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

Span 150 Feet
6 Panels 25 Feet

(Respectfully submitted to Prof. D. C. Humphreys as a thesis for a B.S. degree by
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Introduction

Design of a pin-connected, through Pratt railway truss of 150 feet span, 6 panels. The trusses are spaced 17 feet center to center.

This design is made for all main members of truss but does not go into the detailing of upper chord splice, end bearing, and portal and sway bracing.

A blue print has been made of all principal connections. Pin-plates are shown as they are desinged and do not appear upon print except where they are exposed. The middle vertical is shown in connection with the floor beam.

The dimensions of all pin plates, stringer and floor beam are given for all details. For the other members the cross section is given and the lengths can be scaled off of print.

In the design the specifications were taken from Merriman and Jacoby's "Roofs and Bridges" Part III. Art. 83. This book is followed in the main through the whole design.

References: The above, Cyclopedia of Civil Engineering, Vol. VI of American School of Correspondence, "Roofs and Bridges" Part I, Cambria Steel Handbook.

Floor Timbers

The greatest stress in the cross-ties is produced by the alternative loading specified. The weight on one axle is 50000 lbs. The impact will also be taken as 50000 lbs. According to specifications cross-ties must be spaced 6 inches, be 8 inches wide. The three ties and spaces will cover a length of 3½ ft. Assuming total weight of track 450 lbs. per foot, the weight for 3½ ft. is 1575 lbs. The total load on three ties is 101575 lbs. and for each rail on one tie 17000 lbs. The dead load for ties is so small that it may be considered as concentrated at the tracks. The stringers are spaced 6'-6" and the rails 4'-11½", the rails are placed symmetrical to the stringers. The tie is considered as a beam under two concentrated loads. The distance from rail to to center of stringer is 18.5 inches. The bending moment is 157200 lb.in. From "Mechanics of Materials" $M = \frac{SI}{c}$ and $\frac{I}{c} = \frac{bd^2}{6}$. Since $S = 2000$ lb/in² and

$$b = 8 \text{ in.} \quad d^2 = \frac{M \times 6}{bS} = \frac{157200 \times 6}{16000} = 7.67 \text{ in.}$$

A depth of 8 inches will accordingly be taken for ties.

Cross-ties will be notched over projection of web of stringer above flanges.

The cross-ties will be alternately 10 and 14 feet in length to allow for a foot-walk.

The dimensions of the guard rail will be taken from specifications.

Track Stringers.

The length of span is 25 feet from center to center of floor beams.

The computed weight of track per foot is about 440 lbs. The weight of track carried by one stringer = 220 lbs × 25 ft = 5500 lbs. The weight of stringer and one half of lateral bracing is assumed = 5100 lbs.

The dead load shear = 5300 lbs.

To find maximum live load shear and moment the stringer is divided into 2.5 ft. sections and treated as beam under concentrated loads. Max. live load shear occurs when wheel (2) is at 0 section = 70600 lbs. The coefficient of impact is 0.923 and allowance for impact = 65150 lbs. Total shear = 141050 lbs.

Dead load moment = $\frac{Wl}{8}$ = 33100 lb ft.

Max. live load moment occurs when wheel (3) is 1.25 ft. to left of center of stringer. It is 381500 lb. ft. Allowance for impact = 352000 lb. ft.

The total bending moment then = 766600 lb ft.

Web. The specified unit shearing stress 10,000 lbs./in². Then the required net section of the web = 14.11 in². A thickness of ½" will be taken for the web as this thickness does not require stiffeners. A deduction will be made from the gross area of the web for 7 rivets holes as it has been found that this number will used in inner line of rivets connecting stringer to floor-beam. A depth of 41" will be taken for stringer as it must be deep enough to give cross-ties a clearance above floor-beam. This will make net area $= (\frac{1}{2}'' \times 41'') - (\frac{1}{2}'' \times 7 \text{ rivets}) = 17 \text{ in}^2$ considerably in excess of that required but this will be used.

Flanges. Flange angles 6 in. wide are most suitable as no cover plates will be used. The web will project ½" above top flanges in order that the cross-ties need only be notched over the web-plate. The flange will be designed for the specified tension stress of 15000 lbs/in² and the same section will be used for the compression flange.

6 in. angles will tried and allowing the back of lower flange angles to project ¼" below bottom of web to make effective depth large enough the clear distance between ∠s will be 28.75 in. Assuming an effective depth between centers of gravity of angles = 38.25 in. which has been found to be correct. The required flange area then is found from formula $\frac{M \text{ lb in}}{h' \times \text{unit stress}}$ where h' = effective depth.

$$\frac{M \text{ lb in}}{h' \times 15000} = \frac{766600 \times 12}{38.25 \times 15000} = 16 \text{ in}^2 \text{ required area for } M.$$

The web plate has its resisting moment reduced to 88.4% of gross section by rivet holes in web, connecting flanges. One sixth of this or 14.7% of the gross web section takes bending moment in addition to flanges. The area of web which takes bending moment then is $0.147 \times 41.25 \text{ in} \times \frac{1}{2}'' = 3.03 \text{ in}^2$. Deducting this from required total area the area required

in flanges is $16 in^2 - 3.03 in^2 = 12.97 in^2$.
$6'' \times 6'' \times \frac{5}{8}''$ Ls will be tried. Deducting one rivet hole in each angle the net area is $2(7.11^{\square in} - 0.625 \square in) = 12.97 in^2$. This area could have been a little larger but since the net area equals exactly the required area these angles will be used for flange.

Rivet Pitch in Flanges

The maximum differences of flange stress per linear unit is found from the expression $\frac{V}{h'}$ where the maximum shear at the section is substituded for V. The pitch of the rivets at the end of the stringer will be the uniform pitch throughout. The max. shear at end of stringer = 141000 lbs. The max. difference of flange stress must, however, be multiplied by the ratio of the area of flange net section to the area of the total section since part of the bending moment is taken by the web. The increment of flange stress per linear inch at end is then $= \frac{12.97 in^2}{16.00 in^2} \times \frac{141000 lb}{38.25 in} = 2990 lb.in.$

This is the horizontal component of the stress on rivets. The vertical stress is the load on three ties per inch. This is 25000 lbs for one driver, 17750 for impact, and 700 for track and dead load. Total = 1040 lb.in. The resultant of the vertical and horizontal components of the stress governs the pitch.

Resultant $= \sqrt{(2990)^2 + (1040)^2} = 3136$ lb.

Since bearing value is less than the double shear this will determine the pitch. The bearing value for 7/8" rivet in 1/2" plate = 9630 lb. The pitch of the rivets is now found by dividing the bearing value of one rivet by the stress per linear inch = 9630 lb. ÷ 3136 lb./in = 3.08 in. Therefore a uniform pitch of 3" will be used throughout for flanges. The rivets will be staggered.

Connecting Angles and Rivets

The angles connecting stringer to floor-beam will be straight and fillers whose thickness equals that of the flange angles will be used. They will be wide enough to receive an extra row of rivets beyond connecting angles. Since the rivets connecting filler to web of stringer are in double shear the bearing value will govern their number. This is 141000 ÷ 9630 = 15.

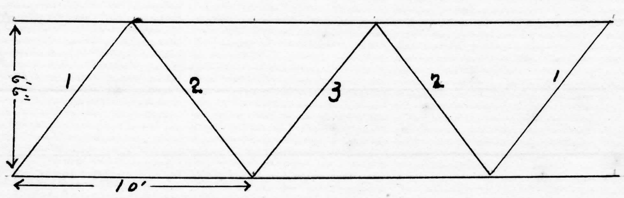
In finding number of rivets which must pass through web plate and angles the value of the rivets in double shear will govern since this is less than their bearing in the two ∠s combined or the two filler plates combined. The number required must not be less than 141000 ÷ 6610×2 = 11. Those passing through flange ∠s are not counted in number required in filler plates or in connecting ∠s.

The rivets connecting other legs of ∠s & web of floor beam are field rivets and since there are two rows in single shear the number in each ∠ must be one fourth more than 11 or 14. These rivets will be staggered and since the number in the other leg including two in the flanges is 15, this number will used to connect angle to floor beam.

