floor beam between the pin and the angles connecting it to the bracing plate to hold it in place. Where the web of the post does not extend between the hinge plate and the outer plate, washers $\frac{1}{2}$" thick and 2" outer diameter will be placed.

Minor Details.

The end floor beam will be of the same size as the others which is far on the side of safety as the maximum load carried by the end floor beam is about two thirds of that for the intermediate ones. The lower flange of the floor beam will be extended only far enough to abut against the inside web of the end post and be rivetted to a horizontal angle $2\frac{1}{2}$" x $3\frac{1}{2}$" x $\frac{1}{2}$" which is rivetted to the inner web of the post with its longer web horizontal. One rivet will connect each angle of the flange of the floor beam to this angle. The web of the floor beam will be cut away enough to allow room for the inside flange angle of the end post. Rivetted to the inside of each web of the post will be a plate of triangular form and 1" thick, extending out to the line of the inside edge of the web of the floor beam. For the rivetting for these triangular plates the rivets which unite the pin plates together will be made long enough to include this plate also. This plate will extend down far enough so as to include the rivets in the underneath flange of the post. The part of the web of the floor beam which is above the post and the outside angle of the upper flange will be extended and united to each of these plates by two angles $3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{1}{2}"$ with pitch of rivets 3". The cover plate of the post will be discontinued at the upper edge of the triangular plates and a triangular cover plate will be put on as shown and rivetted to each of the plates by means of the same size angles as were used in the construction of the end post. It will also be rivetted to the flange of the floor beam which was extended.

The upper chord will be spliced at the left of the points c, d, and e, and since the abutting ends will be planned the splices need only be sufficient to keep all of the parts in contact. These splices will be made of the same width as the cover plate of the upper chord and will be $\frac{3}{4}"$ thick and 4' long: the right hand edge in each case will be 6" from the center of the pin on the right and the right end will be of the form to act as the connection of the wind bracing also. A splice plate of triangular form will be put at b. This will be 3/4" thick and will extend 1' on each side of the point as shown.

The posts will be braced inside opposite the point where the floor beam is attached. This will be done by a diaphragm of a 1/2" and four angles 2 1/2 × 2 1/2 × 1/2" and will extend from a point 3' above the center of the pin to a point 9" above the center of the pin.

The camber will not be calculated. If it is required it can be found by aid of § 109 of the specifications. But in this case as the number of pieces in the bridge were so many and the computation would be so difficult for some of them I did not think it necessary.

The size of the eye bar heads would have been greatly diminished and thus the drawing made to look much nicer had I taken a larger number of bars in the members than I did. But the present arrangement is strong enough and the only advantage that would have been gained would have been in the drawing.

Respectfully submitted
Lynwood Ruff Holmes.