DESIGN

of a

THROUGH RAILROAD BRIDGE

PRATT TRUSS SINGLE TRACK

PIN CONNECTED

RESPECTFULLY SUBMITTED TO
the
DEAN OF ENGINEERING SCHOOL

of

WASHINGTON & LEE UNIVERSITY

as a

THESIS

for the

DEGREE OF BACHELOR OF SCIENCE

in

CIVIL ENGINEERING.

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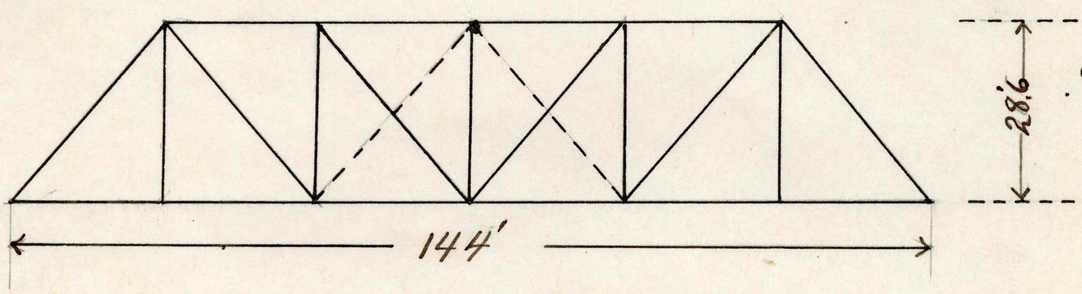
THESIS.

SUBJECT: THROUGH PRATT RAILROAD BRIDGE.

SINGLE TRACK.



PIN CONNECTED TRUSSES.

Span 144 feet. Panels 6. Panel length 24 ft.

Depth 28.6 ft. Length of Diagonal 37.335 ft.

Width center to center of Trusses 16 ft.

Dead Load.

Dead load per panel 30 Kips.

Top panel point 1/3 of 30 or 10 Kips.

Bottom panel point 2/3 of 30 or 20 Kips.

Live Load.

Cooper's Class E - 50 Loading.

(Merriman & Jacoby Part I pp.121.)

$$\text{Tan } \theta = \frac{24}{28.6} = 0.8391 \qquad \theta = 40°$$

$$\text{Sec. } \theta = 37.335/28.6 = 1.3054$$

DEPTH USED = 28.6 ft.

Part 1, page 233. (Merriman and Jacoby)

DISCUSSION OF DEPTH.

Depth should be about 1/3 span for four panels, and about 1/6 span for twelve panels, therefore for six panels the depth should be about 1/5 panel length or 1/5 of 144=28.9 ft., or, by another method:

We have $h/p = \sqrt{\frac{m+1}{3}}$ therefore $h/24 = \sqrt{\frac{6+1}{3}}$ therefore $h = 24 \sqrt{2.33}$, then h = 36.6 ft. which is the economic depth, but economic depth is generally regarded as too large, as there can be considerable variation in depth without increasing the quantity of material.

Part 3. (Merriman and Jacoby.)

The depth may vary 10% from economic depth, without affecting the quantity of material 1%. Therefore depth is about 36 ft. which is too large.

From (I.C.S.) Bridge Specification. The diagonal should not make an angle with the vertical over 40°, i.e., the diagonal should not make an angle less than 50° with the lower chord, therefore we will take the angle as 40 degrees. Therefore tan. 40° = 24/8; therefore h = 28.6 ft.

From Cooper's Specification.

HEADROOM. Clear head room from base of rail is 21 feet, fora width of 6 feet over each track.

CLEAR WIDTH.

Clear width of bridge must be 14 feet, i.e., about 16 feet from center to center of truss.

DEAD LOAD.

Dead Load.

Part I. w = 1100 + 7l where w = weight of bridge

A. in lbs. per linear feet, l = 144 ft.

Therefore, w = 1100 + 7 x 144 = 1100 + 1008 = $2108 \frac{\text{lbs.}}{\text{ft.}}$

From Cooper's Specifications.

Track weight 100 lb. per foot or track, ties, guard timbers shall be taken as 400 lb. per linear foot as minimum. Therefore, suppose we take 440 lbs. per foot as total weight.

From A we have W = 2108 lbs.per linear foot.

Therefore total dead load for bridge per linear foot is W + 440 or 2108 + 440 = 2548 lbs. per linear foot.

Therefore, for one truss, we have 2548 ÷ 2 = 1274 lbs per linear foot. Therefore, dead panel load = 1274 X 24 = 30570 lbs. = 30.57 Kips.

We have from A the Dead Panel load to be equal 30.57 Kips. From B we have the Dead Panel Load to be equal 28.03 Kips.

Part III.

B. W = 600 1+91 where W equal weight of bridge in pounds i.e. for a pin connected bridge (weight not including cross ties, guard-timbers and rails)

Therefore $W = 600 \times 144 + 9 (144)^2 = 86400 + 9 \times 20736$.

Therefore, W = 86400 + 186624 = 273024 lbs. equal weight of bridge.

Therefore weight of truss = 136512 lbs.

" " " one panel = 136512 ÷ 6= 22752 lbs = 22.751 Kips.

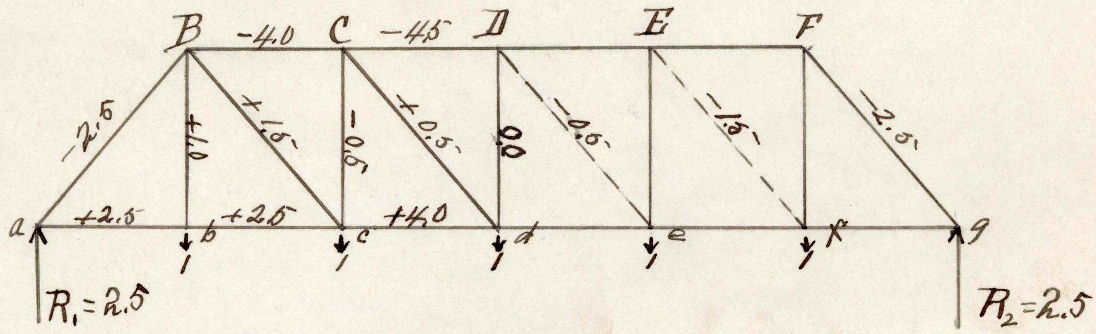
Weight of track equals $\frac{440}{2}$ = 220 lbs per linear foot; therefore for one panel of truss equal 220 X 24 = 5280 lbs. = 5.28 Kips. Dead Panel Load = 22.75 + 5.28 = 28.03 Kips.

Seeing that these two different calculations range close 30 Kips per panel point, we will take the Dead Load per panel as 30 Kips.

Now one-third of this goes to upper panel point, and two-thirds to lower panel point (Part III. p 306)

Dead Load = (Top 10 Kips.) (Bottom 20 Kips.) Total 30 Kips.

Dead Load Stresses.



Factor numbers.

To find the factor numbers, load each panel point with one Kip, therefore total load is equal to five Kips.

Therefore $R_1 = R_2 = 5/2 = 2.5$ Kips.

Compression is noted by - and tension by +

See diagram for factor numbers.

(Ref. Art. 27, part I)

The factor numbers on the web members represent their vertical shears due to the panel loads on diagram. i.e. 1 Kip.

Stresses.

To find Stresses in upper and lower chords multiply factor numbers by panel loads and then by tangent Θ.

Stresses in chords.

Stress BC equals - 4.0 X 30 X .8391 = - 100.69 kips.

Stress CD = - 4.5 X 30 X .8391 = - 113.28 Kips.

Stress ab = +2.5 X 30 X .8391 = + 62.93 " .

Stress bc = + Stress ab = +62.93 " .

Stress cd = + 4.0 X .8391 = +100.69 "

To find stresses in verticals, multiply the total panel load by its factor number and then subtract the upper panel load from this. The upper panel load is 10 Kips.

Stresses in verticals.

Stress Bb =(+1.0 X 30)- 10 = +20 kips.

Stress Cc =(-0.5 X 30) - 10 = -25 Kips.

Stress Dd = (0.0 X 30)- 10 = -10 Kips.

Stress Dd =(+0.5 X 30)- 10 = + 5 Kips.

To find stresses in diagonals multiply factor numbers by panel load and then by Sec. θ.

Stresses in diagonals.

Stress aB = -2.5 X 30 X 1.3054 = - 97.91 Kips

End Post.

Stress Bc = +1.5 X 30 X 1.3054 = 58.74 Kips.

