Design
-of-
Through Railroad Bridge
Pratt Truss.

Respectively submitted as
a thesis to the
Dean of Engineering School
-of-
Washington and Lee University
for
a Bachelor of Science Degree
in Civil Engineering.

Geo. Boyd

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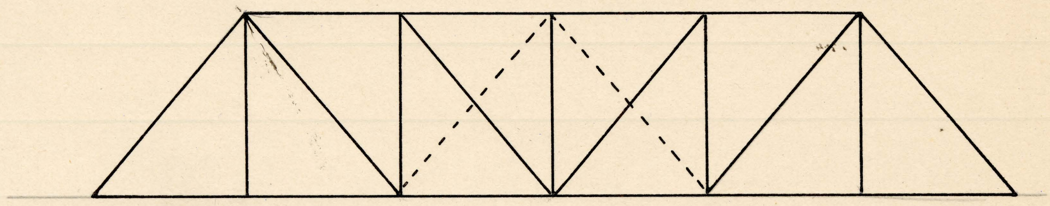
Thesis

Subject Six Panel Through Pratt Truss.



Pin Connected Truss.

Single Track Through Bridge

Pratt Truss

Span 144 feet Panels 6

Depth 28.6 feet Width 16 feet

Dead Load Top 10 Kips
Bottom 20 Kips

Live Load: Cooper's Class E. 50.

Clear distance between trusses 14 feet.

Cooper's Specifications

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

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

Part 1 page 233

Depth

Depth should be about 1/3 span for four panels and about 1/6 span for 12 panels therefore for 6 panels the depth should be about 1/5 panel length or 1/5 of 144 = 28.9 ft.

Part 1 page 233

$h/p = \sqrt{\frac{m+1}{3}} \quad \therefore h/24 = \sqrt{\frac{6+1}{3}} \quad \therefore h = 24\sqrt{2.33}$

$h^2 = 1342. \quad \therefore h = 36.6$ feet 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

The depth may vary 10% from economic depth without affecting the quantity of material 1%

From (I.C.S) Bridge Specifications

The diagonal should not make an angle with the vertical of over 40 degrees; i.e. the diagonal should not make an angle less than 50 degrees with the lower chord, therefore we will take the angle as 40 degrees.

From Cooper's Specifications

Head-room

Clear head room from base of rails is 21 feet for a 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 in lbs per linear

A

feet. $l = 144$ ft.

$\therefore W = 1100 + 7 \times 144 = 1100 + 1008 = 2108 \frac{lb}{ft.}$

From Cooper's Specifications

Track weighs 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.

Part III

B

$W = 600\,l + 9l^2$ 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 $W = 600 \times 144 + 9 \times (144)^2 = 86400 + 9 \times 20736$

" $W = 86400 + 186624 = 273024$ lbs equal weight of bridge

Therefore weight of truss $= 136512$ lbs.

" " " one panel $= 136512 \div 6 = 22752$ lbs $= 22.751$ Kips.

Weight of track $= \frac{440}{2} = 220$ lbs per linear foot, therefore for one panel of truss equal $220 \times 24 = 5280$ lbs $= 5.28$ Kips

Dead panel load $= 22.75 + 5.28 = 28.03$ Kips

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 $2548 \div 2 = 1274$ lbs per linear foot.

Therefore Dead panel load $= 1274 \times 24 = 30570$ lbs $= 30.57$ Kips.

