Coulter

Design of a Single Track, Through, Pin Connected Pratt Truss.

Length 160 Feet. 8 Panels.

Thesis for B. S. Degree —

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— Introduction —

I have taken the design of a single track through railway bridge, having pin connected trusses of the Pratt type with straight upper chord.

The span is 160 feet, divided into 8 panels each 20 feet long. The depth is 30 feet and the distance center to center of trusses is 17 feet.

The design was made in accordance with the general specification of the Seaboard Air Line Railway Co, Bridge specification of 1908.

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

"Merriman & Jacoby "Roofs and Bridges", Parts I, II & III,

Cyclopedia of Civil Engineering, American School of Correspondence,

Sections and prices in accordance with Cambria Steel Edition 1909.

Floor System

<u>Cross Ties</u> (§§ 38, 39, 40 & 41 of Specification).
Each Cross Tie to be 8"x8" and 10' long and quality known as "Prime Inspection" Interstate Rules 1905, and of longleaf yellow pine, ties to be spaced 6" apart in the clear. They will be notched to fit tight over supporting stringer to a depth of 1 inch, with semi-circular grooves cut in them of minimum size necessary to clear rivet heads. The ties will be extend 3 ties beyond the end of the steel work. Every fifth tie to be fastened to the stringer by a $\frac{3}{4}$" bolt. The ties will be placed up close to the cover plate of the floor beam but it will not be necessary to place any on top of the floor beam.

<u>Guard Rails</u> (§§ 42 & 43)
Guard Rails will be 7"x8" laid with 8" face down notched to 5" over each tie, laid parallel with the rails and with the inner face 3'6" from the center line of the track. Each guard rail to have a length of 16' and will be spliced together with a half and half joint directly over a tie. Guard rails will be secured at each splice and at ~~each~~ every fourth tie between splices by a $\frac{3}{4}$" bolt having a flat button head 2" in diameter on top of the guard rail a square shoulder 2" long under the head, a threaded end 2.5" long, a rolled steel washer $\frac{1}{2}$" thick curved to fit the tie and the flange of the support and a "Columbia Improved" lock nut on the bottom. These bolts will be driven through $\frac{3}{4}$" holes in the guard rails and ties and nuts screwed on tight

Halfway between the bolts and also in ~~each~~ first tie each side of a splice in the guard rail there will be driven through guard rail and tie a $\frac{1}{2}$"x10" boat spike thereby securing each alternate tie by a guard rail bolt or boat spike. Both bolts and spikes will be driven into the same tie through each guard rail so that alternate ties shall be held at each end.

Inside Guard rails. (§§ 46)

These will be ~~placed~~ laid parallel to main track and spaced 8" inside of main rail.

Stringer (§§ 56, 57, 59 + 61).

Each stringer is a plate girder of 20' span. They are to be spaced 6'6" ~~off~~ apart center to center. The whole number of stringers is 16. The loading used to determine the cross section of each stringer is that specified in §§ 114, 115, 116. Rails and their fastenings taken as 100 lbs per linear foot of track. Timber $4\frac{1}{2}$ lbs per foot board measure. For a rough estimate of the weight of one stringer M & J R & B Pt III gives $6.3 l^2$ where l = length of stringer. The total weight stringer is $6.3 \times \overline{20}^2 = 2520$. 6000 lbs will be used as weight of stringer. Dead load on one Stringer is:

Wt of stringer	6000	lbs.
" " cross ties $\frac{1}{2}$ of 17 each 10' long	2040	"
" " rails and fastenings $\frac{1}{2}(100 \times 20)$	1000	"
" " Inner Guard rail & fastenings	1000	"
" " Outside " " + "	420	"
Total Weight	10460	lbs

Stress Sheet.

	End Post	Upper chords		
Stress due to	aB	BC	CD	DE
Dead Load	-92.40	-97.02	-121.28	-129.36
Live Load	-271.00	-248.58	-306.85	-329.85
Impact	-202.10	-178.80	-214.90	-194.60
Wind Overturning				
On Truss East	-22.20	-12.30	-12.30	-12.30
" " West	+22.20	+12.30	+12.30	+12.30
" Train East	-22.70	-21.60	-27.00	-28.80
" " West	+22.70	+21.60	+27.00	+28.80
Wind on Truss East		-8.82	-14.112	-15.876
" " " West		+8.82	+14.112	+15.876
" " Train East				
" " " West				
Maximum	-610.40	-567.12	-692.442	-713.786

	Lower chords			Diagonals		
Stress due to	ab= bc	cd	de	Bc	Cd	De
Dead Load	+56.60	+97.02	+121.28	+66.15	+39.62	+13.23
Live	+150.81	+248.58	+306.85	+205.12	+146.42	+95.74
Impact	+144.60	+178.80	+214.90	+155.10	+126.40	+66.81
Wind Overturning						
On Truss East	+12.30	+12.30	+12.30			
" " West	-12.30	-12.30	-12.30			
" Train East	+12.60	+21.60	+27.00	+16.20	+9.70	+3.24
" " West	-12.60	-21.60	-27.00	-16.20	-9.70	-3.24
Wind on Truss East	+21.168	+26.46	+28.224			
" " " West	-21.168	-26.46	-28.224			
" " Train East	+63.508	+79.38	+84.672			
" " " West	-63.508	-79.38	-84.672			
Maximum	+402.99	+664.14	+795.23	+442.77	+322.14	+179.02

	Verticals				
Stress due to	Bb	Cc	Dd	Ee	
Dead Load	+14.00	-38.5	-17.5	-7.00	
Live Load	+81.25	-122.47	-79.69	0.00	
Impact	+69.29	-74.36	-65.34	-7.00	
Wind Overturning					
On Truss East					
" " West					
" Train East	+5.4	+13.5	+8.1	+5.4	
" " "	-5.4	-13.5	-8.1	-5.4	
Wind due Truss East					
" " " West					
" " Train East					
" " Train West					
Maximum	+169.94	-248.73	-170.63	-19.4	

The live load transferred to the leeward stringer by the wind pressure on the train is $10 \times 30 \times 20 \times 8 \div 6.5 = 7485 = 7490$ lbs. As the stringer is riveted to the floor beam throughout the depth of the stringer the lever arm of the wind pressure will be about 8'.
The maximum moment due to the dead load and the load transferred by the wind is $(10460 + 7490) \frac{20}{8} = 44,875$ lbs feet $= 538,500$ lbs".

Live load Stress in Stringer

Distance of Section from end in feet	Wheel at section	Max Bend Moment lb feet	Wheel at Section	Max Vertical shear lbs.
0.0		0	2	62,500
2.50	2	125,000		
5.00	2	187,500		
7.50	3	250,000		
8.75	3	257,813		
10.00	3	250,000		

The moment at 8.75' from the end due to dead load and wind load is 44,173 lb-feet. Therefore the maximum bending moment is $(257,813 + 44,173 + 241,570) = 543,557$ lb feet $=$ 6,522,700 lb inches. 241,570 is the moment due to impact $= (257,813 \times 0.937)$ where .937 is the coefficient of impact.
The depth of the web will be 32". Distance back to back of angle will be taken as $32 + 2 \times \frac{1}{8} = 32.25$".
($2 \times \frac{1}{8}$ due to overlapping of angles).
Maximum moment = 6,522,700 lb. inches.
$\frac{6,522,700}{32.25 \times 17000} = 11.897 = 11.9$ inches^2, approximate net ~~area~~ flange area required.
The stringer is designed assuming that the flange will take all the moment and the web take all

the shear.

$\frac{11.9}{2} = 5.95''^{\square}$ Half to be taken by angle and other half to be taken by cover plate.

$\frac{5.95}{2} = 2.975 \text{ in}^{\square}$ = Net area of one angle.

A $5 \times 5 \times \frac{7}{16}''$ angle gives a gross area of 4.19 in^2 and a net area of $4.19 - 2(\frac{7}{8}+\frac{1}{8}) \times \frac{7}{16} = 4.19 - 0.88 = 3.31 \text{ in}^2$.

0.88 in^2 deducted on account of rivets the rivet hole to be made $\frac{1}{8}''$ larger than rivet.

Net area of cover plate $= 11.9 - 2 \times 3.31 = 5.28 \text{ in}^2$

A $13''$ cover plate will be used. On account of the two rivet holes to be deducted, the net or real width of the cover-plate $= 13 - 2 \times 1 = 11$ inches. Thickness of cover plate to be $\frac{5.28}{11} = .48$ inches, a $\frac{1}{2}''$ thick plate will be used.

Approximate section of flange at the centre is

2 angles $5 \times 5 \times \frac{7}{16}''$	$= 2 \times 3.31 =$	6.62 in^2
Cover plate $(11 \times .5) =$		5.5 in^2
	Total Net Area.	12.12 in^2

In computing the centre of gravity the axis is taken at the centre of the cover plate

The distance of the centre of gravity of the angle from the back is 1.41 in (Cambria)

The distance of the centre of gravity from centre of the cover plate is $1.41 + (.5 \div 2) = 1.66$ inches.