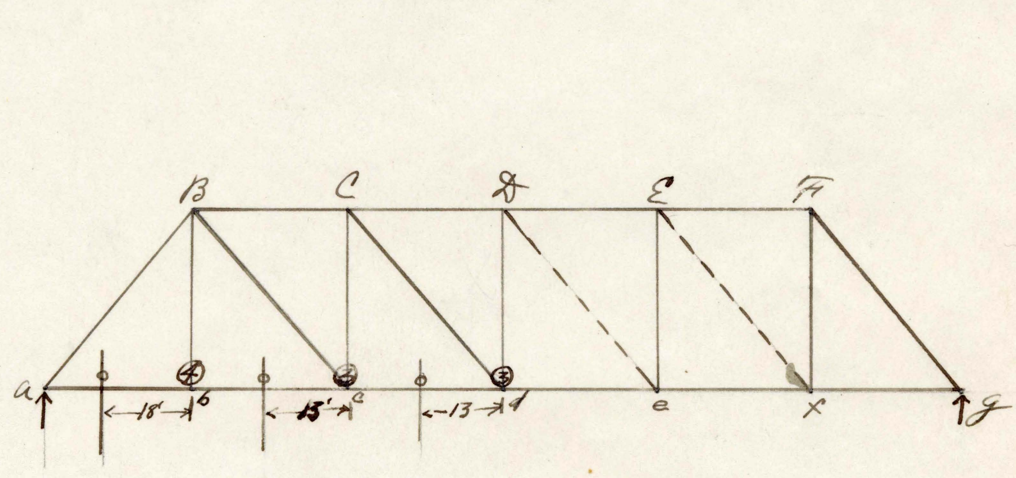
Streee Cd = +0.5 X 30 X 1.3054 = +19.58 Kips.

Stress De = -0.5 X 30 X 1.3054 = -19.58 Kips ⎫
Stress Ef = 1.5 X 30 X 1.3054 = -58.74 " ⎭ Counters

Stress Fg = -2.5 X 30 X 1.3054 = - 97.91 Kips,

End Post.

Stresses due to Live Load.



To find stresses due to Live Load in diagonals.

Diagonal aB.

For first panel, i.e. Stress aB try wheel 4 at b.

From tables $\underline{1}$ = 120 +18 = 138 feet length of train on bridge.

Therefore W = 420+3 X 2.5 = 42.75 Kips, weight of train on bridge. (Below P^1 means P prime.)

$$P^1 = \frac{1}{m} W = \frac{427.6}{6} = 712 \left\{ \begin{array}{ll} \text{wheel (3)} & \text{correctly} \\ \text{" (4)} & \text{loaded.} \end{array} \right.$$

$$R = \frac{M}{l} \quad v_b = \frac{M_4}{p} \quad V = R - v_b \quad S = V \times \operatorname{Sec} \theta$$

$$M = M_1 + V_1 x + \frac{W X^2}{2}$$

$$= 30530 + 420 \times 3 + \frac{2.5 \times (3)^2}{2}$$

$$= 30530 + 1260 + 11.25 = 31801.25$$

$$R = \frac{31801.25}{144} = 220.84 \qquad v_b = \frac{600}{24} = 25 \text{ reaction,}$$

at left panel point due to panel load.

$$V = R - v_b = 220.84 - 25 = 195.8$$

Stress aB = 195.8 X 195.8 X 1.3054 = - 255.6 Kips, Compression piece.

Diagonal Bc.

Try wheel (3) at c.

From table l_t = 24 X 4 + 13 = 109 feet length of train.

W = 355 Kips weight of train of bridge.(Engines.)

$$\text{For Shear } P = 1/m\ W = \frac{355}{6} = 59.13 \left\{ \begin{array}{l} (2) = 37.5 \\ (3) = 62.5 \end{array} \right.$$

$$R \text{ equals } \frac{M}{l} = \frac{M_1 + V_1 X + \frac{Wx^2}{2}}{l} =$$

$$\frac{20455 + 355 \times 0 + 0}{144} = 142.05$$

$$r_c = \frac{287.5}{24} = 11.98$$

$$V = 142.05 - 11.98 = 130.07 = 130.1$$

Stress Bc = 130.1 X 1.3054 = +169.8 Kips Tension piece.

Diagonal Cd.

Try wheel (3) at d.

From table l_t = 85 feet (Engines only)

W = 290 Kips. (wheel (2) = 37.5} Correctly

P = $\frac{290}{6}$ = 48.3 (" (3) = 62.5} loaded.

$$R = \frac{M}{l} = \frac{M_1 + V_1X + \frac{WX^2}{2}}{l} = \frac{10910 + 290 \times 6}{144}$$

$$= \frac{12650}{144}$$

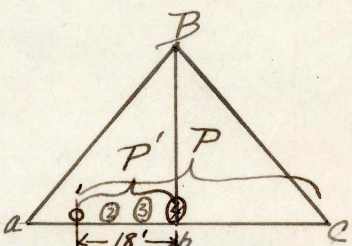
$$= 87.82 \quad r_d = \frac{287.5}{24} = 11.98$$

Therefore V = 87.82 - 11.98 = 75.84

Stress Cd = 75.84 X 1.3054 = +98.95 Kips tension piece.

Line Load Stresses in Verticals.

Vertical Bb



l_t = 18 + 24 = 42 feet.

P = 145

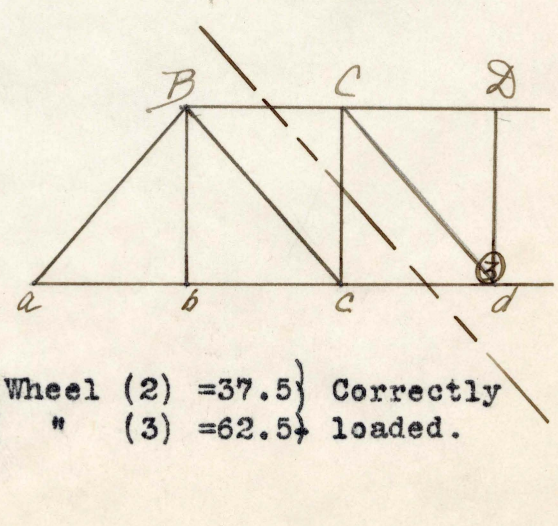
$P' = \frac{145}{2}$ = 72.5 { Wheel (3) = 62.5 Correctly

" (4) = 87.5 loaded.

$$r_b = \frac{Mc - 2Mb}{p} = \frac{2693.75 - 600 \times 2}{24} = 62.24 \text{ Kips}$$

Stress Bb = +62.24 Kips as stress equal to shear in verticals.

Vertical Cc



To get maximum shear in verticals load to right of section.

Try wheel (3) at d

l = 24 X 3 + 13 = 85 ft.

W = 290 (Wheel (2) =37.5} Correctly

Therefore p = $\frac{290}{6}$ = 48.3 (" (3) =62.5} loaded.

$$R = \frac{M}{l} = \frac{M_1 + V_1 X + \frac{W X^2}{2}}{l} = \frac{10910 + 290 \times 6}{144}$$

$R = 87.82 \quad r_c = \frac{287}{24} = 11.98$

Therefore $V = 87.82 - 11.98 = 75.84$

" Stress Cc = 75.84 Kips. (In verticals the vertical shear is the same as the stress.)

Vertical Dd

Try wheel (2) at e

From table $l_t = 24 \times 2 + 8 = 56$ ft.(Engines)

W = 190 Kips (Weight of engines.)

$P = 1/6 \times 190 = 31.7 \begin{cases} \text{wheel (1)} = 12.5 \\ \quad '' \quad (2) = 37.5 \end{cases}$ Correctly loaded.

$R = \frac{M}{l} = \frac{5790}{144} = 40.2 \quad r_d = \frac{100}{24} = 4.1$

Therefore $V = 40.2 - 4.1 = 36.1$ Kips.

Therefore Stress Dd = - 36.1 Kips Compression piece.

Live Load Stresses in Upper Chords.

(1) Assume some wheel at the center of moment of the required chord.

(2) Find length of train on bridge = l_t

(3) " weight of train " " = w_t

(4) Then for chord stresses $P^1 = \frac{n^1}{m} W$ is that condition that must be fulfilled $n^1 =$ number of panels from the left. (P^1 means P' i.e. 1 = prime.)

(5) Find reaction at left support.

(a) find moment M about a point (about right support.)

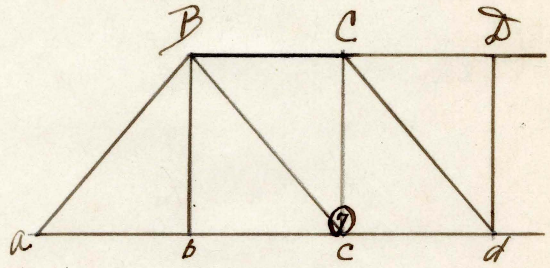
(b) $R = \frac{M}{l}$

(6) Find bending moment $M_b = R\,(n^1 \times p) - M_e$

(n^1 = number of panels and M_θ = moment at wheel found from table 41a)

(7) Therefore Stress = $\frac{Mb}{d}$ where d is depth of truss.

Chord BC
Take center of moments at c .
Try wheel (7) at c, the center of moments.



$l_t = 4 \times 24 + 37 = 133$ feet length of engines and train.

Therefore $W_c = 40.75 + 2.5 \times 3 = 415.$

$P = \frac{n^1}{m} W$ here $n^1 = 2$ $m = 6$

$P = \frac{2}{6} \times 415 = 138.3$ (wheel (6) = 128.75) (" (7) = 145.00) Correctly loaded.

$M = M^1 + V^1X + \frac{W X^2}{2} = 28461.25 + 407.5 \times 3 + \frac{2.5 \times 9}{2}$

$= 28461.25 + 1222.5 + 11.25 = 29695$

$R = \frac{29695}{144} = 206.21$

$M_c = R \times pn^1 - M_7$

$= 206.21 \times 2 \times 24 - 2693.75 = 7204.33$

Stress BC $= \frac{Mc}{d} = \frac{7204.33}{28.6} = -251.9$ Kips

Compression piece.

Chord CD

Take center of moments at d.