Since rivets are staggered 6" x 6" ∠s will be used. Their thickness cannot be determined theoretically but practice shows that 1/2" will be sufficient. 6" x 6" x 1/2 ∠s will be used.
The pitch of the rivets will be 2 5/8 in, the outer row containing eight.
Filler will have 4 rivets outside of ∠. The fillers will be 9" x 5/8 plates.

<u>Lateral</u> <u>Bracing</u> <u>of</u> <u>Stringer</u>.

From specifications unsupported length of girder must not exceed 12 times flange width. The arrangement shown in diagram fulfills these conditions and is symmetrical.



The bracing will be designed for the max. tension and compression stress and all members will be taken this size.
Taking 150 lbs. per linear foot for static wind load and 300 lbs. for moving wind load the stress in member 1 is found to be ±4970 lbs. From specifications length must not be over 120 times radius of gyration. Therefore $r = \frac{90 \text{ in.}}{120} = 0.75$
Unit stress when member is subjected to alternate tension and compression = 17000 lbs/in
From compression formula $p = \frac{S}{1+\frac{1}{11000}\frac{l^2}{r^2}}$

the unit stress for which member must be designed = 6900 lbs.-in.² 4"×4"×5/16" ∠s will be used. This angle gives considerable excess area but 5/16" is minimum thickness of angles which will give required radius of gyration.

The end connecting plate with corner clipped will be 8"×3/8"×11" and others 8"×3/8"×20".

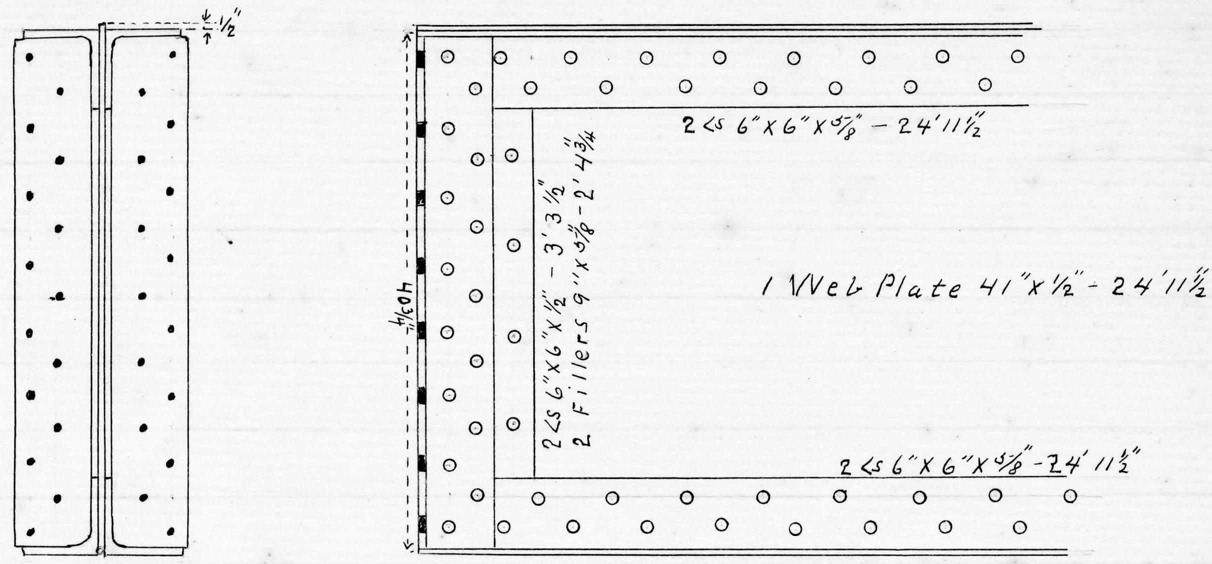
Five rivets will be used to connect plate to flange of stringer 2 of which go through braces also. Two rivets will be used beside these to connect each brace to plate alone.

Estimate of Stringer Weight.

4 flange angles 6"×6"×5/8"×24'×11½" @ 24.2 lb.	2416 lbs.
1 web plate 41"×½"×24'·11½" @ 71.4 lbs.	1740 lbs.
4 connecting ∠s 6"×6"×½"×3'3½" @ 19.6 lbs.	250
4 fillers 9"×5/8"×2' 4¾" @ 19.13 lbs	180
Half lateral system:	
2½ lateral ∠s 4"×4"×5/16"×7'4" @ 8.2 lbs	160
2 connecting plates 8"×3/8"×20" @ 10.2 lbs	34
1 connecting plate 8"×3/8"×11" (Corner clipped)	8
230 pairs rivet heads at 0.37 lbs.	85
Total	4875

Since assumed weight of girder was 5100 lbs the error is one the side of safety and also allows for lateral bracing of truss to be connected to lower flange of stringer.

The diagram on next page shows dimensions of stringers and pitch of rivets.



End Connection of Stringer

Intermediate Floor Beams.

The length of the floor beam used in the design is the distance from center to center of trusses = 17 ft.

In designing the floor beam is taken as a beam under the action of two concentrated loads each 3' 3" from its center. The weight of the floor beam is assumed 4400 lbs. Considering one half of the beam the concentrated load is composed of a dead load due to the weight of one stringer and a max. live load found from the max. floor beam reaction by the formula -

$R_f = \frac{(M_c - 2M_b)}{P}$. Taking weight of one stringer as 10600 lbs. and half of floorbeam = 2200 lbs.

Then total shear at end of beam is -

due to dead load of stringer = 10600 lbs

" " " " " beam = 2200 lbs.

" " live load = 95000 lbs.

" " impact = 95000 lbs × 0.89 = 84600 lbs.

Total end shear = 192400 lbs.

The bending moment at center of beam is from concentrated load = 190200 × 5.25 ft = 1010000 lbs-ft. From dead load of beam = $\frac{1}{8}Wl$ = 9350 lbs. ft. Then total moment at center = 1019350 lbs. ft.

__Web.__ Assuming a depth of 5'-3" for the web as this will give several inches clearance for cross-ties with the stringer resting upon a supporting angle riveted to the flange of floor-beam and having a vertical leg of 5" or 6". With a unit shearing stress of 10000 lbs/in² and a max. end shear of

192400 lbs the required net section of the web is 19.24 in². The depth of 53" would require a thickness of 3/8" for the web but a thickness of 1/2" will be used as this allows for a deduction of 14 rivet holes at splice and this will probable be enough.

Cover Plate and Flanges. Since a cover plate will be used to take a part of the bending moment an effective depth of 52" will be taken. Unit stress for flange = 15000 lb/in². Calculating required area to take moment from formula used in design of stringer, $\frac{M \text{ lb-ins}}{h' \times \text{unit-stress}}$ it is found ~~=~~ (1019350 X 12) ÷ (53" X 15000 lb/in²) = 15.68 in². Deducting one eight of gross area of web to take part of the moment the area to be divided between flange angles and cover plate = 15.68 in² - 3.32 in² = 12.86 in². This must be divided as nearly equally as possible between cover plate and flanges. The section is designed for the tension and the compression section made equal to it. Deducting 1.13 in² for one rivet hole in cover plate and one in ∠s,

2 angles 5" X 4" X 9/16 give 2(4.75 in² - 1.13 in²) = 7.24 in²

1 cover plate 11" X 9/16 " 6.19 in² - 1.13 in² = 5.06 in²

Total area = 12.30 in²

The backs of the angles will be placed 1/8" beyond edge of web. The effective depth now, considering the flanges as solid, is taken between the centers of gravity of the flanges being 0.55" below backs of upper ∠s and 0.60" above backs of lower ∠s. The effective depth = 53" + 0.25" - 0.55" - 0.60" = 53.1".

The required area for this depth is 12.33 in². This is nearly the same as the net area of the flange ∠s and cover plate taken above and these will be used. Clear distance on web between legs of ∠s = 45.25".