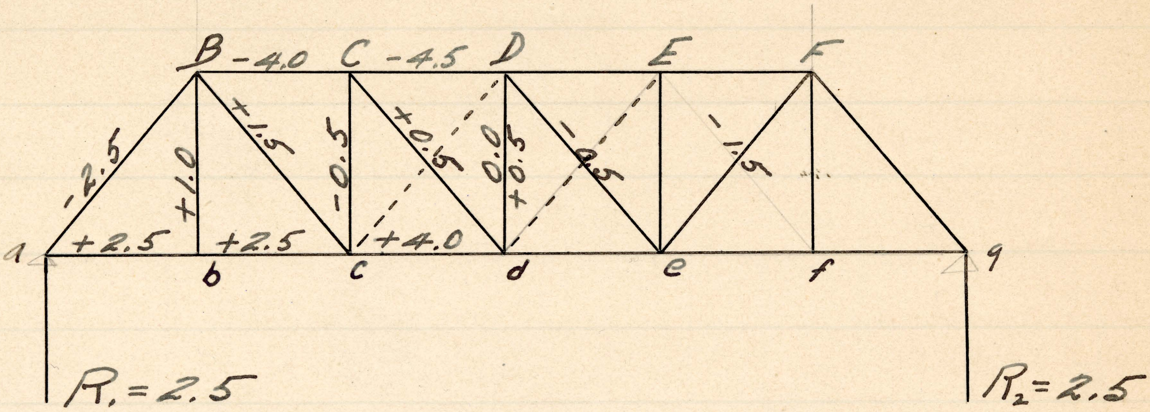
We have from A the Dead Panel Load to be equal 30.57 Kips

From B we have the dead Panel Load to be equal 28.03 Kips.

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 2/3 to lower panel point (Part III. p306)

Dead Load = $\left\{ \begin{array}{l} \text{Top 10 Kips} \\ \text{Bottom 20 Kips.} \end{array} \right\}$ 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 5 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.

Stresses — To find stresses in upper and lower chords multiply factor numbers by panel loads and then by tangent θ

Stresses in chords

$S_{BC} = -4.0 \times 30 \times .8391 = -100.69$ Kips

$S_{CD} = -4.5 \times 30 \times .8391 = -113.28$ "

$S_{ab} = +2.5 \times 30 \times .8391 = +62.93$ "

$S_{bc} = S_{ab} = +62.93$ "

$S_{cd} = +4.0 \times 30 \times .8391 = +100.69$ "

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

$S_{Bb} = +1.0 \times 30 - 10 = +20$ Kips

$S_{Cc} = -0.5 \times 30 - 10 = -25$ "

$S_{Dd} = 0.0 \times 30 - 10 = -10$ "

$S_{Dd} = +0.5 \times 30 - 10 = +5$ "

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

Stresses in diagonals

$S_{aB} = -2.5 \times 30 \times 1.3054 = -97.91$ Kips End Post

$S_{Bc} = +1.5 \times 30 \times 1.3054 = +58.74$ "

$S_{Cd} = +0.5 \times 30 \times 1.3054 = +19.58$ "

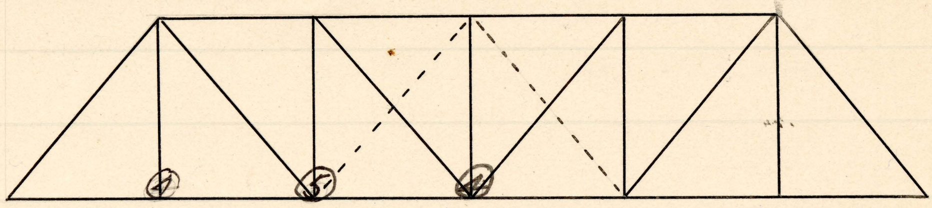
$S_{De} = -0.5 \times 30 \times 1.3054 = -19.58$ "

$S_{Ef} = -1.5 \times 30 \times 1.3054 = -58.74$ "

Counters

$S_{Fg} = -2.5 \times 30 \times 1.3054 = -97.91$ " End Post

Stresses Due to Live Load.



To find stresses due to Live load in diagonals

Diagonal aB

For first panel i.e. S_{aB} try wheel 4 at b

From tables $l = 120 + 18 = 138$ feet length of train on bridge

Therefore $W = 420 + 3 \times 2.5 = 427.5$ Kips weight of train on bridge

$P' = \frac{1}{m} W = \frac{427.6}{6} = 71.2$ $\left\{ \begin{array}{l} \text{wheel } 3 = 62.5 \\ \text{"} \quad 4 = 87.5 \end{array} \right\}$ Correct loaded

$R = \frac{M}{l}$ $\quad r_b = \frac{M_4}{p}$ $\quad V = R - r_b$ $\quad S = V \sec \theta$

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

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

$= 30530 + 1260 + 11.25 = 31801.25$

$R = \frac{31801.25}{144} = 220.84$ $\quad r_b = \frac{600}{24} = 25$ reaction at left panel point due to panel load.