Try wheel (11) at d

$l_t = 24 \times 3 + 64 = 136$ feet (Engines and train)

Therefore $W = 420 + 2.5 \times 1 = 422.5$ Kips; $n' = 3.$

$P = 3/6 \times 422.5 = 211.25$ (wheel (10) =190) (" (11) =215) Correctly loaded

$M = 30530 + 420 \times 1 + \frac{2.5 \times (1)^2}{2} = 3095.25.$

$R = \dfrac{30951.25}{144} = 214.93 \qquad M_{11} = 7310$

$M_d = 214.93 \times 3 \times 24 - 7310 = 8164.96$

Stress CD $= \dfrac{8164.96}{28.6} = -\ 285.4$ Kips Compression piece.

Live Load Stresses in Lower Chords.

To find stresses due to live load in lower chords apply the same rule as for upper chords assuming the wheel directly under the center of moments.

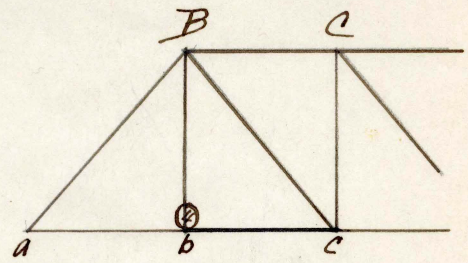
Chord bc

Take center of moments

at B

Try wheel (4) at b below

the center of moments.



$l_t = 24 \times 5 + 18 = 138$ feet (Engines and train)

$W = 420 + 2.5 \times 3 = 427.5$ Kips

$P = \dfrac{nl}{m} W = 1/6 \times 427.5 = 71.25 \begin{cases} \text{wheel } (3) = 62.5 \\ \quad " \quad (4) = 87.5 \end{cases}$ Correctly loaded.

$M = 30530 + 420 \times 3 + \dfrac{2.5 \times 9}{2}$

$= 31801.25$

$R = \dfrac{31801.25}{144} = 220.84 \qquad M_4 = 600.$

$M_b = 220.84\ (1 \times 24) - 600 = 4700.16$

Therefore Stress bc $= \dfrac{4700.16}{28.6} = +\ 164.34$ Kips

Tension piece.

Stress in ab = Stress bc.

Therefore Stress ab = +164.34

Chord cd

Stress in cd is the same as the stress in upper chord BC

Stress cd = +251.9 Kips (Tension piece)

Impact.

$I = S \left\{ \dfrac{300}{L + 300} \right.$ S is the computed live load stress in the member and L is the loaded distance infeet that produces the greatest stress in the member.

Impact in diagonals.

$$I_{aB} = -255.6\left(\frac{300}{138+300}\right) = -173.8 \text{ End Post.}$$

$$I_{Bc} = +169.8\left(\frac{300}{109+300}\right) = +124.5$$

$$I_{Cd} = +98.95\left(\frac{300}{85+300}\right) = +77.1$$

Impact in verticals.

$$I_{Bb} = +62.24\left(\frac{300}{42+300}\right) = +54.5$$

$$I_{Cc} = -75.84\left(\frac{300}{85+300}\right) = -59.1$$

$$I_{Dd} = -36.1\left(\frac{300}{56+300}\right) = -30.4$$

Impact Upper Chords.

$$I_{BC} = -251.9\left(\frac{300}{133+300}\right) = -174.5$$

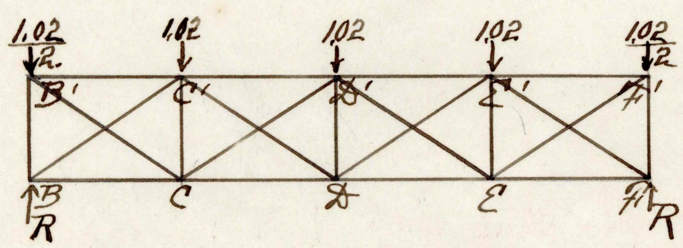
$$I_{CD} = -285.4\left(\frac{300}{136+300}\right) = -196.3$$

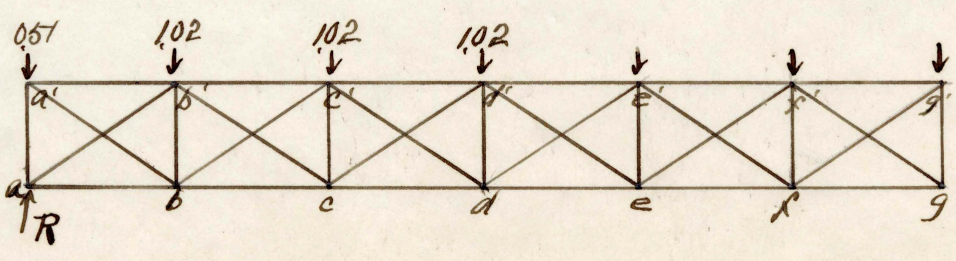
Impact in lower chords.

$$I_{ab} = I_{bc} = +164.3\left(\frac{300}{138+300}\right) = +112.5$$

$$I_{cd} = I_{CD} = \qquad +174.5$$

To find wind stresses in upper and lower laterals:





Take wind load as 30 lb. per square foot.

Consider all these members as being one foot wide.

Therefore, total length of all members in one truss is 144 + 96 + (8 x 37.34) + (5 x 28.6)= 681.7 foot.

681.7 x 1 = 681.7 sq. feet.

681.7 x 30 = 20451.0 lbs.= 10.225 tons.

Number of panels = 6+ 4 = 10.

Therefore panel wind load =

10.225 ÷ 10 = 1.02 tons.

Stress in top lateral:

The top laterals are to be designed for a fixed horizontal force of 85 lbs. per linear foot = $\frac{1.02 \times 2000}{24}$ = 85 lbs., and the bottom lateral for a fixed horizontal force of 440 pounds per linear foot, additional.

Panel dead load for top and bottom laterals = 85 x 24 = 2040 lbs. Panel live load for bottom laterals only = 440 x 24 = 10560 lbs.

Length of diagonals = $\sqrt{28.6^2 + 24^2}$ = 37.335

Top Lateral Stresses.

Shear in Panels.

BC = 2040 x 1-1/2 = 3060 1st.

CD = 2040 x 1/2 = 1020 2nd.

Stresses in Diagonals.

3060 x $\frac{37.335}{17.0}$ = 2.2 = 6732 1st.

1020 x 2.2 = - 2244 2nd.

Bottom Lateral Stresses.

Shear in panels.

ab = (2040 x 2-1/2) + (10560 x 15/6) = 30582.

bc = (2040 x 1-1/2) + (10560 x 10/6) = 20660.

cd = (2040 x 1/2) + (10560 x 6/6) = 11580.

Stress in Diagonals.

1st. 30582 x 2.2 = - 67280 Kips. $\left(\frac{37.335}{17.0} = 2.2\right)$

2nd. 20660 x 2.2 = - 45452 "

3rd. 11580 x 2.2 = - 25476 "

Stresses in Portal Strut. (This is one method of working stresses, see second method in design of portal braces.)

There are 2½ panel loads of wind force applied at the top of portal struct = 2040 x 2½ = 5100 pounds. This force is assumed to be resisted equally at the foot of each post. It is also assumed that each post is fixed at bottom, and that the plane of contraflexure is half-way between the foot of posts and the lower extremities of portal struct. Then, for the purpose of figuring the portal stresses, the ends of the posts may be considered to lie in this plane, as shown in Plate I.

The horizontal reaction at the foot of each post = 5100 x ½ = 2550 pounds, and the bending moments at the knee connection due to these forces = 2550 x 14.5 = 36975 These moments are resisted by forces at the top of post acting with lever arms of 8.2 feet which forces equal 36975 ÷ 8.2 = 4509. The force of 4509 pounds on the leeward side of portal induces a tensile stress of the same amount in this side of the top struct; and on the windward side, the force of 4509, combined with the applied force of 5100 pounds induces a compression stress = 4509 + 5100 = + 9609 pounds.

The horizontal force at the lower end of each knee-brace is equal to the induced force at top of post plus the horizontal reaction at its foot = 4509 + 2550 = 7059 pounds, and stress in the knee-brace is equal to the horizontal force at its foot multiplied by its length and divided by one-half the width of portal = $7059 \times \frac{115}{8} = 10147.3$ pounds. This stress will be tension on the individual side of portal and compression on the leeward side.

Floor Stringers.

The span of the stringer equals the panel length of the truss or 24 feet. The Dead Load on one stringer consists of one-half the assumed weight of floor plus the weight of stringer, = 440 pounds per linear foot. For the maximum live load reaction or end shear, wheel (2) is placed over one support with wheels 3, 4, 5 and 6 on the stringer. Moments of these loads are then taken about the opposite support and divided by the span.