Gross area of angles $(2 \times 4.19) =$		8.38 in^2
" " " cover plate $(13 \times .5) =$		6.5 " ^{2}
	Total	14.88 " ^{2}

The centre of gravity is now found to be $(8.38 \times 1.66) \div 14.88 = 0.9349''$ from the centre of the cover plate and $(.9349 - \frac{.5}{2}) = .68$ inches from the back of the angle.

The effective depth, h_e, is $32.25 - 2 \times .68 = 30.9$ inches and the required flange area is $\frac{6,522,700}{30.9 \times 17000} = 12.42 \text{ in}^2$

A total of $12.72\,in^2$ is given by the section approximately designed and the difference between that and the section as above determined is $(12.42 - 12.12) \div 12.42 = 2.3\%$ This is not an excess of $2\frac{1}{2}\%$ and will therefore be used.

Depth of web = 32" Effective depth 30.9" Unit stress allowed on webs by specifications (§§ 137) is 10,000 lbs per sq inch.

The live load shear on the web is	62,500 lbs
Shear due to impact Cof I.937	58,565 "
Shear due to dead load and wind	8,975 "
Total Shearing Stress	130,040 "

The web must therefore be $\frac{130,040}{10000} = 13$ Sq in

$(30.9 - 2) = 28.9$, 2 inches deducted on account of rivet holes $28.9 \times \frac{1}{2} = 14.45$ Therefore the web will be $32" \times \frac{1}{2}"$.

Stiffners.

Theoretically no stiffners are required except at the ends but the angle now used to connect the stringer to the floor beam will act as stiffners According to specifications (§§ 61) stiffners will be placed at a distance of 32" from each end These will be made up of

2 plats $4" \times \frac{1}{2}" \times 30\frac{1}{2}"$

2 fillers $5 \times \frac{7}{16} \times 22$

There will be 10 rivet holes for $\frac{7}{8}$" rivets in each

Composition of the connecting angle now. (§§ 59)

2 angles $6 \times 6 \times \frac{5}{8}$ 6" leg against web of floor beam

2 fillers $8 \times \frac{7}{16}$ with 8 rivets.

The maximum live load moment at section 3, (3) ft from end ((wheel 2 at section) is 142,500 lb-ft

Moment at the same section due to dead

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and wind load is 22,887 lb ft. The total moment is 165,387 lb ft = 1,984,644 lb inches at the section. The flange area required at the section is $\frac{1984644}{17000 \times 30.9}$ = 3.78 sq in. Therefore the area of the flange is $2 \times 4.19 = 8.38$ in^2 4.19 = area of 1 angle $5 \times 5 \times \frac{7}{16}$", if there is no cover plate. The cover plate of the top flange will be continued to the end but the lower one will be cut off at 3' from the end as the section will readily permit of this as shown above.

Spacing of Rivets

Distance of section from end. ft.	Wheel at Section	Max Shear due to live Load lbs.	Shear allowed for Impact lbs.	Shear due to dead & wind Load lbs.	Total Shear lbs.
0.0	2	62,500	58,563	8975	130,040
2.5	2	50,000	46,850	6732	103,585
5.0	2	37,500	35,138	4488	77,125
7.5	2	28,125	26,354	2244	56,725
10.00	2	17,500	16,398	0	33,900

Section	Total Shear lbs.	he inches	v lbs	Spacing $S = \frac{v\,he}{V}$ inches
0	130,040	30.9	10,500	2.52
2.5	103,585	30.9	10,500	3.17
5.0	77,125	30.9	10,500	4.25
7.5	56,725	30.9	10,500	5.72
10.0	33,900	30.9	10,500	9.57

v is the value of a $\frac{7}{8}$" rivet in bearing in a $\frac{1}{2}$" web.

The rivets will be arranged as shown in diagram
<u>Through the web</u>: First rivet $2\frac{1}{2}$" from ~~web~~ end of stringer Constant pitch of $2\frac{1}{2}$" up to 12th rivet = 30" then a constant pitch of 3" up to 23rd rivet = 33"; then a constant pitch of 4" up to 31st rivet = 32"; then a constant pitch of 5" up to 35th rivet = 20" The last rivet will come at the middle of the stringer. The length of the stringer is $20' - 2 - \frac{3}{8}" - \frac{1}{2}"$ = 240" - 1.25" = 238.75" The rivets in the web below are arranged in the same way -
Total number rivets in top web = 71 and total number in bottom web = 71.
<u>Through the cover plate above</u>:- First rivet half way between 1st + 2nd of web. The others have the same pitch as those through the web each one coming half way between. The rivets are staggered. Since the 36th rivet of the web is in the middle the total number of each side of the cover plate is 70. Lower cover plate will have a length of 168" ~~ft~~ = 14ft The rivets will correspond to the rivets in the top cover plate. The total number of rivets on one side is 17x2 = 34.
The pitch of the rivets in the stiffeners is constant.

<u>Lateral System of Stringers</u>

§§ 57 of specifications calls for a bracing. One lateral system of the "Warren type" will be use and the number of panels will be $2\frac{1}{2}$. The pieces will be designed

30" 33' 32" 20"

to take either tension or compression. The average depth of stringer and track is 41". Taking the wind pressure at 50 lb per sq ft on the stringer the total wind pressure $= \frac{41}{12} \times 50 \times 25 =$ 3417 lbs say 3420 lb. This gives a maximum stress of +4010 or −4010 lbs. For the ~~wind~~ end bracing this will be increased 100% which gives a total stress of 8020 lbs.

§§ 137 Compression $= \dfrac{17000}{1 + \frac{l^2}{11000 r^2}} = \dfrac{17000}{1 + \frac{(7.63 \times 12)^2}{11000 \, .86^2}} = 8018$ lbs in²

Assuming angle $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$" the smallest allowed by specifications, $l = \sqrt{(4)^2 + (6.5)^2} = 7.63'$
For a least radius of gyration = .68 (Carnegie p 177.)
$\frac{8020}{8018} = 1$ sq in = required area of angle. The angle $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ has an area of 2.49 in² which will be used. The length of these will be (7.6′ = 91″) 91″ − (2 × 5″) = 81′. 2 × 5″ deducted on account of the angles of the stringer so 78″ = 6′6″. They will be connected to the upper flange of the stringer by means of plates. The end plate will be 3/8" thick and have about 1 sq ft of area. It will cover 3 rivets in the flange and 3 rivets in the lateral. The other two connecting plates will cover 4 rivets in the flange and 3 in each of the two laterals and will have a thickness of 3/8" and area of 1 sq ft.

Wt of One Stringer

4 Angles	$5 \times 5 \times \frac{7}{16}$"	19′10¾″ long	@ 14.3 lbs per ft	= 1138.1 lbs.
1 Cover plate	$13" \times \frac{1}{2}$"	" "	22.1 " " "	= 439.7 "
1 " "	$13 \times \frac{1}{2}$	14′ "	22.1 " " "	= 309.4 "
1 Web plate	$32 \times \frac{1}{2}$	19′10¾″ "	54.4 " " "	= 1082.4 "
4 plates (stiffners	$4" \times \frac{1}{2}$"	30½"	6.8 " " "	= 69.2 "
4 fillers	$5" \times \frac{7}{16}$"	22 "	7.5 " " "	= 55.0 "

4 Connecting angles $6 \times 6 \times \frac{5}{8}$, $30\frac{1}{2}$" long	@ 24.2	=	246.1 lbs
4 Fillers $9 \times \frac{7}{16}$	22" " @ 13.4		98.3 "
Total			3438.2 "

Weight of one half Lateral System ~~$2\frac{1}{2}$ panels~~

$2\frac{1}{2}$ angles $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$, 6'6" long	@ 8.5 lbs	138.2 "
3 Connecting plates $\frac{3}{8}$" 3 sq ft	@ 15.3 " each	45.9 "
460 pairs rivet heads	@ 0.444 lbs.	204.3 "
Total		3826.6 "

This is much larger than estimated but the stringer will not be changed as the live load is so large in comparison with the weight of the stringer.

Floor Beam (Intermediate)

For the computation of stresses the span of the floor beam is 17' which is the distance center to center of trusses. Section IX of specifications was in the following design of floor beam. It will hold two concentrated loads each $3\frac{1}{4}$' from the center that is 6'6" apart beside weight of the floor beam itself.

Wt of one Stringer with $\frac{1}{2}$ Lateral System	4000 lbs.
" " $\frac{1}{2}$ of 17 ties each 10' long	2040 "
" " 2 rails and fastenings	2000 "
" " 1 guard rail (outside)	420 "
Live Load (max Floor Beam Reaction)	81250 "
Load due to impact (coef 0.946)	76865 "
Total Concentrated Load	166,575 "
Approximate wt of floor beam (4000 lbs)	2000
Total End Shear	168,575 "

Bending moment under the concentrated load of $166,575 = 166,580 \times 5\frac{1}{4} = 874,545$ lb ft.