Rivet Pitch in Flange. The rivet pitch for flanges of floor beam will be found in same way as in stringer from V/h' multiplied by the ratio of flange section to required total section. The stress is then $= \frac{12.3 \text{ in}^2}{15.65 \text{ in}^2} \times \frac{192400 \text{ lbs}}{52.1 \text{ in}}$ = 2900 lbs. per in. The bearing of one rivet in the ½" web = 9630 lbs. The pitch then is 9630 lbs ÷ 2900 lbs per in = 3.32 in. The pitch in flanges will accordingly be taken 3¼" outside of stringer and 6" between stringers since here the shear is practically nothing and 6" is the max. pitch allowed by specifications. This will allow rivets to placed in single row. The same pitch is used in cover plate and angles.

Floor Beam Splice. The floor beam will have to be spliced at each end to allow bottom of beam to be on same level as bottom of verticals and yet have sufficient rivet connection to verticals and give room for pin and the members connected to it.

Since splice plate must be extended to end of the beam in order to act as filler under end connecting angles it will taken same thickness as flange angles 9/16 thick and examined to see if it gives sufficient bearing to the number of rivets required in splice.

Two rows of rivets will be placed on each side of the splice and the number in the outer row must be less than 14 to give sufficient net section to web. The neutral axis of beam is 26.625" from each side.

As the web is ½" thick and 53" deep, and the unit stress in the outer fiber is 15000 lbs in², the resisting moment of ½ gross section of web since only ⅙ of web can take moment $= \frac{1}{2}(\frac{1}{6} \times 15000\text{ lbs in}^2 \times \frac{1}{2}'' \times 53'' \times 53'') = 1,755,000\text{ lbs ins.}$

To find resisting moment of net section of web it is necessary to deduct the diametrical sections of the rivets holes in the outer row of splice. The reduction of the tensile stress of the web for rivet hole at outer fiber is $15000\text{ lbs in}^2 \times 1 \times \frac{1}{2}''$ $= 7500$ lbs., while for one at distance y from neutral axis it is $\frac{7500 y}{26.625\text{ in}}$. The moment of this stress about the neutral axis is $\frac{7500 y^2}{26.625\text{ in}}$ and the sum of the moments for all the rivets in a row is $\frac{7500 \Sigma y^2}{26.625}$

Let rivet spacing be 3" from neutral axis for rivets in inner row of splice. Trying this for outer row, the deduction is then

$\Sigma y^2 = 0 + 3^2 + 6^2 + 9^2 + 12^2 + 15^2 + 18^2 + 21^2 + \overline{24.125}^2 = 1846\text{ in}^2$

$\frac{7500 \times 1846\text{ in}^2}{26.625\text{ in}} = 519000$ lb. in, which gives resisting moment of net section of web = 1,236,000 lbs. in. It must be larger than this since net section of web allows only 14 rivet holes in outer row.

Moment of bearing value of rivets in both rows on same side of splice must approximately equal resisting moment of net web section.

Sum of moments of bearing value of rivets in inner row of splice in web plate allowing 9630 lbs bearing for 7/8" rivet in 1/2" plate $= \frac{9630 \times 1846 \text{ in}^2}{24.125 \text{ in}} = 733000$ lb-in. 24.125 in being distance from neutral axis to outer rivet in flange. This leaves 1,236000 lb-in. − 733000 lb-in = 503000 lb-in to be resisted by outer row and requires Σy^2 for this row to be $\Sigma y^2 = \frac{503000 \text{ lb-in}^2 \times 24.125}{9630} = 1220 \text{ in}^2$. Omitting rivets in outer row at distances 3", 9", 15" from neutral axis $\Sigma y^2 = 1531 \text{ in}^2$.

Revising resisting moment deduction due to rivets in outer row is $\frac{7500 \times 1531 \text{ in}^2}{26.625 \text{ in}^2} = 437700$ lb-in. This leaves net resisting moment of web plate 1,317,300 lb-in. ~~for outer row~~, and moment of bearing value of outer row is 1317300 lb-in − 733000 lb-in = 584300 lb-in and $\Sigma y^2 = \frac{584300 \text{ lb-in.} \times 24.125 \text{ in}}{9630 \text{ lb}} = 1470 \text{ in}^2$.

This indicates that strength of splice is practically equal to that of net section of web plate since Σy^2 found for bearing value of rivets is close to that Σy^2 of the actual rivets. The splice plates have been taken more than twice the thickness of the web plate.

The strength of the net section of the web plate which is taken through the outer row of rivets is 1317300 ÷ 1755000 = 0.75

or 75% of gross section. One sixth of this or 12.5% of the gross section is the equivalent flange area. Since 12.5% = 1/8 this is exactly what was taken and no change need be made.

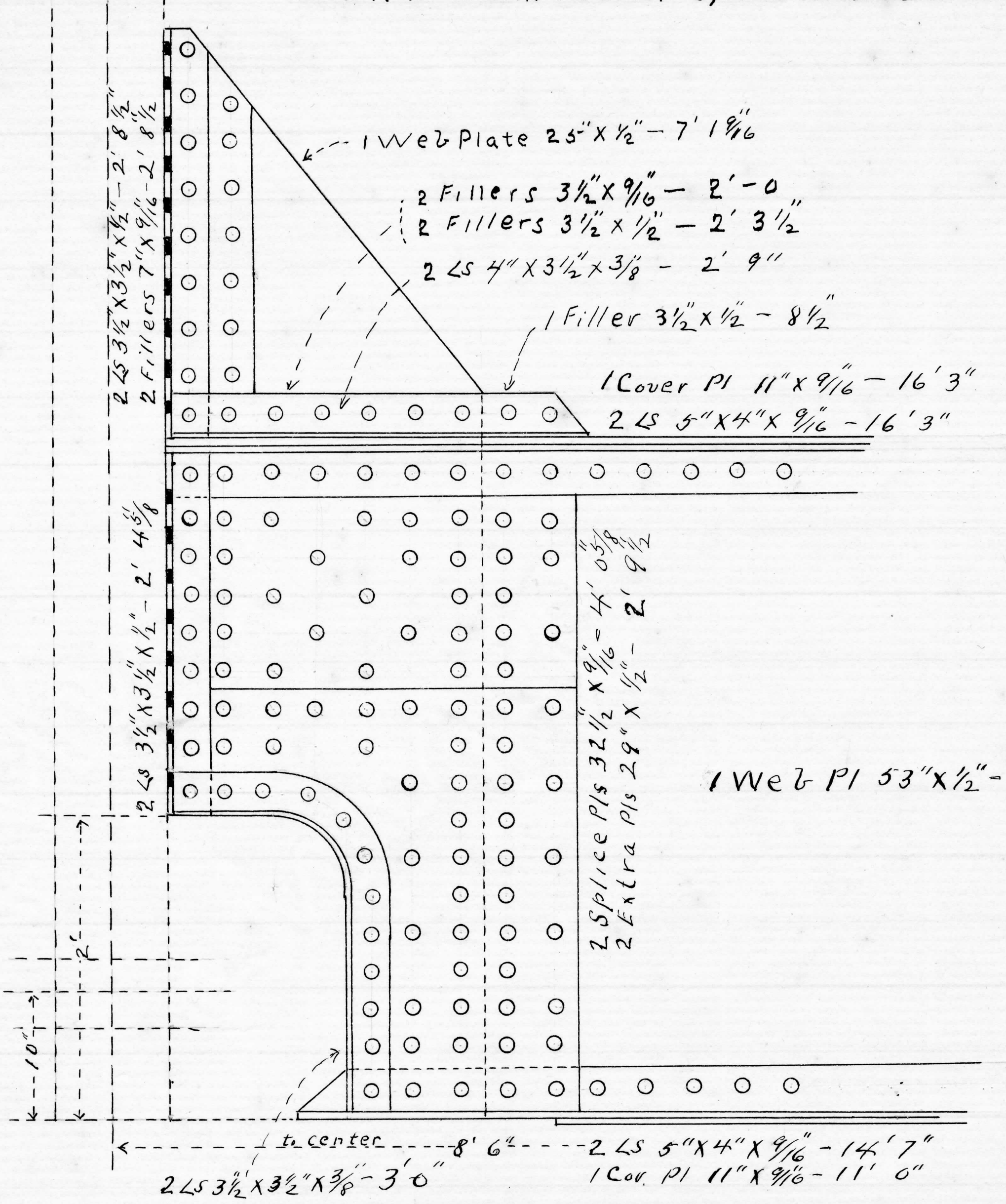
The end connection of the floor beam to the vertical is made as shown in the figure below. The number of shop rivets required to transmit the shear from the web into the fillers which are continuation of the splice plates is found from the bearing value of the rivets in the half inch web plate. This number is end shear divided by bearing value = 192400 ÷ 9630 = 20.
The rivets required to transmit stress to connecting angles are in double shear and their number is 192400 lbs. ÷ 13220 = 15. To connect the end angles to the posts the value of a rivet in single shear governs the number which is $\frac{192400 \text{ lbs}}{6610 \text{ lbs}}$ = 29 shop rivets. Since field rivets are used here the number required is 1/4 more or 36.