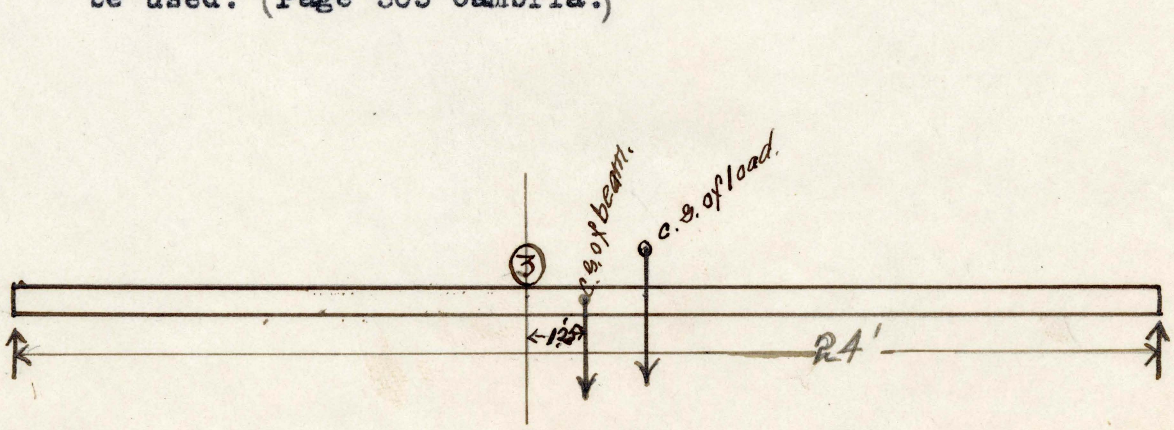
End Shear.

Dead Load = 440 lbs. x 24 x $\frac{1}{2}$	=	5280
Live Load = $\frac{2500\ (9+14+19+24)}{24}$	=	68750
Impact = $\frac{(68750)^2}{68750 + 5280}$	=	63750
		137780 lbs.

For the specified shearing stress of 10000pounds per sq.in. by Cooper's Specifications.

137780 $\div$ 10000= 13.78 sq. in. Area., required for web plate.

A 42 x 3/8 in. web plate = 15.75 sq. in. will be used. (Page 305 Cambria.)



Suppose we try wheels (2), (3), (4) and (5). Section of maximum moment.

As these loads are same the C. G. is midway between (3) and (4). Therefore section of maximum moment passes through wheel (3) which under the condition must be placed 1.25 feet to left of center of beam. Therefore section of maximum moment is 12 - 1.25 = 10.75 feet from left reaction. Therefore moment about (3) is $M_B = R \times 10.75 - 2.5 \times 5$, but $Rl = M$; therefore $R = \frac{M}{l}$

$$M = M_5 + V_5 X + \frac{WX^2}{2}$$

$$= 1037.5 + 172.5 \times 3.25 - 12.5 \times 26.25$$

$$= 1075.0$$

$$R = \frac{1075.0}{24} = 44.8$$

$$M_B = 44.8 \times 10.75 - 2.5 \times 5 = 356.6 \text{ Kips feet.}$$

M_B = 356600 lbs. feet.

Moment

Dead Load = $1/8 \, Wl^2 = \frac{440 \times (24)^2}{8} = 31680$

Live load = 44800 x 10.75 - (23000 x 5 = 366600

Impact $\frac{(366600)^2}{366600 + 31680} = 337800$

Total = 736080 ft. Lbs.

The effective depth of stringer or distance C to center of gravity of flanges will be about 3.25 feet

Flange stress = 736080 ÷ 3.25 = 226480 lbs.

Flange area required = 226480 ÷ 1600=14.16 sq.in.

Then 1/8 of 42"x 3/8 in. web plate = 1.96 sq.in.net

2 angles 6"x 6"x 5/8 in.=14.22 gr.
- 2 In. web holes =12.97 " " "
Net area----------- =14.91 sq. in. (one hole 1 in. diameter in each angle.)

Intermediate floor beams.

The effective length of floor beam is assumedto be equal to the distance center to center of trusses = 16 feet. The stringer concentrations 6.5 feet apart and 4.75 feet from center of trusses. The weight of floor beam is assumed to be 3000 lbs., which is a distributed load. The Dead Load concentrations from stringers = 440 x 24 = 10560 pounds. The Live Load concentrations from stringers, which are equal to the maximum panel. Concentration at b is found by the following method:

Panel length = 24 feet. Placing wheel (5) at b the load P^1 in the panel ab varies from 87.5 to 112.5 kips and $2P^1$ from 175 to 225 Kips. The total load P on both panels varies from 161.25 to 177.5, thus satisfying the criterion $P = 2P^1$

$$R_b = (4370 - 2 \times 1037.5)/24 = 95620$$

End Shear

Dead Load = $(3000 \times \frac{1}{2}) + 10560 = 12060$

Live Load = 95620

Impact = $\dfrac{(95620)^2}{95620 + 12060}$ = 85100

192780 lbs.

Area required in web plate = 192780 ÷ 10000 = 19.28 sq.in. We will use this.

Moment.

Dead Load = $\dfrac{3000 \times 16}{8} + 10560 \times 4.75 = 56160$

Live Load = 95620 x 4.75 = 454000

Impact $\dfrac{(454000)^2}{454000 + 56160}$ = 403000

913160 lbs.

Assuming an effective depth of 53 in. or 4.25 feet and using the specified unit stress.

Flange stress = 91360 ÷ 4.25 = 214800 lbs.

Flange area required = 214800 ÷ 16000 = 13.43 sq. in.
Then 1/8 of 54 in. x 3/8 in. web plate = 2.53 sq.in.net.
2 angles 5 in. x 4 in. = 7.5 sq.in.gr. = 6.60 " " "
1 plate 13 in. x 3/8 in. = 4.87

Net area, -------- = 14.00 sq. in.

2 holes 1 in. in diameter in each angle is accountdd for in the angles.

End Floorbeams.

The effective length and location of stringer concentration are the same as for intermediate floorbeams.

The weight of floorbeams = 3000 lbs.

Dead Load concentrations from stringers = 440 x 12 = 5280 pounds.

Live Load concentration from stringers as detemined in connection with stringer = the maximum panel concentration at a is found by the following method: Panel length = 24 feet. Placing wheel (4) at a.

$$R_a = \frac{2050 - 2 \times 100}{24} = 77080$$

End Shear

Dead Load = $(3000 \times \frac{1}{2}) + 5280 =$ 6780

Live Load = 77080

Impact $= \dfrac{(77080)^2}{77080 + 6780}$ $= \underline{70850}$

154710

Area required in web plates = 154710 ÷ 10000 = 15.47 sq. in.

We will use this

Moment

Dead Load $= \dfrac{3000 \times 16}{8} + 5280 \times 4.75$ = 31080

Live Load = 77080×4.75 = 366100

, Impact $= \dfrac{(366100)^2}{366100 + 31080}$ $= \underline{337800}$

734980

Assuming an effective depth of floorbeams = 4.33 feet. Flange stress = 734980 ÷ 4.33 = 169500 pounds.

Flange area = 169500 ÷ 16000 = 10.05 sq.in.

Then

1/8 of 54 in. x 3/8 in web plate	= 2.53 sq.in.
2 angles $3\frac{1}{2}$ in. x $3\frac{1}{2}$ in. = 4.22 sq. in. gr.	= 3.42 " "
1 plate 13 in. x 3/8 in.	= 4.87
Net	10.82 sq.in.

(1 hole one in. in diameter in each angle.)

The connection angles for stringers or floor beams shall have no leg less than 3½ inches or be of less thickness than ½ inch. (See blue prints for angles used.)

Design of Members.

In designing compression verticals you want the lightest channels possible with the greatest stiffness possible, and also the web cannot be less than 3/8 in. or .38 inch thick.

Bb

The least width of posts from Cooper's Specifications is 10 inches.

The vertical suspender Bb is designed to take tension as it receives stress only from loads on first two panels, and also designed to take compression as it receives impact more directly, and this design reduces the excessive vibration. Therefore its composition will be nade on order of other verticals.

For tension allow one hole in each flange and two holes in web of channel. In suspender 4 holes in angles and 4 in webs will be required.

Required web section area

$= \frac{136740}{15000} = 9.2$ sq. in. approx.

2 channels 12"; 25 lb. = 14.70 sq. in. t = .39

4 holes in web 4 x .39 x 7/8 = 1.36

4 " in flange 4 x .50 x 7/8 = 1.96

3.12

Therefore a web sectional area = 14.70 - 3.12 = 11.58 sq. in., which is greater than 9.2 sq. in., but will use this combination as the channels may be weakened some where floor beam is riveted to it.

Cc Neglecting the wind stresses, which are relatively small, in accordance with the specified unit stresses, the total stress to be considered is 159940 pounds. A trial shows that 15 in. channels are required, (from Cambria pp.256-257)

15 in channel 40 lb. r = 5.44 t = .52 gross area = 23.52 sq. in.

$$\frac{L}{r} = \frac{28.6}{5.44} = 5.25$$

Ultimate strength in lbs. per sq. in. from Cambria pp. 202-203

$$\frac{L}{r} = 5.25 = 40967$$

Therefore to obtain safe unit stress for moving loads as in bridges, divide 40967 by 5 = 8193.4

$$\frac{159940}{8193.4} = 19.52 \text{ sq. in = net area required.}$$

Area of two 7/8 inch holes in web = 2 x 7/8 x .52 = .91 sq. in.

Area of 4 flange rivet holes = 4 x 7/8 x .65 = 2.28 sq. in. Total area to be deducted for ~~xxxx~~ holes is .91 + 2.28 = 3.19 sq. in.

Gross area of channels = 23.52

Net " " " = 23.52 - 3.19 = 20.33

Net area required is 19.52. There is an excess of 0.81 sq. in., therefore we will use this combination.