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

$S_{aB} = 195.8 \times 1.3054 = -255.6$ Kips Compression piece

Diagonal Bc

Try wheel 3 at c

From table $l_t = 24 \times 4 + 13 = 109$ feet length of train.

$W = 355$ kips weight of train on bridge.

For Shear $P = \frac{1}{m} W = \frac{355}{6} = 59.13$ $\begin{cases} ② = 37.5 \\ ③ = 62.5 \end{cases}$

$R = \frac{M}{l} = \frac{M_1 + V_1 x + \frac{wx^2}{2}}{l} = \frac{20455 + 355 \times 0 + 0}{144}$

$= 142.05$

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

$V = 142.05 - 11.98 = 130.07 = 130.1$

$S_{Bc} = 130.1 \times 1.3054 = +169.8$ kips Tension piece.

Diagonal Cd

Try wheel 3 at d

From table $l_t = 85$ feet (Engines only)

$W = 290$ Kips

$P = \frac{290}{6} = 48.3$ $\begin{cases} \text{wheel ②} = 37.5 \\ \text{,,} \quad ③ = 62.5 \end{cases}$ Correctly Loaded

$R = \frac{M}{l} = \frac{M_1 + V_1 x + \frac{wx^2}{2}}{l} = \frac{10910 + 290 \times 6}{144} = \frac{12650}{144}$

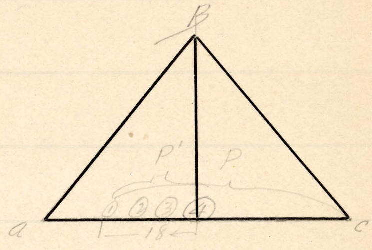
$= 87.82$ $r_d = \frac{287.5}{24} = 11.98$

Therefore $V = 87.82 - 11.98 = 75.84$

$S_{Cd} = 75.84 \times 1.3054 = +98.95$ kips tension piece.

Live Load Stresses in Verticals.

Vertical Bb



$l = 18 + 24 = 42$ feet

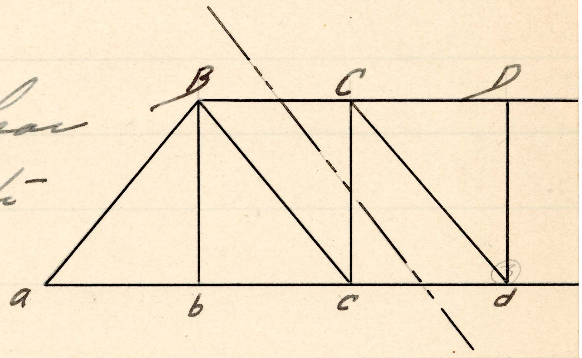
$P = 145$

$P' = \frac{145}{2} = 72.5 \begin{cases} \text{wheel } ③ = 62.5 \\ \text{''} \quad ④ = 87.5 \end{cases}$

$$v_b = \frac{M_c - 2M_b}{p} = \frac{2693.75 - 600 \times 2}{24} = 62.24 \text{ Kips}$$

$S_{Bb} = +62.24$ 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

$l = 24 \times 3 + 13 = 85$ ft

$W = 290$

Therefore $P = \frac{290}{6} = 48.3 \begin{cases} \text{wheel } ② = 37.5 \\ \text{''} \quad ③ = 62.5 \end{cases}$ Correctly loaded

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

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

Therefore $V = 87.82 - 11.98 = 75.84$

" $S_{Cc} = -75.84$ kips.

Vertical Dd

try wheel ② at e

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

$W = 190$ Kips (weight of engines)

$P = 1/6 \times 190 = 31.7$ $\begin{cases} \text{wheel ① } = 12.5 \\ \text{" ② } = 37.5 \end{cases}$ 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

$\therefore S_{\square d} = -36.1$ Kips Compression piece.