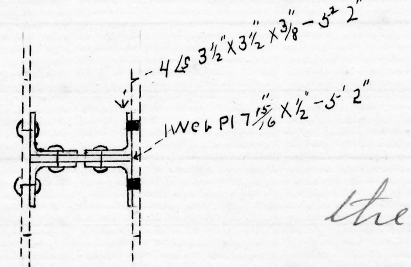
An additional plate is placed around the cut made to allow for pin and members and curved angles are placed around the cut

Two shelf angles 5" x 4" x 3/8 are placed under stringer and riveted to flange of floor-beam.

The upper cover plate is slotted so that web plate spliced can project up through it.

End Connection of Floor Beam





Diaphram

A diaphram is used in each vertical post opposite floor beam to carry part of the shear to the outside channel. The dimensions are given in the figure.

The number of rivets required to connect angles to web are since shear is divided between two angles $= 192400\text{lb} \div (2 \times 9630) = 10$ rivets. These are shop rivets and the same number is required to connect outside angles to channel. The inside angles are held by the same rivets which connect floor beam to channel and these are field rivets.

End Floor Beam.

The end floor beam is designed to take the greatest amount of loading which can come upon it including impact. The reaction at one end is then

Due to Live Load		70700 lbs.
" " Impact		65900 "
" " ½ Stringer		2500 "
" " Assumed wt. ½ floorbeam		2000 "
	Total	140100 lbs.

The bending moment at center of beam is from concentrated loads.
= 138100 lbs × 3.25 ft. = 450000 lb. ft.
and from dead load 6500 lbs. ft. The total moment is then 456500 lb. ft.

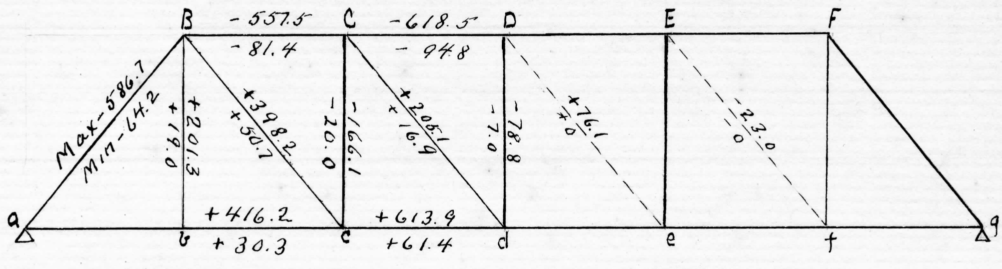
Since end floor beam rests upon bearing plate the depth will be taken 5" deeper than that of intermediate floor beam through depth required is not as great as for the intermediate beam. The web will be 7/16" thick and 58" deep.

The bending moment requires an area of

$\frac{M \text{ lbs. in.}}{h' \times \text{unit stress}} = (456500 \times 12) \div (58" \times 15000) = 6.3 \text{ in}^2$

Two angles 5" × 4" × 9/16 give a net area = 7.24 in.²
As this is greater than the required area no allowance will be made for moment to be taken by web. No cover plate will be used. The rivet pitch in flanges will be the same as for intermediate floor beam.

Stresses in Truss



Span, center to center of end pins	150' 0"
Depth, between centers of chords	30'
Width, between centers of trusses	17'
Number of panels	6
Panel length	25'
Length of end post, center to center of pins	39' 0 1/20"

$\sec\theta = 1.301 \quad \tan\theta = 0.833$

The weight per linear foot of track is 440 lbs, that of stringers and floor beams 540 lbs, that trusses and lateral systems is assumed to be 1100 lbs, making a total of 2080 lbs. The dead panel load is then 26000 lbs. Part of this 7000 lbs is applied at the upper panel points and 19000 lbs at lower.

The live load is Cooper's Class E 50. An allowance is made for impact the coefficient being obtained from formula $I = S\left(\frac{300}{L_1 + 300}\right)$

The static wind load on panel is 3.75 kips, moving wind load on panel is 11.25 kips, and the wind overturning is equivalent to a uniform live panel load of 6.7 kips. The wind stresses are used in finding maximum stresses in truss.

The stresses in the truss members are given in the following table and will be used in designing the rest of the bridge.

	End Post	Upper Chord		Lower Chord	
	aB	BC	CD	bc	cd
Dead Load	-84.5	-86.6	-97.4	+54.1	+86.6
Live Load	-276.0	-266.0	-296.0	+175.0	+266.0
Impact	-184.0	-177.5	-197.5	+117.0	+177.5
Wind overturning On truss, east	-20.4	-13.0	-13.0	+13.0	+13.0
west	+20.4	+13.0	+13.0	-13.0	-13.0
On train, east	-21.8	-22.2	-25.0	+13.9	+22.2
west	+21.8	+22.2	+25.0	-13.9	-22.2
Wind on truss east		+7.8	+10.4	+10.8	+12.1
west		-7.8	-10.4	-10.8	-12.1
Wind on train east				+32.3	+36.4
west				-32.3	-36.4

	Diagonals				Verticals		
	Bc	Cd	De	Ef	Bb	Cc	Dd
Dead Load	+50.7	+16.9	-16.9	-50.7	+19.0	-20.0	-7.0
Live Load	+175.0	+102.0	+48.4	+13.7	+94.6	-78.6	-37.2
Wind overturning On train, east	+14.5	+8.7	+4.3	+1.45	+6.7	-6.7	-3.4
west	-14.5	-8.7	-4.3	-1.4	-6.7	+6.7	+3.4
Impact	+128.0	+77.5	+40.2	+12.5	+81.0	-60.8	-31.2

The design is made from the maximum stresses which occur in each member. There is a max. tension stress in De. Since diagonals are not to be designed for compression a counter will be used in this panel and in 3rd panel to take this stress.

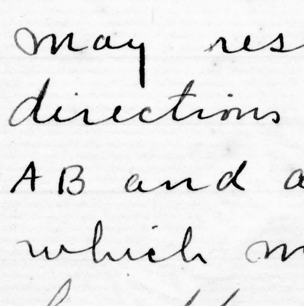
The stresses in lateral bracing due to wind will not be given since the smallest sections for these members allowed by specifications are greatly in excess of the area required by the stress.

Intermediate Posts

<u>Member Cc.</u>

The max compression stress = 166100 lbs. Trial shows that 12 inch channels will be required since ratio of length to radius of gyration must not exceed 100. Taking a 12 inch channel weighing 35 lbs. per foot, radius of gyration = 4.17 in. and length from center to center of pins = 360 in, then l/r = 86.5 From compression formula for unit stress $p = \dfrac{15000}{1 + \frac{1}{13500}(l/r)^2}$, unit stress = 9675 lbs

Required net sectional area 166100 ÷ 9675 = 17.2 in². From hand book these channels give area 10.3 in². Accordingly two 12 inch 35 lb. channels will be used giving an area = 20.6 in². The flanges will be turned inward so as to avoid cutting them at the pin connections. They will be held together by lattice bars which will be designed later.



In order that the column may resist lateral flexure equally in both directions the moments of inertia about axis AB and axis CD must be equal. The spacing which makes these equal is taken from handbook and is 9.6 in. from back to back of channels. Since the channels have some excess strength they will be moved a little from this spacing after the design of the other verticals in order that all may have uniform spacing to make floor beams same length. The uniform spacing has been found to be 9.00 in for all verticals.