Now turning the backs out and spacing the channels, so as to have the same strength both ways. The moment of inertia must be the same about both axes. Using tables on pp. 227, Cambria, we have E = 12.3 in.

Dd The total stress is 76500 lbs. Trying a 10 in. 25 lb. t = .53 r = 3.52 $\frac{L}{r} = \frac{28.6}{3.52} = 8.1$

The ultimate strength from table in Cambria = 32790 and the safe unit stress = $\frac{32790}{5} = 6558$

$\frac{76500}{6558}$ = 11.7 net area required to stand this stress.

Now 2(10 in: 25 lbs.) beams have section area = 7.35 x 2 = 14.7 sq. in. Deducting area of 4 ~~unit~~ holes as this is as many as can come in one section = $\left\{\frac{.62+24}{2} +.24\right\}$ x 7/8 x 4 = 1.52 sq. in. , also one rivet for each web = .53 x 7/8 x 2 = .94

Now 14.7 - (1.52 + .94) = 12.24 sq. in. web area left.

12.24 - 11.7 = 0.54 sq. in., which is an excess. We will use this. E = 12.3 as E has to be the same forall verticals in order to make the floor beams the same length.

Design of Diagonals.

Bc Since the maximum stress in Bc is a tension of 353040 pounds, it may be composed of one or more pairs of eye bars. For a unit tensile stress of 15000 pounds per sq. unit, the sectional area must be 353040 / 15000 = 23.53 sq. inches. Two eye bars 8" x 1½" provides an area of 24 sq. in. The thickness of eye-bars ranges in practice from one-fourth to one-seventh of their depth or width. These bars come in the limits of these.

Cd Sincethe maximum stress in Cd is tension of 195630 lbs., it may be composed of one or more eye-bars. for a unit tensile strength of 15000 pounds per sq. in., the sectional area must be 195630/1500 = 13.04 sq. in. = required net area.

Take two eye-bars 7" x 1"= 14 sq. in., therefore will be used. See page 339, Cambria Head of 15½ inches with 5½ in. pin hole.

Dc Counter.

The area of counters shall be determined by taking the differences in areas due to the live load and dead load strains considered separately; the counters in any one panel must have a combined sectional area of at least three square inches, or else must be capable of carrying all the counter live loads in that panel.

Live Load Stress	=	47040	pounds.
Impact	=	39510	"
Total Stress in Dc	=	86550	"

$\frac{86550}{15000}$ = 5.77 sq. in., net area required.

Take one eye-bar 6" x 1" = 6 sq. in.

Head = 15½ in. pin hole = 5½ inches. We will use this eye-bar.

Design of Lower Chords.

ab=bc The stresses which govern the design of ab and bc are the same and hence a single member may be extended from a to c. The required net area = 339770/15000=22.65sq. in., since the stress in ab = 339770 lbs. Let ac be composed of two built channels. Since the eye-bar heads of the 8 inch eye-bars that will be used for cd are 17 inch deep according to the handbook, let the web plates be made 18 inches deep so as to avoid cutting the angles in order to pass the eye-bar head at c. Selecting two web plates 18" x ½" = 18 sq.in. 4 angles 3" x 3" x 3/8" = 8.44 sq.in.

Total area gross = 18 + 8.44 = 26.44 sq. in. The rivets in the end pin plates can be so arranged as not to deduct more than three rivet holes in each web plate, and one in each angle, giving a web area of 23.64 sq. in.

cd If the wind stress be neglected, the required net area for the lower chord member cd is 527090/15000= 35.14 sq. in., required web area, where 527090 lbs., is the stress in cd.

Take four eye-bars 8" x 1-1/8" = 36 sq. in. therefore we will use these.

The specifications also require that if the unit stress due to the weight of a member is greater than 10% of the safe value allowed/ the sectional area must be increased. To test this, use the formula found in Mechanics of Materials, Art. 103, that; $S. = \dfrac{Mc}{I + \dfrac{nPl^2}{ME}}$

Where P = 169885 lbs. l = 288 in. E = 29000000 $\dfrac{lb}{in^2}$

Medium steel $\dfrac{M}{n}$ = 96. Vol. of bar =

8/12 x 1.125/12 x (24 + 3) = 2.25 cu. ft. Therefore, weight of bar is 2.25 x490 pounds per cu. ft. = 1102.5 lbs. = wl. $M = 1/8\ wl^2 = 1/8 \times 1102.5 \times 24 \times 12 = 39690$ ft. lbs. c = 4 in. $I = 1/12\ bd^3 = 1/12 \times 9/8 \times 8^3 = 64\ in.^3$

$$S = \frac{39690 \times 4}{64 + \dfrac{1 \times 169885 \times 288^2}{9.6 \times 29000000}} = \frac{158760}{64 + 50.6} = 1387$$

1/10 per cent of $\dfrac{15000}{1}$ = 1500; therefore 1387 comes within this limit, and the section area will not have to be increased. The same result is obtained for any thickness of bar, hence this determination is all that will be required for all bars of the same depth.

BC

Upper Chords Design.

Since the specified unit stress involves the radius of gyration, an approximate value must be assumed. A convenient rule makes the radius of gyration about a horizontal axle equal to 4/10 of the depth out to out. This depth is estimated to be 19.44 inches making r = 7.78 in. $\dfrac{l}{r}$ = 37.02

$$p = \frac{15000}{1 + \dfrac{1}{13500} \times \left(\dfrac{l}{r}\right)^2} = 13550 \text{ pounds.per}$$

square inch, and the required sectional area is 527090/13550 = 38.80 sq. in., where 527090 lbs. is stress in BC.

The composition of the section is as follows:

1 cover plate 26" x 3/8"	=	9.75 sq. in.
4 angles 3" x 3" x 3/8"	=	8.44 " "
2 web plates 18" x 3/8"	=	13.50 " "
2 flats, 4" x 1"	=	8.00 " "
Total		39.69 " "

In riveting these together, we will use the minimum spacing of rivets for two feet on each side of joint increasing spacing toward the middle, but never make the spacing greater than six inches.

CD

To find an approximate value for the radius of gyration about a horizontal axis. Take 4/10 of depth out to out of section. Estimated depth = 18" + 7/16" + 1 in. = 19.44 in. Therefore r = 4/10 of 19.44 in. = 7.78 in.

Then $\frac{L}{r} = \frac{(24 \times 12)}{7.78} = 37.02$

$$p = \frac{15000}{1 + \frac{1}{13500}\left(\frac{l}{r}\right)^2} = \frac{15000}{1 + \frac{1}{13500}(37.02)^2}$$

= 13550 unit stress to be used in this piece.

Total stress in CD = 594980 lbs.

Therefore, section area $= \frac{594980}{13550} = 43.91$ sq.in., Net area required.

Width of cover plate = 18.5" + 6" + 1" = 25.5", therefore will use cover plate = 26 inches.

Composition of section.

1 cover plate 26" x 3/8"	= 9.75	sq.	in.
4 angles 3" x 3" x 3/8"	= 8.44	"	"
2 web plates 18" x ½"	=18.00	"	"
2 flats 4" x 1"	= 8.00	"	"
Total	44.19	"	"

This composition is required in order to receive the verticals and the heads of the eye-bars.

Design of End Posts.

aB The maximum direct compression in the end post aB is 527310 lbs. Its length is 37.335 feet = 448.02 in. Using the value obtained for BC of r = 7.78 $\frac{L}{r} = \frac{448.02}{7.78} = 57.5$

$$p = \frac{15000}{1 + \frac{1}{13500}\left(\frac{l}{r}\right)^2} = \frac{15000}{1 + \frac{(57.5)^2}{13500}} = \frac{15000}{1.245} =$$

12500 pounds per sq. inch, and the approximate sectional area is $\frac{527310}{12500} = 42.18$ sq. in.

Therefore, the following composition will be used.

1 cover plate 26" x 3/8"	= 9.75 sq. in.
4 angles 3" x 3" x 3/8"	= 8.44 " "
2 web plates 18" x ½"	=18.00 " "
2 flats 4" x 1"	= 8.00 " "
Total	44.19 " "

Pin Plates.

Specification rivets shall not be counter sunk in the plates less than seven-sixteenths of an inch in thickness. We will use pressure on bearing surface as 13500 lbs. per sq. in. (Combined dead and live load.)

Shearing strain on rivets shall not exceed 9000 per sq. in., therefore, will use 7500 pounds per sq.in.

Pin plates shall be used at all pin holes in built members, for the double purpose of reinforcing the metal cut away, and reducing the unit pressure on pin and bearing, to or below the specified limit.

Each plate shall distribute properly, through the rivets, its pressure to the web and flanges of each member.

In riveted tension numbers the net section through any pin hole shall have a net sectional area 40 % in excess of net section of body of member.

The net section outside pin hole and along the line of stress shall be at least 70% of net section through the pin hole.

Panel Point C

Design of pin plates for all members at the Panel Point C.

At the upper panel point for Post Cc, the maximum bearing value on the pin is the full working strength of post, which is 160,000 pounds. Then (from page 323 Cambria) the bearing value of a 1 in. plate on a 5.5 in pin is 74250 lbs.

Therefore 160000 $\div$ 74250 = 2.15 inches, the required thickness of pin plate and webs for post.

Therefore $\frac{1}{2}$ of 2.15 = 1.075 ; web = .52 in. thick. Therefore, 1.075 - .52 = .55 in. of pin plates required for each ~~xxxxxx~~ channel of post. The minimum plates allowed are 7/16" outside for counter sunk rivets, and 3/8" on inside, which is greater than .55" but will be used.