Bending Moment at middle due to wt of floor beam =

3500 lbs.
The total bending moment at the middle of the floor beam is then for 878,045 lb ft = 10,536,540 lb ins.
The web will have a depth of 48 in making an allowance for clearance between cover of stringer and the flange angles of the beam.
Distance back to back of angle will be taken as $48 + (2 \times \frac{1}{8}) = 48.25$ in.
Approximate effective depth is $48.25 - 2 = 46.25''$.
Flange area required $= \frac{10.536,540}{17000 \times 46.25} = 13.4$ sq in.
Half to be taken by angles $\frac{13.4}{2} = 6.7$ the other half to be taken by cover plate. Area of one angle $\frac{6.7}{2} = 3.35$ sq inches. One angle $6 \times 6 \times \frac{3}{8}$ has an area of 4.35 sq in.
Net area of angle is $4.35 - (2 \times \frac{7}{8} + \frac{1}{8})\frac{3}{8} = 4.35 - .75 = 3.60$ in^2
" " " cover plate must be $13.4 - 2 \times 3.61 = 6.18$ in^2
The width of the plate will be taken as 14 in and its net width is $14 - (\frac{1}{8} + \frac{7}{8})2 = 12$ ~~$\frac{3}{4}$~~ in. The thickness of the plates is then $\frac{6.18}{12} = .515$ in say $\frac{1}{2}''$.
Approximate section at center is

2 angles $6 \times 6 \times \frac{3}{8}$ = 2×3.61		7.22 in^2
Cover plate $\frac{1}{2} \times 14$ = $\frac{1}{2} \times 12$		6.00 "
	Total	13.22 "

The distance of the center of gravity of the angles from back is 1.64 in. The distance of this center of gravity from the center is $1.66 + 0.5 \div 2 = 1.91$ inches.

Gross area of angles (2×4.36)		8.72 in^2
" " " cover plate $(\frac{1}{4} \times 14)$		7.00 "
	Total	15.72 "

The center of gravity is now found to be $(8.72 \times 1.91) \div 15.72 = 1.035$ inches from the back of the angles. The effective depth (h_e) is $48.25 - 2 \times 1.035 = 48.25 - 2.07 = 46.18$ inches and the required flange area is

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$\frac{10536,540}{17000 \times 46.18}$ = 13.42 sq in. (13.42 − 13.22) ÷ 13.42 = 1.5% and as this is less than $2\frac{1}{2}$% the section will be used.

Spacing of Rivets.

Section ft. from end.	Shear lbs.	Spacing S in inches
0 0	168,575	2.876
2.5	167,987	2.886
5.25	775	625.000
6.75	412	1150.000
8.5	0	∞

$S = \frac{v h_e}{V}$

v = value of a $\frac{7}{8}$" rivet in bearing in $\frac{1}{2}$" web = $\frac{7}{8} \times \frac{1}{2} \times 24000$ = 10500 lbs.

h_e = 46.18 inches

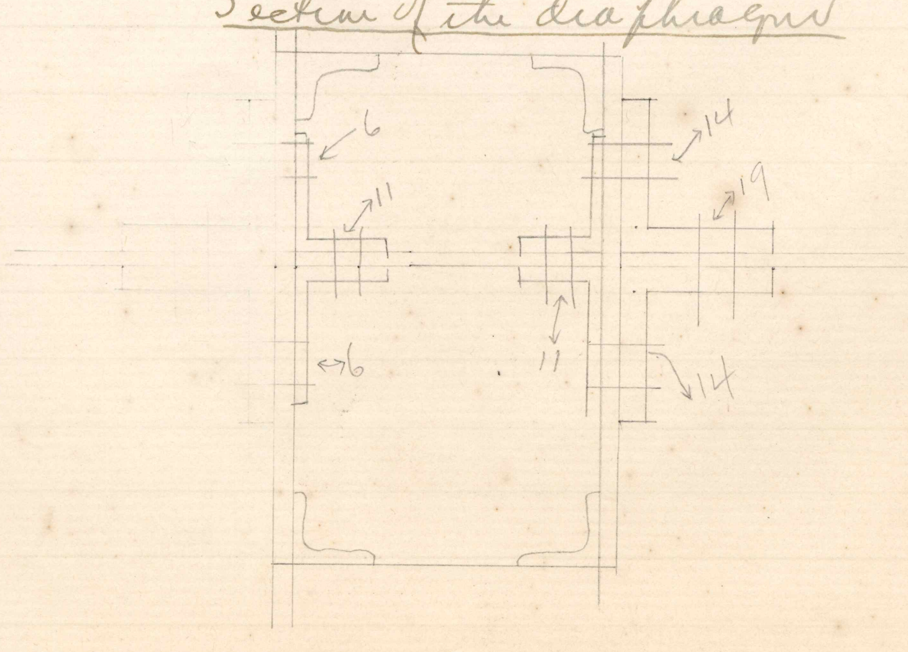
The floor beam will have a length of $(17' - 2 \times \frac{10"}{2}) = 16'5"$ The top cover plate will run the whole length of the floor beam but the bottom cover plate will be cut off 3' $2\frac{1}{2}$" from the end, making its length 13' $2\frac{1}{2}$". The rivets will be arranged as follows:—

First rivet at the middle of the floor beam then a constant pitch of 6" up to the 6th rivet; then a pitch of 2.5" (rivets are staggered as shown in drawing) up to the splice that is up to the 22nd rivet ~~then a~~ pitch The 23rd rivet which is on the other side of the splice is 2" from the end of the gusset plate; then a pitch of 2.5" up to the connecting angle, the 29th rivet Two rivets are put in the angle as is shown in the drawing.

The gusset plate will have a thickness equal to the thickness of the floor beam web $\frac{1}{2}$" Its size is 25" x 63" x $\frac{1}{2}$". The two splice plates, one on either side of the web are place inside the angles. They will consist of two plates 32" x 37" x 3/8" The number of rivets on each side of the splice will be $\frac{160,600}{7880}$ = 21 rivets. The number of rivets through the end connecting angle and the gusset plate is governed by single shear and is equal to $\frac{160600}{6013} = 28$ field rivets.

The Diaphragm. The web of the diaphragm will be 3/8" thick and the angles $3\frac{1}{2}" \times 3\frac{1}{2}" \times 3/8"$. The channels forming the post will be placed at a distance of $9\frac{1}{2}"$ so the web of the diaphragm will be $8\frac{1}{2}"$ long. The number of rivets required to connect the angle to the diaphragm is $\frac{160,600}{2 \times 7880} = 11$ shop rivets. The rivets which connect the diaphragm angle with the outer channel of the post are also shop rivets and the number required is $\frac{160,600}{2 \times 7220} = 12$, 6 on each ~~sd~~ side.
The same rivets which connect the floor beam to the post go through the diaphragm angle on that side of the diaphragm next to the bridge and must therefore be field rivets and take the entire floor beam reaction, the number required is $\frac{160,600}{6013} = 28$, 14 on each side.
A shelf angle $4" \times 4" \times \frac{1}{2}"$, 15" long will be fixed on the floor beam for convenience in erecting.
As in the stringer the rivets in the cover plate of the floor beam are staggered with those in the web.

Section of the diaphragm



Below will be found a table of the pieces composing the floor beam along an estimate of the weight and the number of rivets

Wt of our Floor Beam

2 gusset Plates	313.00 lbs.
Web	999.60 "
4 Connecting plates	592.50 "
4 " "	327.60 "
4 Erecting angles	550.05
8 Connecting angles	400.00
4 Angles	963.73
2 Cover plates	723.8 "
508 Pc Rivets	187.96
Tot.	4558.24 lbs.

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Section of Hip Vertical

The maximum stress in Verticall Bb is 170,000 lbs
The required area is $\frac{170000}{17000} = 10$ sq in.
The depth of the plate will be 9" and the thickness $\frac{3}{8}$".
Then area of plate is $9 \times \frac{3}{8} = 3.375$ sq in
area of 4 angle $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8} =$ 9.96 " "
Gross Area Total 13.33 " "
One rivet hole being taken out of each angle and two out of the web giving a net area of $13.33 - 2.25 = 11.08$ in^2 showing section large enough –
The member will be connected to the upper chord and end post by mean of a pin 7" in diameter.
The total stress 170 000 lbs will be taken by two plate one on each side of the member. The net of the member is 11.08 in^2 and the section through the pin must be 25% in excess of this making the net section through the pin 13.85 in^2 or $\frac{13.85}{2} = 6.93$ in^2 for each plate. The total width of these plates is 14 in and this will give the required thickness $= \frac{6.93}{14-7} = 1$ inch, the connecting plate will have a thickness of $\frac{1}{2}$" and a pin plate $14 \times 14 \times \frac{1}{2}$ will be attached to it to make up the required area. One of the plate will be riveted directly to the member and the other riveted to it as a pin plate. The section back of the pin must be equal to the net section in the body of the member.
The net section is $\frac{11.08}{2} = 5.54$ sq in for one side and the total thickness of the pin plate is 1.125 in making the distance from the end of the member to the pin $\frac{5.54}{1.125} = 5$ inches and the distance to the center of the pin $5 + \frac{7}{2} = 8\frac{1}{2}$" The joint between the plate and main member will be weak in shear, the rivets tending to shear off between the $\frac{3}{8}$" angle and the plate and also between the two plate themselves, as

each side takes one half the above stress the number of rivets required to connect the plate to the main member will be $\frac{170000}{2 \times 7220} = 12$ shop rivets. This plate will have a length of 45 inches.
The number of rivets required to connect inner $\frac{1}{2}$" plate to outer plate which is connected to the member itself is $\frac{170000}{4 \times 7220} = 6$ shop rivets.
At the lower end, this member is connected to the bottom chord by means of a couple of clip angles and 5 rivets. Only sufficient rivets are required to prevent sagging of the lower chord, since the floor beam is connected to the hip vertical above the lower chord and hence no stress comes on the joint on the lower end. A plate $\frac{1}{2}$" thick and long will be riveted to the lower chord to which the vertical will be attached. This will make the vertical have a length of $30' + 8\frac{1}{2}" - 10" - \frac{1}{2}" = 29' 10"$; The $8\frac{1}{2}"$ is the distance from the center of the pin to the top of the vertical and 10" is $\frac{1}{2}$ the depth of lower chord and ½" is the thickness of plate riveted to lower chord.