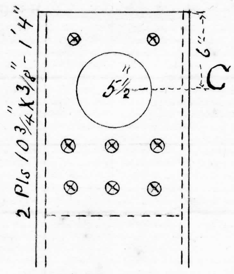
Pin Plates for Cc.

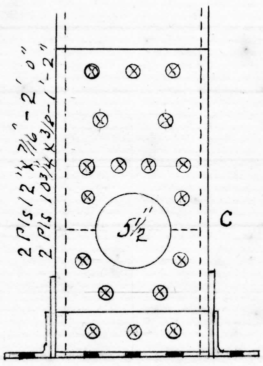
The pins for all points except a and B will have a uniform diameter = 5½". This size has been found fully sufficient for a bridge of 150 ft. span.

In designing pin plates they must be designed for the full working strength of the member and not for the max stress in member.

The pin bearing at bottom of Cc is equal max vertical shear in diagonal Bc. Then this is $\frac{28 \text{ in}^2 \times 15000}{1.301} = 323000$ lbs. The bearing required on each side of post is 323000 lbs ÷ (2 × 5.5 in × 22000 lbs) = 1.335 in on each side. Deducting 0.64 in thickness of channel web 1.335 in − 0.64 in = 0.695 in to be taken by plates. Since 7/16" is thinest plate in which rivets can be counter-sunk according to specifications one 7/16" plate will be used on outside where rivets must be counter-sunk to allow passage of eye-bars, and 3/8" is min. thickness allowed for any plate a 3/8" plate will be used on inside.

Bearing value of rivets used to connect them to channel is 0.875 × 0.64 × 22000 = 12300 lbs in. Bearing value of plates on pin for full working strength is 0.813 × 5.5 × 22000 = 982000 lbs in. Then number of rivets required is $\frac{982000}{12300} = 9$ rivets. More rivets will be used below pin to hold plate to channel as it will be extended to bottom on outside to act as a washer for eye-bars. The dimensions of the plates and the arrangement of the rivets is shown in the figure.





The bearing value on pin at top is working strength of Cc. This is $20.6 \text{ in}^2 \times 9675 \text{ lb} = 201000 \text{ lb in}^2$. The bearing required then is $201000 \text{ lb in}^2 \div (2 \times 5.5 \times 22000) = 0.831 \text{ in}$. on each side. Deducting for channel web the amount to be taken by plate is $0.831 - 0.64 = 0.191 \text{ in}$. One plate 3/8" thick will be used on inside. The number of rivets required is 4 but more than this will be used to stiffen plate.

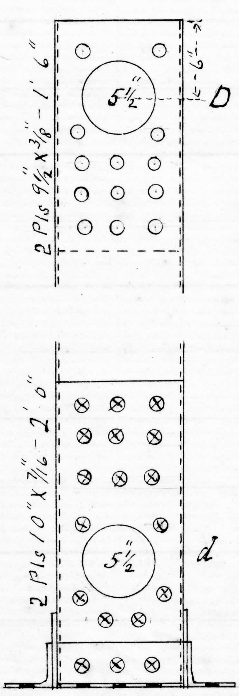
<u>Member Dd</u>.

The total compressive stress = 78 800 lbs. Taking 10 inch channels weighing 15 lbs per ft. $l/r = 93.0$ and from compression formula $p = 9140 \text{ lb in}^2$. Then required area is $78800 \text{ lb} \div 9140 \text{ lb in}^2 = 8.65 \text{ in}^2$. The area of two 10 inch 15 lb. channels is 8.92 in^2, these channels will accordingly be used.

The spacing of the channels to give equal moments of inertia around both axes is 8.89 in back to back of channels. The channels have a slight excess of section and they will accordingly be moved out to a spacing of 9.00 in. to agree with that of Cc.

Pin Plates for Bd.

The pin bearing is equal to the max vertical shear in Cd. This is $\frac{14 \text{ in}^2 \times 15000}{1.301} = 161500$ lb. The linear bearing required on each side is $161500 \div (2 \times 5.5 \times 22000) = 0.6675$ in. Deducting amount taken by channel web leaves $0.6675 - 0.24 = 0.4275$ in to be taken by pin plates. One pin plate on outside will be used 7/16" thick and extending to end of vertical. This is for connection at bottom of Bd.



The rivet bearing in the channel web is $0.875 \times 0.24 \times 22000 = 4620$ lb/in². The pin bearing for the full strength of the plate is $0.4375 \times 5.5 \times 22000 = 52932$ lb/in², then the number of rivets required is $52932 \div 4620 = 12$. The arrangement of rivets is shown in figure.

The pin bearing at top of the post is designed for working strength of member which is $8.92 \text{ in}^2 \times 9140 \text{ lb} = 81500$ lb/in². The bearing required on each side is $81500 \text{ lb} \div (2 \times 5.5 \times 22000) = 0.336$ in. Deducting for channel web leaves $0.336 \text{ in} - 0.24 \text{ in} = 0.116$ in to be taken by plate. Accordingly a plate 3/8" thick will be used on inside. The pin bearing for plate is $0.375 \times 5.5 \times 22000 \text{ lb} = 45320$ lb/in². The number of rivets required is then $45320 \text{ lb/in}^2 \div 4620 \text{ lb/in}^2 = 10$ rivets. The arrangement is shown in figure.

Suspender and Diagonals

Suspender Bb.

The maximum tension in member = 201300 lb.
The composition of the suspender will be like that of the intermediate posts since it is desirable to have a stiff member at this place to take up excessive vibration due to impact.

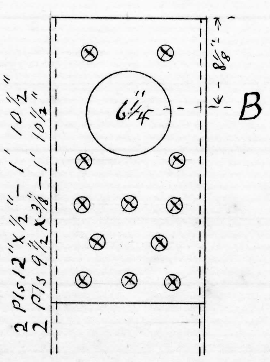
The required net area = $201300 \div 15000 = 13.42$ in². Since this is a tension member rivet holes must be deducted. 2.04 in² will be deducted for two holes in each web and 1.8 in² for one hole in each flange. This is for two 12 inch 30 lb. Channels which give a gross area = 17.64 in². After deducting for rivet holes the remaining area = 13.80 in². These channels will be used spaced 9.0 in. back to back to agree with spacing of intermediate posts.

Pin Plates for Bb.

See next Sheet. * Since Bb is a tension member, in accordance with specifications, its net sectional area at the pin hole must be 40% in excess of the net area of its main body. The area for each side is then $\frac{13.80 \times 1.40}{2} =$ 9.66 in². Taking a plate 12" x ½" on outside and one 9½" x ⅜" on inside the net area of each plate deducting for pin hole is 6.00 in² − 3.12 in² = 2.88 in² and 3.56 in² − 2.34 in² = 1.22 in² giving a total net area for plates = 4.1 in². The net area of channel is 8.82 in² − 3.20 in² = 5.62 in. The total net area at pin hole is then 9.72 in² which is sufficient. It remains to be seen whether the clearance below cover plate of upper chord is sufficient to give required section beyond pin hole. By specifications the net sections along center line of pin hole beyond the hole shall be 70% of net section through pin hole. Then the required distance beyond pin hole is $\frac{9.72 \text{ in}^2 \times 0.70}{0.875 + 0.513} = 4\frac{9}{10}$ in. These plates will be used since distance beyond pin hole to cover plate is $7\frac{3}{8}''$.

* The pins used at connections a and B are $6\frac{1}{4}''$ diameter the larger size being used to give bearing to large area of pin plates required at connections of end post.

The bearing required on each side is $(13.8 \text{ in}^2 \times 15000) \div (2 \times 6.25 \times 22000) = 0.755$ in. therefore bearing area of plates is sufficient.



The bearing value of the rivets in the web of the channel is $0.875 \times 0.513 \times 22000 = 9875$ lb in.

The bearing value of plates on pin is $0.875 \times 6.25 \times 22000 = 120000$ lb in. Making both plates same length the number of rivets required is $120000 \text{ lb in} \div 9875 \text{ lb in} = 12$. The arrangement of the rivets is shown in figure.