Bearing value for 5.5"pin 1" plate = 67500 lbs., therefore for .813" plate = 54878 lbs., bearing value of a 7/8" rivet in web is .875 x .52 x 13500 = 6142 lbs.

Therefore, $\frac{54878}{6142}$ = 8.9 rivets or 9 rivets.

Use mimimum spacing in pin plates. Two additional rivets are placed above the pin to keep the plates in contact. Pitch should not be less than three diameters of rivet 7/8" x 3 = 2.6 in. say 3 in. is pitch to be used.

Pin Plates C

Pin bearing at point C in upper chord is to be designed to take the horizontal component of the full tensile strength of the diagonal Cd or 195630 x Sin 40° = 125751. Therefore thickness required for $5\frac{1}{2}$ in. pin is $\frac{125751}{74250}$ = 1.70 in. Now thickness of plates is $\frac{1}{2}$ of 1.7 = .85 in. = linear bearing on each side. As the web plate is $\frac{1}{2}$" thick, we will use $\frac{1}{2}$" plate on outside of web.

Bearing value of 1 in. plate of 5.5 in. plate is 74250; therefofe, $\frac{1}{2}$ in. plate the bearing value is 37120 lbs., which is pin plates share of bearing, and this must be transferred to web plate. Bearing value of 7/8" rivet in web is

.875 x $\frac{1}{2}$ x 13500 = 5900 lbs. , which is greater than single shear 4510 for 7/8" rivet. (Unit shear 7.500 lb. sq.in. Cambria, pp.272)

Therefore, 37120 ÷ 4510 = 8+ rivets.

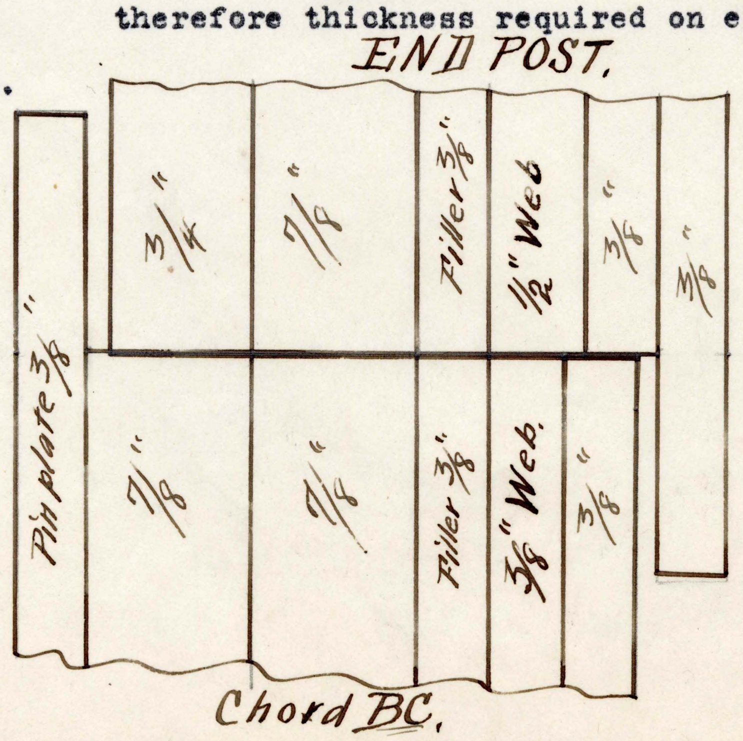
Therefore, $\frac{37120}{5900}$ = 6.3 rivets. It will take seven rivets. Most of them must be placed on the right hand side, but will make other side symmetrical for appearance sake. Use minimum spacing in pin platesabout 3 inches. The head of the eye-bars at this joint are the same thickness as body of bar, and 15.5 in. in diameter.

Hip Joint B DESIGN OF HIP JOINT.

At the hip joint B, the center stress in the upper chord member BC, and that in the end post aB are transferred to the pin; all the plates and shapes, except the hinge or lap plates being faced parallel to the bisecting plane of the angle and about 1/8 inch from it. The hinge plates of each member consist of two plates, located on the inside in one case, and on the outside in the other, and extend past the pin. Their purpose is to prevent any accidental blow from displacing these members, and to facilitate the erection of the truss. The combined pin plates on both members must be arranged with respect to each other, so as to provide a clearance of at least 1/8 inch between them.

CHORD BC For this joint, we will use the limit of a bearing value of 15000 $\frac{\text{lbs.}}{\text{in}^2}$ The full strength of upper chord is 39.69 x 13550 = 537800 lb/in. Full strength required in upper chord is 527100 lbs., the mean is 532500 lbs. Therefore the thickness of pin plates and web combined for a $5\frac{1}{2}$" pin is by Cambria pp. 323, for 1" thickness we findin table 82500, therefore $\frac{532500}{82500}$ = 6.45 in.

therefore thickness required on each ~~inch~~ web is 3.22 in.



$3/8" + 3/8" + 3/8" + 7/8" + 7/8" + 3/8 = \frac{26}{8} = 3.25"$

Chord BC the bearing of plates is as follows:

(1) 5.5 x 15000	=	30900 lbs.
(2)	=	30900 "
(3)	=	72150 "
(4)	=	72150 "
(5) filler	=	30900 "
web	=	30900 "
Total --------		267900 "

Directly from pin bearing web takes 30900 lbs., but web will take 89500 lbs.

Therefore 89500 - 30900 = 58600 lbs., more stress than it gets directly from bearing.

Considering only one side of the members, the division of stresses is as follows:

	Gross area.	stresses.
½ of 3/8" cover plate	4.87 sq.in.	
1 upper angle	2.11 " "	
	6.98 " "	93800 lbs.
1 web plate	6.75 " "	89500 "
1 lower angle	2.11 " "	
1 flat	4.00 " "	
	6.11 " "	82500 "
Total	19.84	265800 "

Plate (5) 3/8" filler is not directly connected to angles, and it will take 5.5" x 3/8" x 15000 = 30900 lbs., which is transferred to web. Therefore, 58600 - 30900 = 17700 lbs., the web will stand yet, and this will be transferred through the extra rivets in webs from other plates, which are always more than the number of rivets here required to transmit this amount to web.

Bearing value of 7/8" rivet in 3/8" web is 7/8" x 3/8" x 13500 = 4350 lbs., which is less than the shear for a 7/8" (4.510) rivet, so will use this.

Plate (1) Bearing value = 4350 lbs.

Therefore $\frac{30900}{4350}$ = 7.1 or 8 rivets, or 4 in shorter angle as its stress must be divided about equally betwean angles.

Plate (2) $\frac{30900}{4350}$ = 7.1 or 6 rivets, as web takes one rivet from each plate or say 3 rivets to angle beyond extremity of plate (1) to make symmetrical.

Plate (3) $\frac{72150}{4350}$ = 16.6 or 17 rivets used, beyond end of plate 2.

Plate (4) $\frac{72150}{4350}$ = 16.6, therefore, will use 14 extra rivets for symmetry beyond extremity of plate (3)

Plate (5) Number of rivets required to carry stresses from the filler plate to web is $\frac{30900}{4350}$ = 8 rivets, but many more than this number must be inserted to keep the plates in contact, and give necessary stiffness in compression. One rivet from plate (2) and three from plate (4) through the web will transmit 4 x 4350 = 17400 lbs., to the web, which is about what the web was able to take (17700), besides the filler.

End Post, Joint B

Full strength of end post is 44.19 x 12500 = 552400 lbs. Full strength required in end post is 527400 lbs., say 540000 (i.e., a mean between the two) and will use this.

Therefore, $\frac{540000}{82500}$ = 6.54 in. Therefore, for each web = 3.27 in.

$3/8" + 3/8" + \frac{1}{2}" + 3/8" + 7/8" + 6/8" = \frac{26}{8} = 3.25$ in.

Distance out to out of web of end post is 19"; therefore, 19" -(2 webs ½" + 4 plates 3/8") = 19"-2.5 = 16.5". Distance out to out of diagonals BC is 16.4"; therefore the diagonals will fit between these pin plates.

Considering only one side of the member, the division of stresses is as follows:

	Gross area		Stresses.
½ of 3/8" cover plate	4.87 sq.in.		
1 upper angle	2.11 " "	6.98	84500
1 web plate	9.00 sq.in.		111500
1 lower angle	2.11		
1 flat	4.00	6.11	74000
Total		22.09	2700000

Stress in half end post is 270000 lbs.

Directly from pin bearing, web plate gets 41250 lbs., therefore web takes 111500 - 41250 = 69250 lbs. more stress than it gets directly from bearing andfiller , will also transfer 30900 to web bearing 69250 - 30900 = 38350 lbs., that the web will still stand, and which will be transferred by the extra rivets in plates through the web.

The bearing of plates are as follows:

(1)	= 5.5" x 3/8" x 15000	=	30900 lbs.	
(2)	= 5.5 + 3/4 x 15000	=	61800	"
(3)	=	=	30900	"
(4)	= 5.5 x 7/8" x 1500	=	72150	"
(5)	= filler	=	30900	"
web 5.5 x ½ x 1500		=	41250	"

Bearing value of 7/8" rivet in ½" web is 7/8" x ½" x 13500, = 5900 lbs.