<u>Vertical Cc</u> Max. stress in this member is 248 730 lbs say 248,800 lbs. The length of this post is 30 ft center to center of pins. $\frac{l}{r}$ must not be more than 100; $\underline{r} = \frac{30 \times 12}{100} = 3.6$, that is $\underline{r}$ must not be less than 3.6. A 12" channel 30 lbs per linear foot has an area of 8.82 sq in and a radius of gyration of 4.28
To find allowable unit stress for compression
$p = \frac{17000}{1 + \frac{l^2}{11000 r^2}}$ was used where r is the least radius of gyration 0.77 + l length of member in inches. The allowable unit stress is therefore 14,160 lbs per in² and the required area of the section is $\frac{248,800}{14160} = 17.57$ in² and the required area of one channel is $\frac{17.57}{2} = 8.78$ in², So two 12"

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30 lbs channels having a sectional area of $2 \times 8.82 = 17.64$ sq in will be sufficient. The channels will be placed 9.5" apart so that the post will be safe about the two axis, the flanges will be turned inward.

Vertical Dd + Ee. The verticals will be similar to the vertical Cc, the same size channels will have to to be used and the channels cannot be placed nearer than 9.5 in apart in order to get the diag. inside the lower chord, if smaller channels are used the channels will have to be placed nearer than $9\frac{1}{2}$" apart in order to give the same strength about the two axis.

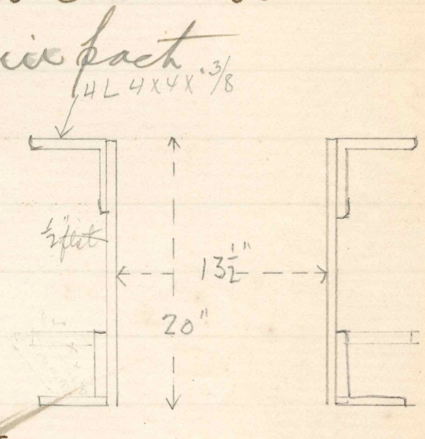
— Stay Plate and Lattice Bars —

Compression members united by lattice bars have plates at the end whose function is to aid in properly dividing the stress between the two segments of the members. According to Cambria p 258, since the depth of the channel is 12" the width of the lattice bars is $2\frac{1}{4}$" and thickness $\frac{3}{8}$" and inclined 30° to cross section of the chord. The latticing which is connected to the posts on the outside is lapped as shown in drawing and must fill the space between the stay plates. The bottom stay plate will have a length of 13" and a thickness of $\frac{3}{8}$". It has 4 - $\frac{3}{4}$" rivets on each side. The top stay plate will have a length of 19" and a thickness of 3/8" and have 6 3/4" rivets on each side. The rivets will be spaced 3" apart in the plates.

The bottom stay plate is to be placed at the end of the diaphragm. The distance from the bottom of the vertical to the center of the pin is 10" and the distance from the top of the ~~xx~~ vertical to the center of top pin hole is 8.5" Distance from center to center of pin hole is 30' and therefore the length of the vertical will be $30' + 10'' + 8\frac{1}{2}'' = 31'6\frac{1}{2}''$.

Lower Chord.

The first two panels of the lower chord will be built up to reduce effect of impact.



The maximum stress in ab = bc is 403,000 lbs Allowable unit stress in tension is 17000 $\frac{lb}{in^2}$ there for required area is $\frac{403000}{17000} = 24\ in^2$.

Gross Area of plates $2(20 \times \frac{1}{2}) = 20\ in^2$

" " " angle $11.44\ in^2$

$31.44\ in^2$

5 rivet holes are assumed to be taken out of each web and one rivet hole out of each angle. Then area to be deducted is

Out of webs $2 \times 5(\frac{7}{8} + \frac{1}{8})\frac{1}{2} = 5.00\ in^2$

" " angle $4 \times (\frac{7}{8} + \frac{1}{2})\frac{3}{8} = 1.50\ in^2$

Total 6.50

Then the net section of the section is $31.44 - 6.5 = 24.94\ in^2$, this is sufficient and will be used.

The horizontal legs of the angles are to be cut off to allow the lower chord to fit inside the end post. The end post is 16" wide inside measurement, all the pin plates on the end post being on the outside and those on lower chord being on the inside. Then the width of the lower chord back to back of angles will be $16 - (2 \times \frac{1}{2} + 2 \times \frac{3}{8}) = 16 - 1.75 = 14.25$ in. say 14 in

The web is taken as 20" deep in order to allow the heads of the diagonal eye bars to fit side the angle, the heads having a diameter of 18".

Lower Chord cd:- Eye bars, 18" head, and a width of 8" will be used. The maximum stress in cd is 665000 lbs. Unit stress is 17000 lbs per in^2 hence the area required is $\frac{665000}{17000} = 36.77$ in^2. 4 eye bars will be used so that the area of each will be $\frac{36.77}{4} = 9.19$ in^2. An eye bar having a width of 8" and a thickness of $1\frac{1}{4}$" gives an area of 10 in^2, this is a little larger than required but will be used so as to allow for the weight of the bar. Section cd will therefore consist of 4 eye bars, $8" \times 1\frac{1}{4}"$.

Lower Chord de:- Eye bars having an 18" head and a width of 8" will be used for this section. The maximum stress in de is 796,000 therefore the area required is $\frac{796000}{17000} = 47.42$ in^2, using 4 eye bars the area of each is $\frac{47.42}{4} = 11.85$ in^2, an eye bar $8" \times 1\frac{1}{2}"$ gives an area of 12 in^2, 4 of this size will be used, giving a total area of 48 in^2.

— Diagonals. —

The stress in Bc is 442,770 lbs.
Using two Eye Bars each will have to hold $\frac{442770}{2} = 221385$ lbs. The allowable unit stress in tension members is 17000 lbs per square inch, therefore the amount of material required in each Eye Bar is $\frac{221385}{17000} = 13+$ sq inches, sectional area. An Eye Bar $8 \times 1\frac{3}{4}$ gives a sectional area of 14 sq inches. The head of the Eye Bar to be 19" in diameter and the head hole for the pin to be 7" in diameter.

The stress Cd is 322140 lbs
As above $\frac{322140}{2} = 161070$ lbs. Then $\frac{161070}{17000} = 9+$ sq required sectional area. Using an Eye Bar $8 \times 1\frac{1}{4} = 10$ sq in. Head to be 19" in diameter and hole to be 6" in diam.

The stress in De is 179200 lbs.
As above $\frac{179200}{2} = 89600$ and $\frac{89600}{17000} = 6$ sq inches, required sectional area. An 8×1 bar with head and hole as the one for Cd will be used.

Each Eye Bar to be:–
$\sqrt{30^2+20^2} = 36'\frac{2}{3}"$ long from center to center of pin holes.

— Counters —

Counters, by the method given in M of R & B Pt I are only necessary in the two middle panels.
Stress in Ef = 45120 lbs, then the area of the cross section required is $\frac{45120}{17000} = 2.66 \text{ in}^2$. An eye bar 4" x 1" gives an area of 4 sq in. This is rather large but will have to be used as no smaller bar can be obtained with a pin hole the required size.

Pin Plate for Lower Chord.

At point a of this member the pin is 7" in diameter and the horizontal legs of the angles of the lower chord are cut so as to allow the lower chord to slip inside the end post. The total bearing area for one side is 15.72 in^2 and the required thickness is $\frac{15.72}{2 \times 7} = 1.12$ inches. Subtracting from this the thickness of the web, $\frac{1}{2}$", gives 0.62 in. A pin plate $\frac{5}{8}$" in thickness will be used.