The same plates will be used used bottom of Bb since the pin here is $5\frac{1}{2}''$ diameter and the thickness required would be less.

Member Bc.

The max. tension stress in this member = 398200 lbs. Since this is a simple tension member eye bars will be used. The required area 398200 lbs ÷ 15000 = 26.54 in². Two eye bars 8" X 1 3/4" give area = 28 in². These will be used since they possess the proper ratio of depth to width as taken from hand-book. The head will be 17" in diameter to allow bars to go inside of upper chord.

Member Cd.

Counters will be used in this panel to avoid designing diagonal to take compression which would be due to train coming on from right end of bridge. The max. tension stress in member = 205100 lbs. The area to be taken by eye bars = 205100 ÷ 15000 = 13.67 in². Two eye-bars 7" X 1" give an area of 14 in². These will be used.

The max. compressive strength which would come in diagonal = 76100 lbs. This will be a tension stress in counters. The required area is 76100 lbs ÷ 15000 lbs/in² = 5.07 in². One eye bar 5" X 1" gives an area of 5 in². This will be used as counter.

Lower Chords.

Member cd.

The lower chord is a tension member and the two middle panels will be composed of simple eye-bars.

The max tension stress in member = 613900 lbs.

The required section of member is $40.92\ in^2$.

Four eye bars $8'' \times 1\frac{5}{16}$ give area $= 42\ in^2$.

These will be used for member. The diameter of the head outside is 17 inches.

Members ab and bc

Since stress in ab and bc is equal these members will be same section and continuous. It will be a stiff member of built up channels in order to take compression due to traction. Since this compressive stress is very small compared with the max tension in member it will be designed for tension alone.

The max. tension stress in member = 416150 lbs.

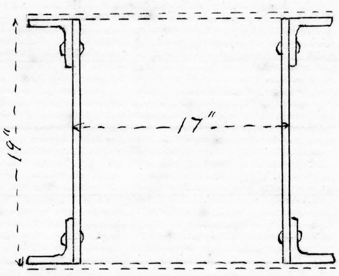
The required net area of member is $416150\ lbs \div 15000\ lbs = 27.74\ in^2$. Since chord must be deep enough to give clearance for eye bar heads it will taken 19" deep. The smallest angles allowable for 7/8" rivets will be used. The pin plates can be so arranged that only 3 rivet holes will be deducted from each web and one from each angle. The spacing of the channels will be such that they can fit inside end post and alternate with diagonal and lower chord eye bars at connection c. This spacing is 17" back to back of channels as shown in figure.

The following composition of lower chord gives sufficient area and will be used.

2 web plates 19" × ½", (19 in² − 3 in²) = 16 in²

4 ∠s 3½" × 3½ × 9/16, (14.52 in² − 2.2 in²) = 12.32 in²

Total = 28.32 in²



The channels are stiffened and held together by lattice bars and tie plates.

Pin Plates for Member ac

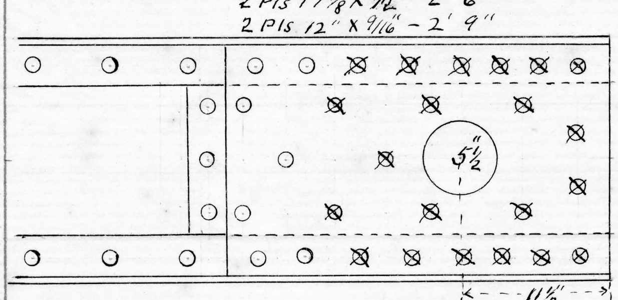
The pin plates for the connection at c will first be designed as they are considerably different from those at (a)

The net section of chord on one side is 14.16 in² then the required net section at pin hole = 14.16 in² × 1.40 = 19.80 in². In determining the net section at pin hole two rivet holes will also be deducted. The following table gives the composition of the section at the pin and the full tensile strength of each piece.

	Net Section	Stresses
1 web plate 19" × ½"	5.75 in²	86200 lbs.
2 ∠s 3½ × 3½ × 9/16"	6.13 in²	93900 lbs.
1 pin plate 12" × 9/16"	4.00 in²	60000 lbs.
1 pin plate 17⅛" × ½"	4.19 in²	62800 lbs
Total	20.07 in²	

The distance beyond edge of pin hole to end of member is found in same way as for Bb. It is $\frac{20.07 \text{ in}^2 \times 0.70}{1.5625 \text{ in}}$ = 9.00 in.

At the end of the pin plates the web takes a stress equal to that carried by the pin by the web plate and inner pin plate so that the only stress to be transferred to angles is that from outer pin plate and the rivets are in single shear.



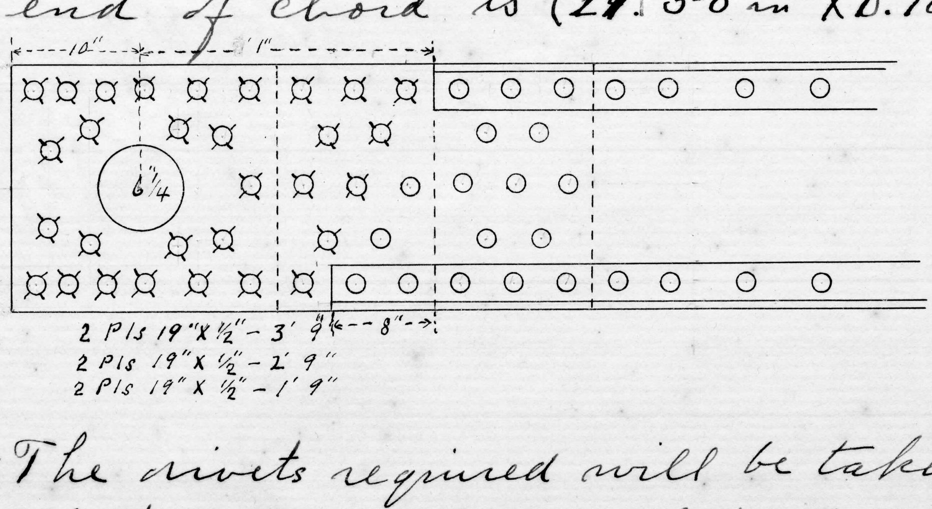
The number of rivets connecting outer pin plate to flanges are 62800 ÷ 6610 = 10. These are to be placed on left of pin. Dividing stress in proportion to bearing web and outer angles take 68000 lbs apiece and inner pin plate 76200 lbs. Then the stress due to bearing of inner plate exceeds that which it can carry past pin hole by 76200 lbs − 60000 lbs = 16200 lbs. Then 16200 lbs ÷ 9630 lb = 2 rivets required on right but more will be used to stiffen member. The pitch of the rivets in flanges is 4 in. on left of pin hole and 3 in on right. The uniform pitch of rivets in flange throughout chord is 6 in. the max. pitch allowed.

At connection b in lower no pin plates are required for bearing stress since stress is same in both panels but a plate will be used to reinforce web where the pin hole is cut out. A plate 12" X 9/16" will make up required area. It will be shown in large drawing.

At connection a the pin plates will have to take a stress in addition to that at c because both angles will have to be cut away entirely to allow the lower chord to fit inside end post. The web plate of lower chord will fit close to inside to inside of post and the section cut away will be made up of pin plates placed inside of chord. The pin here is 6¼" diameter. The net section at pin must be as at c 19.80 in². After angles are cut away the remaining net section allowing for two rivet holes is 5.375 in² for web. The bearing required on each side will be less than the thicknesses of plate needed to make up net section. The following composition gives required area.

	Net Section	Stresses
1 web plate 19" X ½"	5.375	80600
3 pin plates 19" X ½"	16.125	241800
	21.50	

The distance from outer edge of pin hole to end of chord is (21.50 in² X 0.70) ÷ 2 in = 7¼ ins



The rivets required will be taken from full tensile strength of plate according to specifications. The number of rivets required to outermost plate is 80600 lb ÷ 6610 = 12 since the plates will tend to shear off. The number to hold two outer plates is 24 rivets and that to hold all three plates to web is 36. The plates are extended back to engage some of the rivets in the flange angles. The pitch of the rivets is four inches to right of pin holes until pin plates are past, the number of rivets required being considered as only those on right. The arrangement of rivets is shown in figure remembering that the net section was taken for only 3 rivets in section a vertical row.