The shearing value of a 7/8" rivet is 4510 using unit shearing stress as 7500 lbs. per sq. in.

Therefore, when the pin plates are thicker than the web, we will have to use this shearing stress, as it is less than bearing value in ½" plate, but when the pin plates are thinner than the web, we will have to use the bearingvalue of 7/8" web in plate under consideration.

Plate (1) As this is a 3/8" plate, and is thinner than the web, we will use bearing value of rivet for this thickness of plate. This is 7/8" x 3/8" x 13500 = 4350 lbs.

Therefore, $\frac{30900}{4350}$ = 7.1; therefore 8 rivets will be placed, four of these being placed in shorter angle as stress must be equally divided between the angles.

These rivets will also take 4510 (shearing value of 7/8" rivet) - 4350 (bearing value in the 3/8 plate) or 160 lbs. per rivet from another plate, say plate (2); therefore, the total bearing value remaining in these 8 rivets is 8 x 160 = 1280 lbs.

Plate(2) This plate is thicker than web, therefore, will use shearing value for rivets.

Then plate (2) will require $\frac{61800 - 1280}{4510}$ = 13.4 use 14 rivets.

Will use 8 rivets through the angles beyond end of plate (1), and the other 6 through the web.

Now the web will still stand a little more, i.e., 38350 - (6 x 4510) or 11290 lbs.

Plate (3) This plate is same as plate one, and will require 7.1 rivets. Will use six of them in the angles beyond end of plate (2), and one in the web, which leaves 11290 - 4350 or 6940 lbs. of excess strength in web.

These rivets will also take 1280 lbs., pressure from another plate, say plate (4) (See calculation for plate (1).)

Plate(4) This plate is thicker than web, therefore, will have to use shearing value for rivets.

Then plate (4) will require

$\frac{72150 - 1280}{4510}$ = 15 rivets.

Will use ten of these rivets in angles beyond end of plate (3) and the web will take $\left(\frac{11290}{4510}\right)$ or about three of them while the other two can be located at the end of post through the larger angle.

Filler (5) Number of rivets required to carry stress from filler plates to web are 8, but many more than this number will be used to keep all the pin plates in contact with themselves, and with the web, also there must be enough used to give the necessary stiffness in compression.

Diagonal Bc Heads of Eye Bars. At joint B, the connection for the diagonal Bc is two eye bars whose body is 8 in. x $1\frac{1}{2}$ in. The heads on these bars are the same thickness as the body of the bar, and of 17 inches in diameter. (See Cambria pp. 339.)

Suspender Bb, Pin Plates. The net section area of suspender Bb is 147 sq. in., i.e., (2 channels, 12 in. 25. lb, t = 89 in.)

Since this is a tension member , its net sectional area at the pin hole must be 40% in excess of the net area in its main body. The area of each side is, therefore, $14.7 \times \frac{1.40}{2} = 10.29$ sq. in.

Using a pin plate 12" x 7/16" on outside, and 9-1/2" x 3/4" on inside. Net area of these pin plates is 2.62 sq. in. and 2.62 sq. in. Web area of channel pin hole is 7.35 - (39 x 5.5) = 5.20 sq. in., therefore

$5.2 + 2.62 + 2.62 = 10.44$ sq. in., therefore,

tensile strength in web $= \frac{5.2}{10.44}$ of $\frac{136800}{2} = 34000$ lbs.

in 7/16" plate x 12" = 17200 "

" 3/4" " x $9\frac{1}{2}$" = 17200 "

Total ------ ------------= 68400 "

Bearing value of rivet in web of channel is 7/8" x .39 x 13500 = 4607 lbs., which is less than single shearing value of rivet.

Therefore, 3/4" plate will require $\frac{17200}{4607} =$ 3.7 or four rivets to transfer stress to web of channel likewise (7/16" x 12") plate will require $\frac{17200}{4607} = 3.7$

or four rivets; therefore, will make both plates same length, and put 8 rivets through them, which will transmit the stress to the channel, also there will be a few extra put in beyond pin to hold plates and web in contact.

Web section through pin is 10.44 sq. in. Web section outside of pin along center line of stress shall be 70% of this,

Therefore, 10.44 x .7 = 73 sq. in. As the thickness of plates and webs are (7/16"+ 3/4" + .39") = 1.57, we have,

7.3 ÷ 1.57 or 4.65 in. as length beyond the pin hole;

Therefore, $4.65 + \frac{5.5}{2} = 7.4$, or use 7.5 in. from center of pin hole to end of suspender.

Now since allowance was made for two rivet holes in flanges, and also two in web of each channel, only two rivets will be placed in any one section of pin plates after going between the tie plates shown in the detail drawing.

Joint (a) Pin Platesas of aB.

End post at panel point (a), is the same as at panel point large B, but must also take the reaction, or the additional vertical load transferred by the end floor beam (See details on blue print)

The vertical load due to end floor beam is 126.720 lbs. (Unit bearing value 15000 $\frac{lb}{in^2}$ Cambria, pp.323)

From table we have bearing value of 1" plate on 5.5 in pin = 82500, therefore $\frac{126720}{82500}$ = 1.54 in additional. thickness of of pin plates to be added to those of upper end of End post 1.54 divided by 2 = .77 in. or about 3/4" plate to be connected to each web.

Let these plates extend down to the pin, then they would not necessarily need to be riveted to the end post, but it is a better plan to fasten them and about $\frac{126720}{4510x2}$ or fourteen additional rivets will be used. The other pin plates at this joint are arranged a little different from those at upper end of post, and also the flanges of the angles turned off in order to let end post in between the bottom chords.

New arrangement of pin plates.

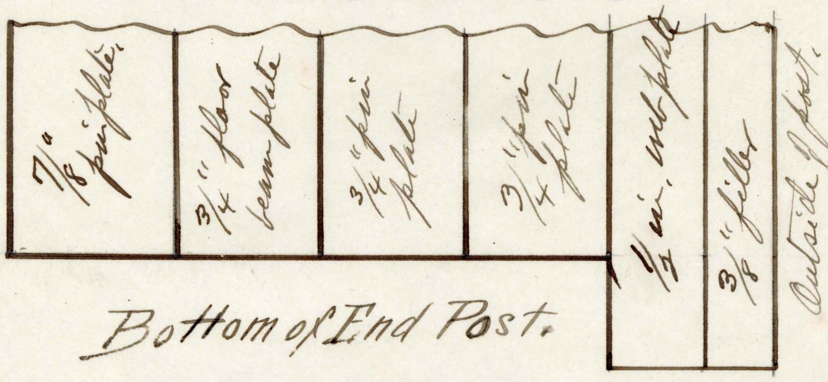


Plate (1) will require sixteen rivets; plates (2), (3) and (4), about fourteen rivets each; about 48 of these rivets will be put through the angles, and the rest through the web. (See blue print for detail)

The floor beam will have to be fastened to these extra pin plates extending up from end post and will require $\frac{126720}{4510}$ or 28 rivets in single shear. (See blue print for connection.)

Joint (a) Pin Plates of ab. — Pin plates at (a) for lower chord are plaeed on outside or web in order to let the end post extend down between the channels. Stress in this chord is 339770lbs., therefore $\frac{339770}{82500}$ = 4" nearly required thickness of plates. 4 divided by 2 =2 in for each half of member. 2" - [½"(web) + 3/8" (filler)] = 1-1/8";

Therefore will use one 5/8" and one ½" pin plate both on outside of web; the ½" plate takes 41250 lbs., of stress, and the 5/8" plate takes 5155 lbs. of stress from the pin.

Therefore, ½ in. pin plate requires $\frac{41250}{4510}$ or ten rivets, while the 5/8" plate requires $\frac{51250}{4570}$, or twelve rivets.

There would also be a pedestal at this joint, but have not designed it.

Joint b

This end of suspender Bb would have the same design as at end B. (For this see joint B).

The lower chord is continuous at this point, and would be reinforced to the amount cut away by a 5.5 in. pin. Use plate ½ in. thick. (See the blue print for details.)

Joint c

The end (c) of the lower chord bc, would be designed exactly the same as the end at panel point (a) (See point a)

There are four eye bars connected to this joint representing the lower chord cd. The body of these bars is 8" x 1-1/8", therefore, the heads of the bars are 1-1/8" thick, and 17" in diameter. (Cambria, pp. 339) See blue print for arrangement)

There are two eye bars connected here from the diagonal Bc; the hads of these are 1½" thick by 17" in diameter.

Pin Plates Vertical Cc at c

The maximum pin bearing at the bottom of the post Cc equals the maximum vertical shear in the diagonal Bc, and according to the rule, the value to be used in designing the pin plates of the post is the vertical component of the full working strength of Bc, which is 24 x 15000/1.3054 = 275700 pounds, the sectional area of Bc being 24 sq. inches, and 1.3054 the secant of the angle which it makes with the vertical. As the diameter of the pin is 5.5 inches, the bearing required on each side of post is 275700/2 x 5.5 x 13500 = 1.225 inches. The thickness of the channel web is .52 inches, and hence two pin plates are required whose thickness are respectively 7/16" and 3/8" If both plates be extended the same distance above the pin the number of rivets required to connect them will be determined entirely by their bearing value in the channel web, or .875 x .52 x 13500 = 6141 pounds for each rivet. This is less than for double shear of rivet, therefore will be used.