Net area through the pin must be 15.59 in^2. This net area, allowing for angle legs cut (a bar $4 \times \frac{3}{8}$") is as follows:-

2 angle cut		3 sq in
1 Web	$20" \times \frac{1}{2}" - 7" \times \frac{1}{2}"$	6.5 " "
2 pin plates	$11\frac{3}{4}" \times \frac{5}{8}" - 7" \times \frac{5}{8}$	5.94 " "
	Total	15.44 " "

Total thickness of bearing is $\frac{1}{2} + \frac{5}{8} = 1\frac{1}{8}$". This pin plate will take $\frac{\frac{5}{8}}{1\frac{1}{8}} \times \frac{403000}{2} = 111,944$ lbs. The number of rivets required is $\frac{111,944}{7220} = 20$ shop rivets.

Pin plates for a in End post aB will be the same as for point B on end post.

Pin Plate at b: no plate required here as there is no pin, the lower chord be a built up section and continuous for the first two panels, and the vertical is connected to the lower chord by means of a couple of ~~ship~~ clip angles.

Pin Plate at C. The net section required is 24 in^2 but this must be increased 25% so that the total section required is $24 \times 1\frac{1}{4} = 30$ in^2 or $\frac{30}{2} = 15$ in^2 for one side. The plate which is to increase this section must be on the outside since the intermediate vertical and the eye bars must go inside. The gross width of this plate is $20 - 7 - \frac{1}{4} = 12\frac{3}{4}$ and net width is $12\frac{3}{4} - 6 = 6\frac{3}{4}$ inches. The net area through the pin is

2 angles	$4 \times 4 \times \frac{3}{8} =$	5.72 in^2
1 Web	$20 \times \frac{1}{2} - 5 \times \frac{1}{2} =$	7.50 "
	Total area	13.22 "

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Since this is less than $15 in^2$ a pin plate is needed.
A plate $11\frac{3}{4}" \times \frac{3}{8}" = 4.4$ sq in will make up this deficiency.
Sufficient bearing area must be provided at this joint. The total stress is 403000 lbs the total bearing area required is $\frac{403000}{24000} = 16.7$ sq in. and the total thickness on one side is $\frac{16.7}{2 \times 6} = 1.4$ in. Since the thickness of the web is $\frac{1}{2}"$ and the thickness of the pin plate is $\frac{3}{8}"$ another pin plate $1.4 - (\frac{1}{2} + \frac{3}{8}) = .52"$, a $\frac{1}{2}"$ pin plate will be used. As the total bearing area is now 1.37 in these pin plates will take $\frac{0.87}{1.37} \times \frac{403000}{2} = 127,960$ lbs. The joint is weakest in shear and will therefore require $\frac{127,960}{7220} = 18$ sl of rivets.
The distance from the center of the pin to the end will now be determined. The total section of the body of the member is $31.44 in^2$ or $15.72 in^2$ for one side and the thickness of the web and pin plate is $1\frac{1}{2}"$ The distance of the pin to the end of the member is $\frac{15.72}{1.5} = 10.5$ in and the distance to the centre of the pin is $10.5 + \frac{6}{2} = 13.5$ in, say 14 in.

Top Chord II E.

The head of the eye bar is 18" in diam and allowing a clearance of $\frac{1}{2}$" on either side of the head the total depth of the chord, inside, must not be less than 19", that is the web will be 19" deep. For sections of this character the radius of gyration is approximately equal to $.4h = .4 \times 19 = 7.6$ and the length is equal to one panel or 20'. The allowable unit stress is $p = \frac{17000}{1 + \frac{(20 \times 12)^2}{11000 (7.6)^2}} = 15,600$ lbs/in^2. The required area is $\frac{713,800}{15,600} = 45.76$ in^2, where 713,800 lbs is the largest stress in the upper chord, namely in C II. The correct proportion of section of this character is that 4/10 of the area should be taken up by the web then the area of the web is $45.76 \times .4 = 19.31$ in^2 and the thickness of the web is $\frac{19.31}{2 \times 19} = 0.51$ in say $\frac{1}{2}$" web. Then the webs will be $19" \times \frac{1}{2}"$. Also the correct proportion is that the width between the webs should be 7/8 of the depth of the web, the width between the webs will therefore be $\frac{7}{8} \times 19 = 16.63$", say $16\frac{3}{4}$".

The cover plate plates (Specifications § 148) must not be thinner than $\frac{1}{60}$ the distance between the connecting rivet lines, the lines are in this case $16.75 + (2 \times 2) = 20.75$ in and therefore the thickness of the cover plate cannot be less than $\frac{20.75}{60} = 0.346$ in. The cover plate will be taken as $\frac{1}{2}$" thick.

The width of the cover plate must be $16.75 + (2 \times 3\frac{1}{2}) = 23.75$ in say $24" \times \frac{1}{2}"$.

The center lines of the pins will be taken at the center line of the web and the center of gravity will be assumed $\frac{1}{2}$" above this. In order that the center of gravity may be near that assumed, the moment of the cover plate about the assumed center of gravity should be about equal to the moment of the ~~cover plate~~ plates about the same axis. The moment of the cover plate about the assumed axis is

$24 \times \frac{1}{2}(9.5 - 0.5 + 0.25 + 0.25) = \frac{24 \times 9.5}{2}$ and the moment

of the plats about the same axis
$A(9.5+0.5+0.25+0.25) = 10.50A$, in which A is the area of both plats. Equating these two values and solving for A there results $A = \frac{24 \times 9.5}{2 \times 10.5} = 10.86 \text{ in}^2$.
Assuming the plats to be $4\frac{1}{2}''$ wide the thickness of each is $\frac{10.86}{2 \times 4.5} = 1.2$ say $1\frac{1}{4}''$, as this is too thick to punch 2 plats each $5/8''$ will be used.
The total area is

One cover plate $24 \times \frac{1}{2}$ =		12.00 in^2
Two web plates $2(19 \times \frac{1}{2})$ =		19.00 "
Two plats $(4\frac{1}{2} \times 1\frac{1}{4})2$ =		11.25
	Total ----	42.25 "

The area of a $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ angle is 2.49 in^2 making the total area $(4 \times 2.49) + 42.25 = 52.21 \text{ in}^2$. The required area is 45.76 in^2.
In the determination of the center of gravity of the section, the moment is taken about the top cover plate.
The moment is as follows:-

Cover plate $(24 \times \frac{1}{2})\frac{1}{4}$ =	3.00
Web $2(19 \times \frac{1}{2}) \times (9.5 \times \frac{3}{4})$ =	194.75
Top angle $2(2.49) \times (1.01 + \frac{1}{2})$ =	8.02
Lower " $2(2.49) \times (\frac{1}{2} + \frac{1}{4} + 19 + \frac{1}{4} - 1.01)$	94.57
Plats $2(4\frac{1}{2} \times \frac{11}{8}) \times 20\frac{5}{8}$	232.03
	532.37

The center of gravity is now found to be $\frac{532.37}{52.21} = 10.21$ in from the top of the cover plate. The distance from the top of the cover plate to the middle line of the web is $9.5 + \frac{1}{4} + \frac{1}{2} = 10.25$ and this leaves a distance of $10.25 - 10.20 = 0.05$ in (below) the center line of the web to the neutral ~~line~~ axis. This is called the eccentricity of the section e, e is here equal to 0.05 as this is so small the eccentricity will be considered equal to zero.
The moment of inertia about the axis perpendicular to the

web 3756.53 and the radius of gyration is $r = \sqrt{\frac{3756.53}{52.21}} =$ 8.48. Using this radius of gyration in the compression formula gives an allowable unit stress of 15,800 lbs, and this gives a required area of 45.18 in^2.
The moment of inertia about the axis perpendicular to the cover plate is 3627.46 which gives a radius of gyration equal to 8.35 which shows the section to be a trifle weak about this axis but as the section is greater than the required section it will be used.

Top Chords CD and BC.
The same size angles and web will be used for these sections as in the section DE but the cover plate will have a thickness of 3/8" instead of 1/2" and the flats will consist of strips 4 1/2" x 1/2".
The maximum stress in CD is 698,500 lbs. Assuming 7.60 as the radius of gyration gives an allowable unit stress of 15600 lbs in^2 The required area is thus far 44.65 in^2.
The same sections will be used in BC + CD.
The area is as follows:-

One cover plate	24 x 3/8	= 9.00 sq in
Two web plates	19 x 1/2	= 19.00 " "
Two flats	4 1/2 x 1	= 9.00 " "
Four angles	3 1/2 x 3 1/2 x 3/8 (4 x 2.49) =	= 9.96
	Total	46.96 " "

In the determination of the center of gravity of the section, the moment was taken about the top of the cover-plate The moments are as follows:

Cover plate	1.687
Two webs	195.943
Top Angles	6.152
Bottom Angles	93.202
Flats	185.063

Total --- 482.047

The center of gravity is $\frac{482.047}{46.96} = 10.265$ in from the top of the cover plate. The distance from the top of the cover plate to the middle line of the web is $\frac{19}{2} + \frac{1}{4} + \frac{3}{8} = 10.125$ inches and this leaves a distance of $10.265 - 10.125 = 0.04''$ from the center line of the web to the neutral axis.