Upper Chords.

The upper chord will be designed for the panel CD where the stress is the greatest and the same section will be used for BC since the difference in stress between them is not more than 60 Kips.

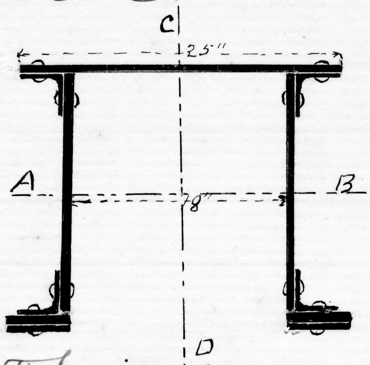
The max. compressive strength in member CD = 618500 lbs. The member will be composed of built up channels with cover plate and flats. The web must be 19" deep to give clearance for heads eye-bars of diagonals which come inside.

Assuming a radius of gyration 40% of depth r = 7.6 in and computing p, unit stress, from compression formula p = 13450 lbs. This gives required area of section = 46 in². The proportion of this to be taken by web is 40% and area of web then equals 18.4 in.^{2} Width of web plate $= \frac{18.4 \text{ in}^2}{2 \times 19 \text{ in.}} = 0.485$ in. A $\frac{1}{2}$" web plate will accordingly be used.

The distance from outside to outside of web will be 18" to allow for pin plates on chord at connection B and to allow vertical and diagonal to come inside of chord. Since practically all of required area will be contained in web plates, cover plate, and flats, small angles will be used. The distance between centers of lines of rivets used to connect cover plate to angles is 22". The thickness of cover plate cannot be less than 1/50 of this or 0.42". A cover plate 25" X $\frac{1}{2}$" will be used. Center line of pins will be taken at center line of web plate and the center of gravity taken 0.5" above this for computation. The angles will project 1/8" beyond edge of web.

To determine area to be taken by flats. The moment of cover plate about assumed center of gravity must equal that of flats about same axis. Then moment of cover plate $= 24" \times \frac{1}{2}"(9.5" - 0.5 + 0.25" + 0.125") = 112.5$
Setting A = area of flats the moment of flats about same axis = A$(9.5" + 0.5" + 0.125 + 0.5) = 10.625$A.
Then A $= 112.5 \div 10.625 = 10.6$ in.^{2} Since plates thicker than 11/16" cannot be punched 2 plates 9/16 thick will be used on each side to form flats. The angles used will be $3\frac{1}{2}$" X $3\frac{1}{2}$ X 7/16. The following

composition of the section gives required area



1 cover plate 25"X½"	= 12.5 in²
2 web plates 19"X½"	= 19.0 in²
4 angles 3½"X3½X$\frac{7}{16}$"	= 11.5 in²
2 flats 5"X1⅛"	= 11.2 in²
Total	54.2 in²

This is considerably more than required but it will be used since it is well balanced. The center of gravity is now found by taking center of moments around top of cover plate. The total moment 539.90 in³ and the center of gravity is 539.90 in³ ÷ 54.2 in² = 10.05 in. from top of cover plate. The distance from top of cover plate to center line of pins = 10.125 in and the eccentricity of the section is then 10.125 in − 10.05 in = 0.075 in. This lies within allowable limits and the section will be used.

Computing moment of inertia with reference to neutral axis AB.

1 cover plate, $\frac{1}{12}\times 25(\frac{1}{2})^3$	= 0.26
$12.5\text{in}^2(0.25+0.125+9.5)^2$	= 1218.00
4 angles, 4×3.26	13.00
$11.48(9.625-1.04)^2$	= 845.00
2 web plates, $2\times\frac{1}{12}\times\frac{1}{2}\times 18^3$	= 485.00
2 flats $2\times\frac{1}{12}\times 5\times(1\frac{1}{8})^3$	= 1.18
$11.2(9.625+0.5625)^2$	= 1160.00
$54.2(0.075)^2$	= − 0.35
	I = 3712.09 in.⁴

The radius of gyration for the section now is found by taking square root of quotient of $\frac{I}{A}$. This is $r = \sqrt{\frac{3712.09 in^4}{54.2 in^2}} = 8.2 in.$

The required area now is 45.4 in² which shows that section used is safe.

<u>Pin Plates</u> for <u>CD</u>.

At connection C the pin bearing is designed to take horizontal component of working stress in CD. This is 134500 lbs. Pin bearing required on each side = 134500 ÷ (2 × 5.5 × 22000) = 0.555. As web is ½" thick a plate 12" × 3/8" will be used. No plates will be needed at B.

The section for BC is same as that used for CD and the pin plates at B will be given in connection with those for end post.

A splice will be made in upper chord near connection D. The section is made to fit flush and plates are placed on four sides of section to give stiffness.

End Post

Member aB.

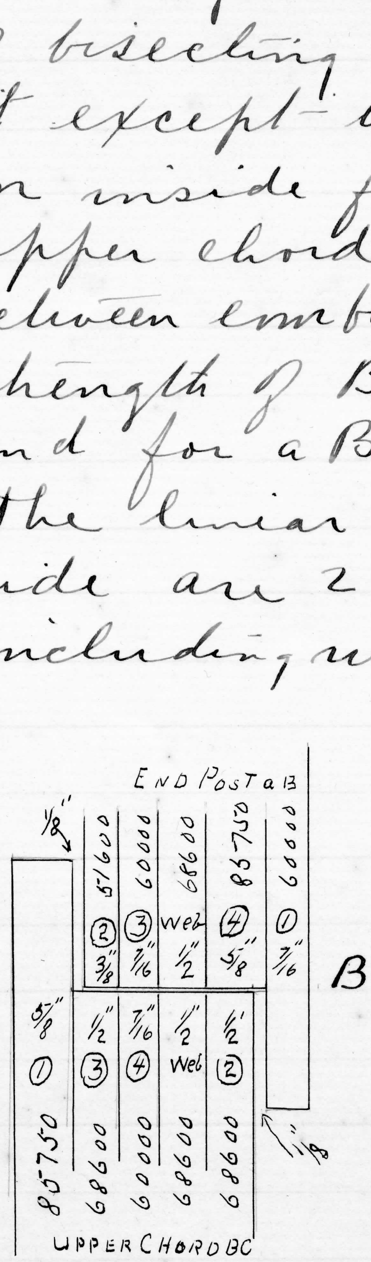
The max. compressive stress = 586650 lbs. The total length between centers of pins is 468.6 in. The length between upper and lower portal struts is assumed 412 in. Since max. stress in this member is less than that in CD the section of the upper chord will be examined to see if it is strong enough to be used as end post. From specifications sum of stresses due to compression and bending moment from wind must not exceed 19000 lb/in² in outermost fiber.

The moment from wind = 5625 lb × 206 in. = 1158750 lb in. The moment of inertia about axis C D is 4225 in⁴. Then stress in outermost fiber due to wind is found from formula $S = \dfrac{Me}{I - \dfrac{Pl^2}{6E}} = \dfrac{1158750 \times 12.50}{4225\,\text{in}^4 - \dfrac{586650 \times (406)^2}{6 \times 26000000}}$ $= 4300$ lb/in². Then total stress in outermost fiber is $\dfrac{586650}{54.2\,\text{in}^2} + 4300\,\text{lb/in}^2 = 15300\,\text{lb/in}^2$.

This is less than the allowable unit stress and the section of the upper chord will be used for the end post.

The radius of gyration for member is $r = \sqrt{\dfrac{4225\,\text{in}^4}{54.2\,\text{in}^2}} = 9$ in. which shows that member is safe against lateral flexure.