Live Load Stresses in Upper Chords.

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

(2) Find length of train on bridge $= l_t$

(3) " weight of train " " $= W_t$

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

(5) Find reaction at left support.

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

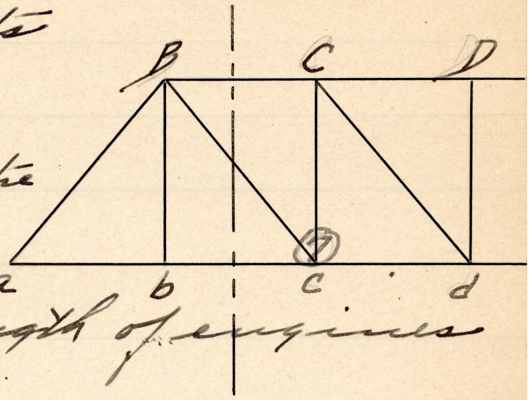
(b) $R = \frac{M}{l}$

(6) Find bending moment $M_b = R(n' \times p) - M_{\oplus}$ (n' = number of panels and $M_{\oplus}$ = moment at wheel found from table 41a)

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

Chord BC

Take centre of moments at c.



Try wheel 7 at c, the centre of moments.

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

Therefore $W_c = 407.5 + 2.5 \times 3 = 415$

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

$P = \frac{2}{6} \times 415 = 138.3$ $\left\{ \begin{array}{l} \text{wheel } 6 = 128.75 \\ \quad " \quad 7 = 145.00 \end{array} \right\}$ Correctly loaded

$M = M' + V'x + \frac{wx^2}{2} = 28461.25 + 407.5 \times 3 + \frac{2.5 \times 9}{2}$

$= 28461.25 + 1222.5 + 11.25 = 29695$

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

$M_c = R \times pn' - M_7$

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

$S_{BC} = \frac{M_c}{d} = \frac{7204.33}{28.6} = -251.9$ Kips.

Compression piece.

Chord CD

Take centre 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

$P = \frac{3}{6} \times 422.5 = 211.25$ $\left\{ \begin{array}{l} \text{wheel } 10 = 190 \\ \quad " \quad 11 = 215 \end{array} \right\}$ Correctly Loaded

$M = 30530 + 420 \times 1 + 2.5 \times (1)^2/2 = 30951.25$

$R = \frac{30951.25}{144} = 214.93$

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

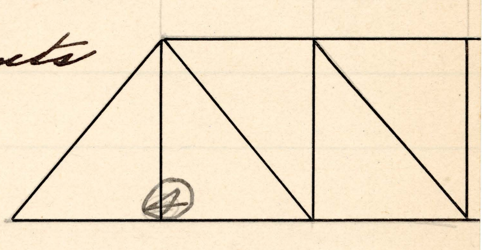
$S_{CD} = \frac{8164.96}{28.6} = -285.4$ Kips Compression piece.

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

$P = \frac{n'}{m} W = \frac{1}{6} \times 427.5 = 71.25$ $\left\{ \begin{array}{l} \text{wheel ③} = 62.5 \\ \text{wheel ④} = 87.5 \end{array} \right\}$ correctly Loaded

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

$= 31801.25$

$R = \frac{31801.25}{144} = 220.84$

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

Therefore $S_{bc} = \frac{4700.16}{28.6} = +164.34$ Kips tension piece.

Stress in ab = Stress bc

Therefore $S_{ab} = +164.34$

Chord cd

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

$S_{cd} = +251.9$ Kips (tension piece.)