Moment of Inertia of the section is as follows:—

Cover plate	844.81
Two Webs	571.58
Top Angles	364.70
Bottom "	364.70
Flats	900.74
	3046.56

The radius of gyration is equal to $\sqrt{\frac{3046.56}{46.96}} = 8.054$. Substituting this value of r in the compression gives an allowable unit stress of 15,725 lbs in², then the required area is 44.37 in², showing the section as designed to be sufficient.

Lattice and Tie Plates for Upper Chord

(Section XVII § 150) The tie plates will be as near as practicable and shall have a length not less than the distance between the lines of rivets connecting them to the flanges. Their thickness shall not be less than $\frac{1}{50}$ of the same distance—

As the cover plate is 24" wide the tie plates will also be 24" wide. The distance between the lines of rivets is 21" so the thickness of the tie plates cannot be less than $\frac{21}{50} = 0.42''$ or $\frac{7}{16}''$. Their length will be 31". There will be 11, 7/8" rivets on each side having a constant pitch of 3". Plates will be placed on each side of each panel point and placed far enough away from the pins to allow room for the diagonals.

The latticeing by which the two sides of the upper chord is to be connected on the under side will be arranged to fill up the space between the tie plates. This latticeing will be $\frac{9}{16}$" thick and 3" wide. Thickness $t = \frac{c}{40}$ where c is the distance from one rivet to another along the lattice bar = 23.1," $\frac{7}{8}$" rivet will be used. The bars will have a length so that the distance from the center of the rivet hole to the end will be $1\frac{9}{16}$". The bars will be lapped as shown in the drawing.

— Pin Plates for Top Chord & End Post —

Pin Plates at B. A pin 7" in diam will be used at B. Stress in BC is 567,200 lbs and the bearing area required is $\frac{567,200}{24000} = 23.6 in^2$ or $12.8 in^2$ for each side. This makes a total thickness of $\frac{12.8}{7} = 1.9$ in. for one side. Since the thickness of the web plate is $\frac{1}{2}$" it will be necessary to provide pin plates whose total thickness must be $1.9 - .5 = 1.4$" Two $\frac{3}{4}$" plates will give a thickness of 1.5" Total thickness of bearing area is now

2 pin plates	1.5 in
1 Web	0.5 "
Total ___	2.0 "

Stress transferred to the two $\frac{3}{4}$" plates is 212,700 lbs. The rivets required to keep the keep the outer plate from shearing off the others are 15 shop rivets, and the rivets required to keep the inner $\frac{3}{4}$" plate from shearing off the web of the section is $\frac{212\,700}{7220} = 29$ rivets in single shear. The bearing of a $\frac{7}{8}$" shop rivet in a $\frac{1}{2}$" web is 10500 lbs, and there for the number of rivets required to keep the plates from tearing the rivets out of the $\frac{1}{2}$" web is $\frac{212,700}{10500} =$ 21 rivets, shop. $\frac{3}{8}$" strips 3" wide as filler to fill the space between the angle and pin plate, length to be equal to that of pin plate.

Pin Plate at C. Horizontal component of the stress in Cd is 178,800 lbs ∴ $\frac{178,800}{24000} = 7.4$ in^2 or $\cancel{3.4}$ 3.7 in^2 are required on each side. A 6" pin is used at C. $\frac{3.7}{6} =$ 0.616" thickness of plate. Since the thickness of the web is $\frac{1}{2}$" the thickness of the plate is $.616 - .5 = .116$" but according to specifications a $\frac{3}{8}$" plate will have to be used. The stress transferred to the $\frac{3}{8}$" plate is
$S = \frac{\frac{3}{8}}{.616} = 0.6$; $\frac{1}{2} \times 178,800 \times 0.6 = 53,640$ lbs. The rivets required to keep the $\frac{3}{8}$" plate from shearing off the web of the chord section is $\frac{53,640}{7220} = 8$ rivets. This plate will be 38" long, as shown in drawing.

Pin Plate at II. Stress in diagonal De is 17,900 lbs horizontal component of this stress is 10,000 lbs. $\frac{10000}{24000} = 0.416$ in^2 or 0.208 in^2 on each side. $\frac{.208}{6} = .036$" = thickness of pin plate. Since the thickness of the web is $\frac{1}{2}$" no pin plate will be needed at this point.

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End Post. This section shall consist of 2 cover plates, and 4 angles, 2 webs

2 Cover plates $25'' \times \frac{1}{2}''$	=	25.00 in^2
2 Webs $19'' \times 5/8''$	=	23.75 "
4 Angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$	=	15.96 "
Total Area	—	64.71 "

Moment of Inertia about axis parallel to cover plate

Cover plates	2500.52
Webs	714.48
Angles	2422.96
Total	5637.96

Radius of gyration $r = \sqrt{\frac{5637.96}{64.71}} = 9.33$

Since the end post is stressed by a combination of bending and compressive stresses, this fact will be considered in the design. In determining the stress in the end post due to its own weight, the entire was not used in computation, but only that component perpendicular to the post. The formula used for computing the stresses due to bending when the member is all subject to compression is $S = -\frac{My_1}{I - \frac{Pl^2}{10E}}$ in which

S = Stress in lbs per in^2 in extreme upper fiber of beam

M = Exterior moment causing the stress = $Wl \sin\phi$

y_1 = distance from neutral axis to extreme upper fiber.

I = Moment of inertia of section = 5638.

P = direct compressive stress in lbs.

l = total length in inches.

E = Modulus of elasticity of steel take as 28,000,000 lbs in^2

$y_1 = \frac{19}{2} + 0.5 = 10;$

$M = \frac{1}{8} Wl \sin\phi$ ($\sin\phi = 0.55484$,

$l = 36' = 432''$

Weight of post, W,

2 cover plates	=	3031.50 lbs
2 Webs	=	2879.64 "

4 angles 1958.40 lbs.
Add 25% for details 1967.38 "

Total Weight 9836.92 ", say
9840 lbs, Stress in members is 610,400 lbs.

$$\therefore S = \frac{\frac{1}{8} \times 9840 \times 36 \times 12 \times 0.55\,484 \times 10}{5638 - \frac{610\,400 \times (36 \times 12)^2}{10 \times 28000000}} = 563.3 \text{ lbs. per in}^2,$$

compression in the upper fiber due to bending and since the neutral axis comes at the middle, the lower fiber will have the same amount of stress except that it will be tension.

An end post is considered fixed when the product of $\frac{1}{2}$ of the total stress times the distance between the web plates is greater than the product of wind load acting at the hip joint B times the length of the end post.

In the first case the value is $\frac{610,400}{2} \times 16 = 4,883,200$

In the second case the value is $11600 \times 36 \times 12 = 5,011,200$.

Since the latter is greater than the former the post is hinged, and the bending moment of the post at the portal strut which joins the end post (is 28.2' from the end) is $9000 \times 28.2 \times 12 = 3,045,600$ lbs inches.

The stress in the extreme fibre due to the moment is

$$S = \frac{3045600 \times 13}{5638 - \frac{610,400 \times (36 \times 12)^2}{10 \times 28,000,000}} = 7568 \text{ lbs per sq in.}$$

The total direct unit stress is $\frac{610,400}{64.71} = 9294$ lbs. per sq in. and this added to 7568 gives 16,867 lbs per in² on extreme fibres only, allowable unit stress is $\frac{17000}{1 + \frac{(36 \times 12)^2}{11000 \times (9.33)^2}} = 13,600$ lbs in², when wind is not taken into account and $1.25 \times 13,600 = 17000$ lbs when wind is taken into account. The difference between this and the actual stress is $16862 - 17000 = -138$ which shows that the section is strong enough —

<u>Pin Plates for End Post.</u> The pins used at each end of the end post are 7" in diameter. The total

stress is 610,400 lbs which requires a bearing area of $\frac{610,400}{24000} = 25.44$ in^2 for both sides or 12.72 in^2 for one side and a total thickness of $\frac{12.72}{7} = 1.815$ in for one side. Since the thickness of the web is $\frac{1}{2}''$ a remainder of $1.815 - .5 = 1.315''$ is left to be provided for by pin plates. The first plate will be $5/8''$ thick to fit with the angles, the other plate will be $\frac{11}{16}''$ thick giving a total thickness of 1.313 in.
The proportion of the stress which is taken by the $5/8''$ plate is $5/8 \times \frac{1}{1.815} \times \frac{610400}{2} = 10500$ lbs, and that taken by the $\frac{11}{16}''$ plate is $\frac{11}{16} \times \frac{1}{1.815} \times \frac{610,400}{2} = 12,421$ lbs
The number of rivets required to transfer the stress from the $5/8''$ plate to the $\frac{11}{16}''$ plate is $\frac{10,500}{7220} = 15$ rivets
The number of rivets required to transfer the stress from both pin plates to the web is $\frac{10500+12421}{7220} = 30$.

Bearing for End of Bridge.