Pin Plates for BC and aB at Connection B.
The pin here is 6¼" diameter. All plates and shapes are faced parallel to plane of bisecting angle and about 1/8 from it except two hinge plates on each member on inside for end post and outside for upper chord. There must be 1/8 clearance between combined pin plates. The full strength of BC = 54.2 x 13450 = 728000 lbs. and for aB = 54.2 x 11740 = 636000 lbs. The linear bearings required for each side are 2 5/8 in. and 2 3/8 in. respectively, including web. The arrangement of the plates, their thickness and their bearing stresses are indicated in figure.



The rivets for plates on BC are found in the following way, the method being to get as many rivets in double shear as possible. Considering one side of member the division of stresses is

	Gross area	Stresses
½ cover plate	6.25 in²	
1 upper ∠	2.88 9.13	122500
1 web plate	9.5	128000
1 lower ∠	2.88	
1 flat	5.625 8.505	114200
	27.13 in²	364700 lbs.

The web plate takes 128000 lbs − 68600 lbs = 59400 lbs more stress than it gets directly from the pin bearing and

as 7/16" plate which also serves as a filler between the upper and lower angles is not directly connected to the angles it will be assumed that 59400 lbs of its stress is transmitted directly into web plate and the remainder 9200 lbs is transferred indirectly to angles through other plates. Dividing the 9200 lbs between plates (1) takes 2700 lbs; (2), 2170 lbs; (3) 2170 lbs; (4) 1900 lbs.
Total working stress in (1) = 88450 lbs and requires 88450 lbs ÷ 6610 lbs = 14 rivets since they are in single shear. These 14 rivets pass through (2) also, being in double shear then their bearing in angle will determine the stress they take out of (1) and (2) which is 14 × 8430 = 118000 lbs. The combined working strength of (1) and (2) = 154300 lbs, this leaves 154300 lbs − 118000 lbs = 36300 lbs to be taken by rivets in single shear. Their number is 6 and plate (2) is extended beyond (1) to engage 3 rivets in each angle. The combined strength of (1), (2), (3) = 222900 lbs and the bearing value of the 20 rivets in double shear = 168600 lbs. Then the additional stress to be taken by (3) is 222900 − 168600 = 54300 lbs and 10 rivets will be placed in extension of (3). The number of rivets required to carry stresses from (4) to web and (2) is (59400 + 2175 + 1908) ÷ 6610 = 10 rivets. More than this number will be used to give stiffness to

plates. Three rivets are placed in last vertical row in each plate. The pitch of rivets in angles through plates is 3" and a 5" pitch will be used throughout upper chord beyond fin plates.

The division of stresses for plates on aB is for upper part, 107000 lbs; web, 111500 lbs; lower part 99750 lbs. In same manner as in BC the number of rivets for each plate are found. For ①, 10 rivets; ② 4 additional rivets; ③, 4 additional rivets, and to hold ④ to web and ② and ③ 16 rivets are required.

The arrangement of plates, rivets, and their dimensions are shown on blue print.

End Bearings.

The end bearing is designed for the free end and the same composition will be used at fixed end with substitution of rocker for rollers.

The end reaction of the bridge equals the vertical component of stress in end post which is $\frac{30^{ft}}{39^{ft}} \times 586650$ lbs $= 450000$ lbs. This requires a bearing area on the masonry of 450000 lbs $\div 400 \frac{lbs}{in^2} = 1125\ in^2$. The masonry plate will be taken 30 in. long.

The total thickness of vertical plates for bearing $= 450000 \div (2 \times 6.25 \times 22000$ lbs$)$ $= 1.64$ in. Two vertical plates 3/4" thick will be used and one 3/8" plate to make up bearing. Since both 3/4" plates rest on roller plate only enough rivets will be needed to hold them together. The 3/8" will require $\frac{0.375}{1.875} \times \frac{450000}{2 \times 6610} = 7$ rivets to hold it since rivets are in single shear.

The width of the masonry plate will be $1125\ in^2 \div 30 = 37.5$ in but the actual width will be much greater since it must be sufficient to allow for connecting angles and the bearings of end floor beam. The distance from center of pins will be such that the lower chord will clear the shoe angles which hold the vertical plates. This distance is 24."

The tops of all the floor beams will be the same height so as tops of intermediate floor beams are $43\frac{3}{4}''$ above center line of pins the bottom of the end floor beam will be $-43\frac{3}{4}''=$ below center line of pins. If end floor beam does not rest directly upon bearing plate the space will be filled with a cast steel pedestal.

Two plates, shoe hinge plates, will be riveted on outside of vertical plate to keep end post in line. Since they do not take any stress only enough rivets to hold them in position will be needed. A filler $\frac{13}{16}''$ cut out to fit around hinge plate of end post, which comes outside at this connection, will be used under shoe hinge plates.

The rivets through the vertical legs of the shoe angles have less double shear than bearing so that the double shear will be used to determine their number $= \frac{45000}{13320} = 17$ rivets. Same number will be used in horizontal legs though they are not required.

Segmental rollers, 6" in diameter and $4\frac{1}{2}''$ across narrow part will be used. There must be sufficient clearance between flat portions to prevent binding. This 1.02 in for 6" rollers. For the 30" masonry plate 5 rollers will be used.

The rollers are held in position by guide plates and small angles at the end. The masonry plate is held by anchor bolts 9" long.

Lateral Bracing.

Upper Laterals

For the upper chords the lateral bracing will consist of two pairs of angles with one system of lacing. Since these angles must, in addition to wind stresses, hold the chords in line and prevent vibrations they must be made stiff enough. The section required by the wind stresses is greatly below that required by ratio l/r and ~~so~~ this will be used to determine size of angles to be used.

Using 3/4 rivets in connections, 5/8" rivets in lacing $3" \times 2\frac{1}{2}" \times 3/8"$ angles can be used with their longer legs horizontal. The angle will be spaced 3/4" back to back to allow for lacing. This gives $l/r = 97.5$ which is below the limit and these angles will be used. The net area requires 20 field rivets in end connections. This connection is made by two plates riveted to top and bottom of chord.

Lower Laterals.

Since the lower laterals are riveted to lower flanges of stringers the estimated length between connections is 94". The stress in the first panel is 66,900 lbs. The laterals are designed to take this whole stress in tension or half in compression. Placing two angles $3\frac{1}{2}" \times 3\frac{1}{2}" \times \frac{1}{2}"$ back to back the required area for compression is 3.5 in² and that for tension is 2.24 in² making a total of 5.74 in². The total area for the angles is 7.50 in². Deducting one rivet from each leg of the angles for 7/8" rivets the net section is 5.25 in² while that required for tension alone is 4.48 in². Therefore two angles 3½" X 3½" X ½" back to back will be used in first panel.

In same way the second panel requires two angles 3½" X 3½" X 7/16", and the third panel 3½" X 3½" X 3/8" angles.

The connections require 12 field rivets in first panel, 10 field rivets in second, and 8 field rivets in third panel. The connecting plates are designed to allow for the rivet connections and are shown in drawing.

Portal and Sway Bracing

Portal Bracing.

The portal bracing must be designed to give a clearance of 21' above top of rail. It is shown in large drawing.

The portal struts or diagonals have a length of 144 inches. Two angles 3½" X 3" X 3/8" give an area of 4.6 in² and $r = 1.75$. Since the stress is small compared the requirement for radius of gyration these angles will be used. These angles will be used throughout the bracing since their area is sufficient. The rivets used to connect bracing to end posts are field rivets while the rest are shop rivets.

Sway Bracing.

The sway bracing is of the same general style as the portal bracing and is made up of $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$ angles. Two angles are used at top, connected by plates to chords and held together by lacing. The arrangement is shown in blue print.

Tie Plates and Lacing.

The tie plates are designed as required by specifications, being shorter in tension than in compression members. The are placed near the ends of the members as possible.

The lacing is given in principal dimensions in specifications. Double lacing, making an angle of 45° with a normal to member and held at crossing by single $\frac{3}{4}''$ rivet are used in chords and end post. Single lacing making an angle of 30° with normal is used for verticals.

Conclusion.

This design has been made for all principal members of the truss and all main connections. Many details have been omitted in the design and in the blue print owing to lack of time but the designer has endeavored to give some idea of the main principles of bridge design.