Impact

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

Impact in diagonals

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

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

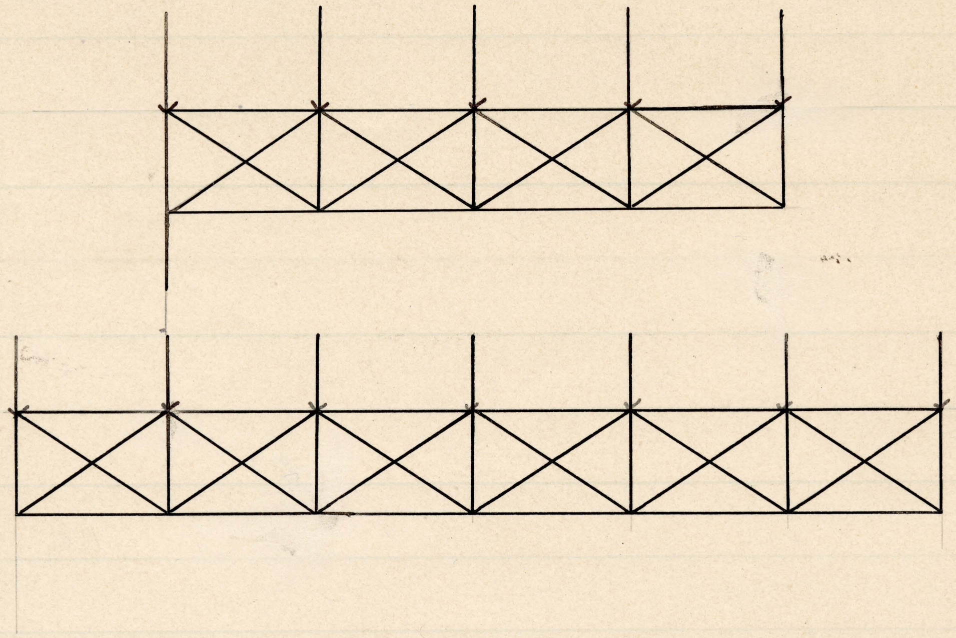
$I_{CD} = -285.4\left(\frac{300}{136+300}\right) = -196.3$

Impact in Lower Chords

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

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

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

$144 + 96 + (8 \times 37.34) + (5 \times 28.6) = 681.7$ foot.

$681.7 \times 1 = 681.7$ sq. feet.

$681.7 \times 30 = 20451.0$ lbs. $= 10.225$ tons

Number of panels $= 6 + 4 = 10$

Therefore panel wind load $=$

$10.225 \div 10 = 1.02$ tons

Stress in Top Laterals

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 laterals for a fixed horizontal force plus a moving horizontal force of 440 lbs per linear foot.

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

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

Top Lateral Stresses

Shear in Panels

$BC = 2040 \times 1\tfrac{1}{2} = 3060$ 1st.

$CD = 2040 \times \tfrac{1}{2} = 1020$ 2nd

Stresses in Diagonals

$3060 \times \frac{37.335}{17.0} = 2.2 = -6732$ 1st

$1020 \times 2.2 = -2244$ 2nd.

Bottom Lateral Stresses.

Shear in Panels

$ab = (2040 \times 2\tfrac{1}{2}) + (10560 \times \tfrac{15}{6}) = 30582$

$bc = (2040 \times 1\tfrac{1}{2}) + (10560 \times \tfrac{10}{6}) = 20660$

$cd = (2040 \times \tfrac{1}{2}) + (10560 \times \tfrac{6}{6}) = 11580$

Stress in diagonals

1st $30.582 \times 2.2 = -67.280$ Kips $\frac{37.335}{17.0} = 2.2$

2nd $20.660 \times 2.2 = -45.452$ "

3rd $11.580 \times 2.2 = -25.476$ "

Stresses in Portal Strut

There are $2\frac{1}{2}$ panel loads of wind force applied at the top of portal strut $= 2040 \times 2\frac{1}{2} = 5100$ 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 strut. 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 $= 5100 \times \frac{1}{2} = 2550$ pounds, and the bending moments at the knee-connections due to these forces $= 2550 \times 14.5 = 36975$. These moments are resisted by forces at the top of post acting with lever arms of 8.2 feet which forces = 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 the stress in knee brace is equal to the horizontal force at its foot multiplied by its length and divided by one-half the width of portal = $7059 \times \frac{11.5}{8}$ = 10147.3 pounds. This stress will be tension on the windward 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 × 24 × 1/2 = 5280

Live Load = $\frac{25000(9+14+19+24)}{24}$ = 68750

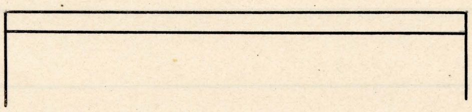
Impact $\frac{(68750)^2}{68750+5280}$ = 63750

13778 0 lbs.