The end reaction of the bridge proper is equal to the vertical component of the stress in the end post here it is $\frac{30}{36} \times 610,400 = 510,400$ lbs which requires a bearing area on the masonry of $\frac{510,400}{600} = 850 \text{ in}^2$.

The plate will be 30 in long, the total bearing area for one on the vertical plates is ~~10.64~~ $\frac{510,400}{2 \times 24000} = 10.64 \text{ in}^2$. and the total required thickness is $\frac{10.64}{7} = 1.55$ inches, a 7" pin being used at a. Since the vertical plate will be made $\frac{3}{4}$" thick $1.55 - 0.75 = 0.80 = \frac{13}{16}$" to be made up of pin plate. The amount of stress which is carried by the $\frac{13}{16}$" plate is $\frac{0.81}{1.55} \times \frac{510,400}{2} = 133,370$ lbs. These plates will tend to shear off the rivets at a place between the plates and therefore $\frac{133,370}{7220} = 19$ $\frac{7}{8}$" rivets will be required to fasten them to the vertical plate.

Since the length of the masonry plate is 30" and the total required area is 850 in^2, the required width is $\frac{850}{30} = 29$"

The actual width will be greater than this since it must be sufficient to allow for the connecting angle and also for the bearing of the end floor beam.

The connecting angle will be $\frac{3}{4}$" thick, $6 \times 6 \times \frac{3}{4}$, the plate to which they will be connected $\frac{3}{4}$" thick. The bottom plate will extend 3" outward in order to allow sufficient room for the anchor bolts, which will be $\frac{7}{8}$ in diameter and will extend into the masonry [illegible] inches.

In addition to the reaction of the bridge proper the masonry plate must be of sufficient area to give bearing for end reactions of the end floor beam.

The maximum reaction of the end floor beam is 104,780 lbs, the bearing area required on the masonry is $\frac{104,780}{600} = 175 \text{ in}^2$ and assuming that the base of the bearing will be 12" long the required width will be $\frac{175}{12} = 16$", the bearing is extended the whole length

of the masonry plate which is 30" long in this case — The rivets which go through the horizontal leg of the angle and through the cap angles and cap plate do not take stress but the rivets in the horizontal leg of the angle will be staggered with those in the vertical leg. The cap plate tends to keep the vertical plate in line and to keep out the dust and dirt and other deteriorating influences of the elements.
The rivets through the vertical leg of the shoe are in double bearing in the $\frac{3}{4}$" angle, in single bearing in the vertical plate and in double shear. The value of a $\frac{7}{8}$" rivet in double shear is 14,440 lbs and therefore the number of shop rivets required through the vertical legs of the angles is $\frac{510,400}{2 \times 14440} = 18$.
The space for the anchor bolt, that for the connecting angle, that for the bearing of the end floor beam $= 2 \times \frac{3}{4} + 15\frac{1}{2} + 2 \times 6 + \frac{1}{2} + 3 + 12 = 3'8\frac{1}{2}"$ for the fixed end.
The design of the roller end requires that the length of the masonry bearing, the size of the vertical plates and angles and also the number of rivets shall be the same as that for the fixed end.
The width of the masonry plate is determined by the length of the rollers and their connections at the end.
The rollers are required to be 6" in diam and the unit stress per linear inch is $6 \times 600 = 3600$ lbs which required $\frac{510,400}{3600} = 141.8$ linear inches of rollers. This is for the reaction alone of the bridge, and in addition to this there are required for the floor beam reaction $\frac{104,780}{3600} = 29.12$ linear inches, then the total number of linear inches required is $141.8 + 29.12 = 170.92$ inches and if 5 rollers are used they must be $\frac{170.92}{5} = 34.2$ inches long each —

The distance from the center lines of pins to top of masonry can now be determined and is
$10 + \frac{1}{4} + 6 + \frac{3}{4} + 6 + \frac{3}{4} = 23\frac{3}{4}$".
On account of putting in sufficient ~~bearing~~ connections and angles, the masonry must be considerably wider than that theoretically determined.
From center line truss to outer edge;
$\frac{15.5}{2} + \frac{3}{4} + 6 + \frac{1}{2} + 2\frac{5}{8} + 3\frac{1}{8} + 3 = 23\frac{3}{4}$" say 2 ft.
From center line of truss to inner edge
$\frac{15.5}{2} + \frac{3}{4} + 6 + 1 + 12 + \frac{1}{2} + 2\frac{5}{8} + 3\frac{1}{8} + 3 = 3'\frac{3}{4}$" say 3'1"
Total -- 5'1"
Allowing guide-plates $3" \times \frac{1}{2}"$ and guide bars $2\frac{1}{2}" \times \frac{1}{2}"$ and assuming $\frac{1}{8}$" clearance at the end the total length of the rollers are
$5'1" - 2(3 + 3\frac{1}{2} + \frac{1}{8} + \frac{1}{2}) = 46.5$" This shows them to be amply long enough, as 34.2" is theoretically required.
The guide plates are small bars riveted to the top of the bottom plates and serve to keep the rollers in line. The guide bars are connected to rollers at their ends and serve to keep the rollers equidistant, thereby causing them to roll easier and keeping them from becoming worn by contact with each other -
Expansion must be allowed for at the rate of $\frac{1}{8}$" for every 10' of span This makes a total allowance for expansion due to change of temperature $\frac{160}{10} \times \frac{1}{8} = 2$". No slotted holes are to be provided for the anchor bolts holes since they do not go through that part of the bridge which slides The shoe slides over the rollers and is kept in place by the angles at the end, which are riveted to the masonry plate.
Unless sufficient room is allowed between the segmental rollers which are to be used, they will tend to bind when the bridge has reached the extreme

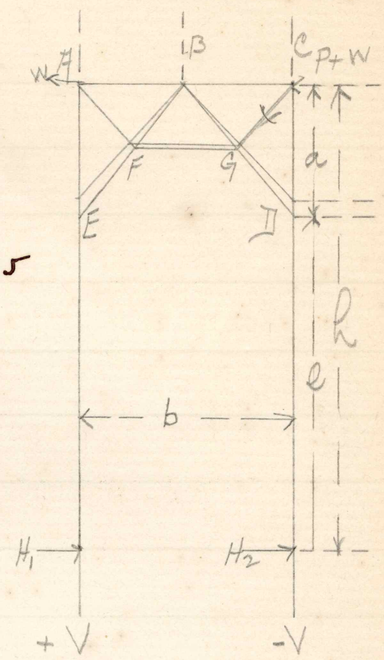
position for expansion or contraction. This distance can be computed from proportion and from the following formula $y = \frac{1}{\cos\phi}$, $\phi = \frac{e}{4} \times \frac{360}{3.1416 D}$ in which e is the amount allowed for the change in temperature, and D is the diameter of the rollers, both taken in inches. The angle ϕ as computed from the above formula is $9°30'$. Substituting in the equation giving the value for $y = 1.02''$ say $1\frac{1}{4}''$ for the distance between rollers. The rollers will be $4\frac{1}{2}''$ wide since there are 5 rollers there are 4 spaces between them. Also since the rollers must occupy a space of 30".

The width of the guide bars must be such as to allow freedom of motion for rollers. The maximum width allowable is given by the formula

$w = \frac{D}{2} \cos\phi$, in which ϕ and D are indicated above.

This requires the bar to be, $w = \frac{6}{2} \times 0.985 = 2.96''$ say $2\frac{1}{2}''$, wide.

-Portal Bracing-



W = wind panel load in Upper Chord = 3000 lbs.

n' = number of panel in Upper Chord = 6

$P = \{(n'-1)\} W = (6-1) 3000 = 15000$

$V \pm = \{(P+W)+W\} \frac{h_1}{b} = \pm \{(15000+3000+3000)\} \frac{36}{17} = \pm 44,475$

$H_1 = H_2 = \{(P+W)+W\} \div 2 = \frac{21000}{2} = 10,5000$

The stress in BC, center of moment at D is

$S_{BC} = -\frac{(P+W)a + H_2 l}{a} = -45,3000$

The Stress in AB, center of moment at E

$S_{AB} = + \frac{Wa + H_1 l}{a} = +30,3000$

Stress in BD, center of moments is at C and the perpendicular distance x is 6.477 ft.

$S_{BD} = + H_2 \frac{h_1}{x} = +58,361$

$S_{BE} = - H_1 \frac{h_1}{x} = -58,361$

BD + BE take the largest stress, these will be designed and the others similar to it except AF, FG + CG which will consist of single angles $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}$ as they take no stress - length of BE = 13.13 ft. Ratio of the length to the radius of gyration should not exceed 120, this means that the radius of gyration $r = \frac{13.13 \times 12}{120} = 1.3$. This section will consist of two angles, $4 \times 3\frac{1}{2} \times \frac{3}{8}$ placed back to back. The radius of gyration of the two angles = 2.12. Allowable unit stress $p = 1\frac{1}{4}\left[\frac{17000}{1+\frac{(13.13\times12)^2}{11000(2.12)^2}}\right] = 14,130$ lbs per sq in

The required area is hence $\frac{58,361}{14,130} = 4.14 \text{ in}^2$, the area of the two angles chosen above is 5.36 in^2, therefore they will be used.