Then the full bearing in the two plates is (7/16 + 3/8 = .813) ; 0.81 x 5.5 x 13500 = 54810 pounds.

Therefore, the required number of rivets is then 54.810 divided by 6142 = 9 nearly, therefore will use ten rivets for this plate.

Panel Point d

Pin plates at d on vertical Dd.

The maximum pin bearing at bottom of post Dd equals the maximum vertical shear in diagonal Cd; the value to be used is the vertical component of the full working strength of Cd, which is:

$$\frac{14 \times 15000}{\sec 40°} = \frac{14 \times 15000}{1.3054} = 159000$$

By use of Cambria, we find

$\frac{159000}{42500}$ = 2.1 in. of thickness, which is a little less than that used at panel point C; therefore, we will have to use the minimum thickness of pin plates, i. e. 7/16" on outside, with counter sunk rivet heads and a 3/8" in. plate on inside of web; from calculation at (C), we find that it takes 9 rivets for these plates.

Next, we have 8 eye bars coming into this joint from the two lower chords cd and de. These bars have a body of 8" x 1-1/8", therefore, thickness of heads of each of these eye bars is 1-1/8", while diameter of head is 17" (Cambria, pp.339) Likewise we have 4 eye bars coming into this same joint from the two diagonals Cd and dE. These bars have a body of 7" x 1"

Therefore, thickness of head is 1" and diameter of head is 15½" (Cambria.)

<u>Panel Point D</u>

At this upper panel point the maximum bearing value on the pin is the full working strength of post which is 12.24 x 6558 = 78700 lbs., this is smaller than the stress which gives the minimum sized plates, therefore will use same pin plates as at bottom of this post, a 7/16" outside and 3/8" inside.

The pin bearing at D in upper chord is designed to take the horizontal component of the full tensile strength of De, or $\frac{86550}{1.30544}$ = 66200

A stress of 66200 lbs. does not require quite an inch thickness of plates. Here the two webs of the upper chord are = 1", and therefore no pin plate is needed on account of pin bearing, but the upper chord needs to be reinforced to the amount of web cut away by a 5.5 in. pin; therefore will use a plate ½ in. thick, and about 12 in. wide, which will increase the web by that cut away. The compressive stress in this plate would be about 5.5 x 13500 x ½ or 37125 lbs., therefore use $\frac{37125}{4510}$ = 8 +, i.e., use 8 rivets om each side of the pin hole. Note- (if pin fit closely there would be no need of reinforcing as it would make up for the part cut away in the compression piece).

There are also extending to this panel point two counter bars, whose body is 6" x 1", (the stress in them being 86550 lbs.) therefore, the head of these bars is 1" thick and 15½" in diameter. (Cambria)

Portal bracing.

From previous calculations the panel wind load at top chord is 1.02 tons or 2040 lbs., therefore, the reaction from the wind load at the end of panel is $\frac{2040 \times 4}{2}$ = 4080 lbs., which represents W on portal.

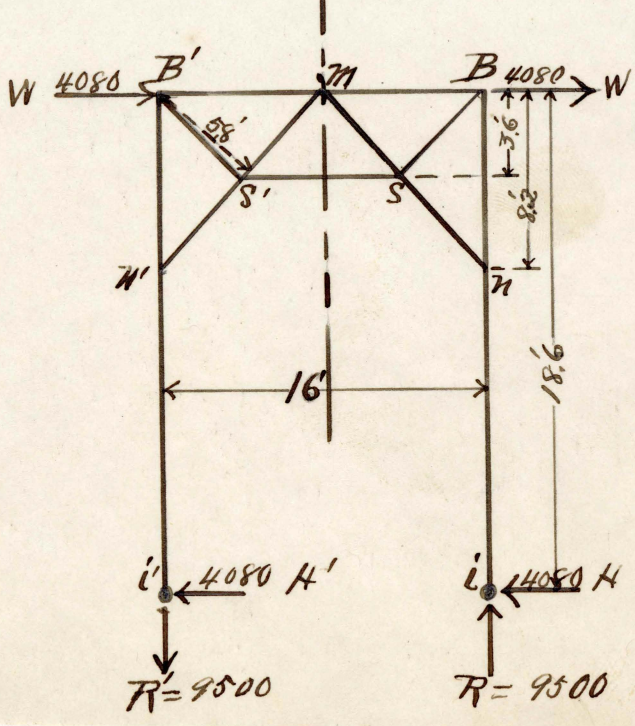


Diagram-

Therefore $W = W^1$ = 4080 lbs.

H = W = 4080 "

Take moments about i' to get R

" $(W+W^1)18.6 - R \times 16 = 0$

" $R = \frac{8160 \times 18.6}{16}$
= 9500 lbs.

" $R = R^1$ = 9500

Since the moment is 0 in the middle section through M of BB^1, there is no stress in S^1 S, and hence there can be no stress in B^1S or B S. The object of insetting these pieces is to hold the diagonal in line and to improve the appearance. Same sections will be used for these as for the diagonals.

Now passing a section cutting the members B^1 and n^1m, and taking moments about B^1, we have for stress n^1m

$$- \text{Stress } n^1m \times 5.8 + H \times 18.6 = 0$$

$$\text{Therefore Stress } n^1m = \frac{4080 \times 18.6}{5.8} = 13000 \text{ lbs.}$$

Likewise Stress n m = 13000 lbs.

Next taking center of moments at n^1, we can find the stress in B^1 m

$$\text{Therefore Stress } B^1m = - \frac{4080 \times 26.8}{8.2} =$$

13350 lbs.

Likewise, Stress B m = + 13350 lbs.

These members must be designed to take compression.

Therefore from Cooper unit strain in comp. members is $P = 13000 - 60\frac{l}{r}$ where $\frac{l}{r}$ shall not exceed 120 for lateral systems. Let $\frac{l}{r}$ = 110 in this case . l for m n = 138 in.

$$\text{Therefore } r = \frac{110}{138} = 0.8$$

P = 13000 - 6600 = 6400 lbs., the unit comp. strain that we will use.

Therefore, Section area of m n =

$\frac{13000}{6400}$ = 2.03 sq. in.

By specifications, no angle shall be less than 3 x 2½ x 3/8 in., and the area of two such angles is 3.86 sq. in.

Deducting two rivet holes 1 sq. in., we have left 2.86 sq.in., which is over the required area.

But the least radius of a struct built up of two pairs of these angles, with one system of lacing between the pairs, the angles being placed 3/4" apart, is 1.63, therefore $\frac{l}{r} = \frac{138}{1.63} = 85$, which is within the limit for $\frac{l}{r}$.

Likewise using the same combination $\frac{l}{r}$ for B^1 m is found to fall with the limit 120.

Now the net strength of the sections making up the portal members is,

4 angles 3 x 2½ x 3/8 = 7.72 sq. in.

deducting for four rivet holes we have

7.72 - 2 = 5.72 sq. in.

Therefore 5.72 x 13500 = 71000 lbs.

Taking bearing value of rivet in 3/8 plate, as 5000 lbs. we have,

$\frac{71000}{5000}$ = 14 rivets to section or 7 rivets to each connection.

A section area of $\frac{6.32}{2}$ or 3.16 in. will be required of each connecting plate.

The lacing to be used between these angles is 2 in. x 3/8 in., with 5/8 in. rivets.

Upper lateral System.

The diagonals in the upper lateral are 37 feet or 444 in. long, but considering them stayed at the middle l would be 222 in., therefore, $\frac{l}{r} = \frac{222}{1.63} = 136$, this does not come in limit of 120 and therefore the angles 3 x 3½ x 3/8 cannot be used.

From tables in Cambria, we find that r = 1.92 for 2 angles 3½ x 2½ x ½ with the 2½ in. flanges 3/4" apart, therefore $\frac{l}{r} = \frac{222}{1.92} = 115$, which falls within the limit, therefore will use this combination.

Net strength of section making up the top lateral 4 angle = 11.0 sq. in. 11 - 2 = 9 in x 13500 = 121500 lbs. $\frac{121500}{6013}$ = 22 rivets (unit shear being 10000 lbs)

Therefore will be 11 rivets to each connection.

Each connection plate in this case will require $\frac{10}{2}$,or 5 sq. in. for each member connected.

The struts in the upper lateral system have various stresses, and therefore will use 4 $\angle$s $4\frac{1}{2}$" x $3\frac{1}{2}$"x$\frac{1}{2}$" latticed.

The bottom Laterals.

For the bottom strut we just use the floor beams.

The diagonal can be riveted to the stringers at the points where they cross them, and this will reduce the value of l; therefore here we will consider l as the longest part of free lateral. Let l = 13 ft. = 156 in.

Here we will use two angles.

$4\frac{1}{2}$" x $4\frac{1}{2}$" x 5/16" placed back to back. For this combination the last radius of gyration is 1.40

$$\frac{l}{r} = \frac{156}{1.40} = 1.11,$$ which falls within the limit of 120, therefore area of this section is 5.44 x 13500 = 73300 lbs., the greatest stress in any of the bottom laterals is 67280 lbs., therefore, this section is sufficient all around for the diagonals, but they must be riveted to the floor stringers where they cross them, and also riveted together in the center at intersection.

$$\frac{73300}{6013} = 12 \text{ rivets at a connection.}$$

The area of connecting plate must be 5.44 sq. in., net for each connecting member.

See the blue print for the remainder of details.

For design of Pins see Stress Sheet.