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

137780 ÷ 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 $Rl = M \;\therefore\; R = \frac{M}{l}$

$M = M_5 + V_5 x + \frac{w x^2}{2}$

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

$= 1075.0$

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

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

M_B = 356600 lbs feet.

Moment

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

Live Load = 44800 × 10.75 − (23000 × 5 = 366600

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

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 ÷ 16000 = 14.16 sq. in.

Then 1/8 of 42 × 3/8 in. web plate = 1.94

2 angles 6 × 6 × 3/8 in = 13.19

sq. in. net. 15.13 (one hole 1 in. diameter in each angle.

Intermediate Floor beams

The effective length of floorbeam is assumed to 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 floorbeam is assumed to be 3000 lbs., which is a distributed load. The dead load concentrations from stringers = 440 × 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' in the panel ab varies from 87.5 to 112.5 kips and 2P' from 175 to 225 kips. The total load P on both panels varies from 161.25 to 177.5 thus satisfying the criterion $P = 2P'$

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

End Shear

Dead Load	$= (3000 \times 1/2) + 10560 =$	12060
Live Load	$=$ $=$	95620
Impact	$= \dfrac{(95620)^2}{95620 + 12060} =$	85100
		192780 lb.

Area required in web plate $= 192780 \div 10000 = 19.28$ sq. in.

A $54 \times 3/8$ in web plate $= 20.25$ sq in we will use this.

Moment

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

Live Load = 95620×4.75 = 454000

Impact $\frac{(454000)^2}{454000 + 56160}$ = <u>403000</u>

ft lbs 913160

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

Flange stress = 913160 ÷ 4.25 = 214800 lbs.

Flange area required = 214800 ÷ 16000 = 13.43 sq. in.

Then 1/8 of 54 × 3/8 in. web plate = 2.53

2 angles 6 × 6 × 3/8 in = <u>11.62</u>

sq. in. net. 14.15.

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

End Floorbeams.

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

The weight of floorbeam = 3000 lbs.

Dead load concentractions from stringers = 440 × 12 = 5280 pounds.

Live load concentraction from

stringers as determined in connection with stringers = 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 = $(3000 \times 1/2) + 5280 =$ 6780

Live Load = = 77080

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

154710

Area required in web plate =

$154710 \div 10000 = 15.47$ sq. in.

A $54 \times 3/8$ in. web plate = 20.25 sq. in.

We will use this

Moment

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

Live Load = 77080×4.75 = 366100

Impact = $\frac{(366100)^2}{366100 + 31080}$ = 337800

734980

Assuming a effective depth of floorbeam = 4.33 feet. Flange stress = $734980 \div 4.33 = 169500$ pounds.

Flange area = $169500 \div 16000 = 10.05$ sq. in.

Then

$1/8$ of $54 \times 3/8$ in web plate = 2,53 sq. in.
2 angles $6 \times 3\frac{1}{2} \times 9/16$ in = 8,49 (1 hole 1 in. diameter in each angle) 11,07 sq. in net.

The connection angles for stringers or floor beams shall have no leg less than $3\frac{1}{2}$ inches or be of less thickness than $\frac{1}{2}$ inch.