The above angles will also be tested for tension, it be considered that one rivet hole is taken out of the section of each angle. The net section of the two angle is now $5.36 - 2\left(\frac{7}{8}+\frac{1}{8}\right)\frac{3}{8} = 4.61 \text{ in}^2$ and the area required in tension is $\frac{58360}{1.25 \times 17000} = 2.75 \text{ in}^2$, therefore the section is amply sufficient.

In this case the stress for which the connections will be designed is $2 \times 58,360 = 116,720$ lbs. The unit stress in

this case is increased 25 %, over that allowed for dead and live load.
The number of rivets required in the end connection will be governed by bearing in the connection plates and these plates will be $\frac{3}{8}$" thick. The number of rivets required is
$\frac{116,720}{7880 \times 1.25} = 12$ shop rivets or $\frac{116720}{1.25 \times 6550} = 15$ field rivets.
Same size angles will be used in the top part of the portal bracing.
The number of rivets is determined by bearing in the $\frac{3}{8}$" connection plates and is $-\frac{2 \times 45300}{7880 \times 1.25} = 10$ shop rivets and $\frac{2 \times 45300}{6550 \times 1.25} = 12$ field rivets

Floor Beam – [End]

Weight of one stringer with half of lateral system	4000 lbs
" " 1/2 of 17 ties each 10 ft long	2040 "
" " 2 rails and fastenings	2000 "
" " 1 outside guard rail	420 "
" due to live load	62500
" " " Impact	59125
Total	130,085 "

Say 130,100 lbs which is the load placed 3'3" on each side of the middle of the floor beam. Bending moment due to this load is $130,100 \times 5\frac{1}{4} = 683,025$ lb ft. Assume the weight of the floor beam as 3200 lbs then the moment due to its weight at the middle is $\frac{1}{8}Wl = \frac{1}{8} \times 3200 \times 17 = 6800$ lb ft. The total bending moment at the middle is 689,825 lb ft or 8,277,900 lb inches.

The web is to have an area of $\frac{131,685}{10000} = 13.17$ in^2. The effective depth will be assumed as 48", the required thickness must therefore be $\frac{13.17}{48} = 0.28"$, the web will be $48" \times \frac{3}{8}"$.

The depth of the end floor beam is somewhat greater than the depth of the intermediate floor beams, due to the fact that it will extend downward a greater distance, resting upon the bearings which come directly upon the top of the rollers. The exact depth can be found since the roller bearings are already designed and is $48 + 8 = 56$ inches.

The effective depth (approx) is $56 - 2 = 54$ inches. The flange area required is $\frac{8277900}{17000 \times 54} = 9.02$ in^2.

Two angles $6 \times 6 \times \frac{7}{16}$ give an area of $2 \times 5.06 = 10.12$ in^2 and a net area of $2\left[5.06 - \left(\frac{7}{8} + \frac{1}{8}\right)\frac{7}{16}\right] = 9.24$ in^2, the area is amply sufficient, and no cover plate need be used.

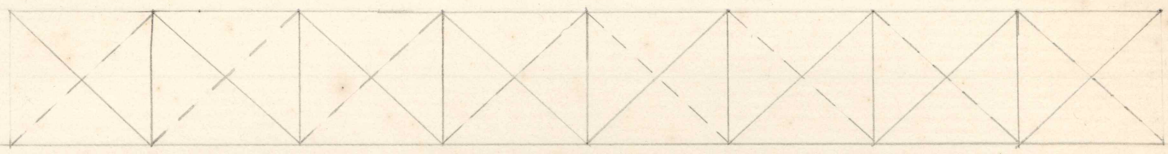
The pitch of the rivets in the flange was determined by the formula $S = \frac{vh_e}{V}$. The shear being practically

constant from connection of the stringer to the end of the floor beam, the pitch of the rivets will be made constant and in this case will be
$S = \frac{7880 \times 54}{131,300} = 3.24''$, the rivets will be spaced 3" apart, this pitch will be used a little distance beyond the place where the stringer is connected to the floor beam, then a pitch of 6".
The gusset plate will have a thickness of equal to the thickness of the floor beam web, $\frac{3}{8}''$, Its size is $25'' \times 71'' \times \frac{3}{8}''$.
The two splice plates, one on either side of the web are placed inside the angles. They will be $40'' \times 37'' \times \frac{3}{8}''$. The number of rivets on each side of the splice plate will be
$\frac{131,700}{7880} = 17$ shop rivets.
The number of rivets through the end connection angle and the gusset plate is governed by single shear and is equal to $\frac{131,700}{6013} = 22$ field rivets
There will be no need of a diaphragm here —

-Wind Stresses-

The effect of the wind tending to overturn the truss was considered for all the truss members. The stresses, due to this, are shown in the table.

Lower Lateral System



The fixed wind load was taken as 150 lbs per linear foot of chord. The panel load is hence 20×150 = 3 Kips, which is considered on one chord. The stress are as follows:

d_1 = 16170 lbs d_3 = 6925 lbs

d_2 = 11550 " d_4 = 2310 lbs.

The wind load acting on the train was taken as 450 lbs per linear foot, or 9 Kips per panel, which was considered as a moving load. The stresses due to this load are as follows.

d_1 = 14560 lbs d_3 = 6925 lbs

d_2 10400 " d_4 = 4170 "

The total stresses in the diagonals of the lateral system due to these two loads are as follows:

d_1 = 30,730 lbs d_3 = 13850 lbs

d_2 = 21,950 lbs d_4 = 6480 "

Single shear governs the number of rivets which are as follows

1st panel $\frac{30730\times2}{6013\times1.25}$ = 9 field rivets

2nd " $\frac{21950\times2}{6013\times1.25}$ = 6 " "

3rd " $\frac{13850\times2}{6013\times1.25}$ = 4 " "

4th " $\frac{6480\times2}{6013\times1.25}$ = 2 " "

The length of a diagonal is $\frac{1}{2}\sqrt{17^2+20^2}$ = 13.13 ft, $\therefore$ $\frac{1}{2}$ because the diagonals are joined at the middle to a plate.

Radius of gyration must be larger than $\frac{13.13\times12}{120}$ = 1.32

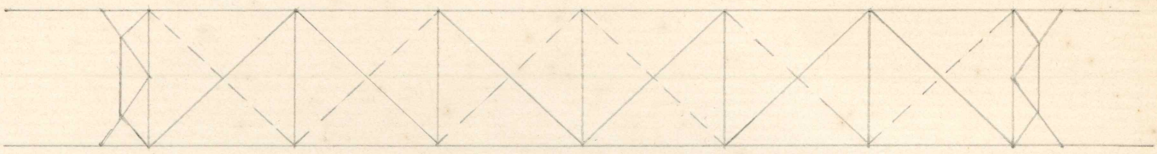
An angle $4\frac{1}{2}\times3\times\frac{3}{8}$ has a radius of gyration = 1.44

The pieces are designed to take only tension so the unit stress is $1\frac{1}{4} \times 17000 = 21,250$ lbs per in.^{2}, 25% excess when wind load is considered.

The required area is then $= \frac{30,780}{21250} = 1.45$ sq. in.

An angle $4\frac{1}{2} \times 3 \times \frac{3}{8}$ has an area of 2.68 in.^{2} and a net area of $2.68 - (\frac{7}{8} + \frac{1}{8}) \times \frac{3}{8} = 2.3$ in.^{2}, allowance made for one rivet. The required area is much less than the area designed but must be used in order to get the correct radius of gyration. All members of the lower lateral system will be the same size.

Upper Lateral System



The wind was considered as 150 lbs per linear foot of chord the end post being considered part of the top chord.
The diagonals that are broken take no stress, they are meant only to take tension. The stresses are as follows,
$d_1 = 18480$ lbs, $d_2 = 12320$ lbs, $d_3 = 4000$ lbs.
The length is the same as in the top lateral system 13.13 ft. Radius of gyration must be greater than $\frac{13.13 \times 12}{120} =$ 1.32 Angle $4\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$ has a radius of gyration of 1.44 Angles must be used for all members on account of §130, Specifications. Allowable unit stress is $1\frac{1}{4} \times 17000 =$ 21250 lbs per in^2 The required area is $\frac{18,480}{21250} = 0.88$ in^2
Area of angle $4\frac{1}{2} \times 3 \times \frac{3}{8} = 2.68$ in^2, net area deducting one rivet hole is $2.68 - (\frac{7}{8} + \frac{1}{8})\frac{3}{8} = 2.3$ in^2. The required area is much less than the area of angle chosen but must be used.
Single shear governs the number of rivets required, the number is as follows.

1st Panel $\frac{18500 \times 2}{6013 \times 1\frac{1}{4}} = 8$ field rivets.
2nd " $\frac{12,400 \times 2}{6013 \times 1\frac{1}{4}} = 5$ " ".
3rd " $\frac{4000 \times 2}{6013 \times 1\frac{1}{4}} = 4$ " ".

Respectfully Submitted,
Bolling W. Carter.