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 vibrations. Therefore its composition will be made 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 net section area
$= \frac{136740}{15000} = 9.2\square''$ approx

2 channels 12"; 25 lb $= 14.70$ sq. in $t = .39$
4 rivets in web $4 \times .39 \times 7/8 = 1.36$
4 " " flange $4 \times .50 \times 7/8 = \underline{1.76}$
3.12

Therefore a net 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 to affect the result according to the specified unit stresses, the total stress to be considered is 159940 pounds. A trial shows that 15-unit 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
$\frac{L}{r} = 5.25 \quad = 40967$
Therefore to obtain safe unit stress for moving loads as in bridges divide 40967 by 5 = 8193.4

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

required. Area of two 7/8 rivet holes in web = 2×7/8×.52 = .91 sq. in
Area of 4 flange rivet holes = 4×7/8×.65 = 2.28 sq. in. Total area to be deducted for rivet holes is .91 + 2.28 = 3.19 sq in
Gross area of channel = 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 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 tables in Cambria = 32790 and the safe unit stress = $\frac{32790}{5} = 6558$
$\frac{76500}{6558} = 11.7^{\square}$ net area required to stand this stress.
Now 2 (10 in; 25 lbs) beams have sections area = $7.35 \times 2 = 14.7$ sq. in.
Deducting area of 4 rivet holes as this is as many as can come in one section =
$(\frac{.63 + .24}{2} + .24) \times 7/8 \times 4 = 1.52$ sq. in

also one rivet for each web =
$.53 \times 7/8 \times 2 = .94$
Now $14.7 - (1.52 + .94) = 12.24$ sq. in. net 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 for all verticals in order to make the floor beams the same length.

Design of Diagonals

Bc Since the minimum 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. inch, the sectional area must be $353040/15000 = 23.53$ sq. inches. Two eye bars $8'' \times 1\frac{1}{2}''$ 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

30

in the limits of these.

Cd Since the minimum stress in Cd is tension of 195630 lbs it may be composed of one or more eye bars. For a unit tensile stress of 15000 pounds per sq. in. the sectional area must be 195630/15000 = 13.04 sq. in = required net area.
Take two eye bars 7" × 1" = 14 sq. in. therefore will be used. See page 339 Cambria Head of 15½ inches with 5½ in pin hole.

Dc Counter
The areas 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 1 eye-bar 6" × 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 = 22.65 sq. 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 hand-book, 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 2 web plates $18'' \times 1/2'' = 18$ sq. in. 4 angles $3'' \times 3'' \times 3/8 =$ 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 3 rivet holes in each web plate and one in each angle, giving a net 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 net area, where 527090 lbs is the stress in cd.

Take four eye-bars $8'' \times 1\,1/8'' = 36$ sq. in. Therefore we

The specification also require that, if the unit stress due to the weight of a member is greater than 10 per cent 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

$$S_1 = \frac{Mc}{I + \frac{nPl^2}{mE}}$$

where $P = 169885$ lbs $l = 288$ in
$E = 29000000 \frac{lb}{in^2}$ medium steel
$\frac{m}{n} = 96 \quad \frac{n}{m} = \frac{1}{9.6}$

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

$$\therefore S_1 = \frac{158760}{114.6} = 1387$$

1/10 per cent of $\frac{15000}{1} = 1500 \therefore 1387$ 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

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

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

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

$$p = \frac{15000}{1 + \frac{1}{13500}\left(\frac{l}{r}\right)^2} = \frac{15000}{1 + \frac{1}{13500}(37.02)^2} = 13550$$

Total stress in CD = 594980 lbs.
∴ Section area = $\frac{594980}{13550}$ = 43.91 sq. in net area required.
Width of cover plate = 18.5" + 6" + 1" = 25.5"
so will use cover plate = 26 in.

Composition of section

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

is all that will be required for all bars of same depth and length.

Upper chords Design

BC 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 axis equal to four-tenths of the depth out to out. This depth is estimated to be 19.44 inches, making $r = 7.78$ inches, $\frac{l}{r} = 37.02$

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

per sq. in, 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.

In riveting these together we will use the minimum spacing of rivets for 2 feet on each side of joint increasing spacing toward the middle but never greater than 6 inches

Design of End Post.

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.

∴ The following composition will be used

1 cover plate $26'' \times 3/8''$	= 9.75 sq. in
4 angles $3'' \times 3'' \times 3/8''$	= 8.44 " "
2 web plate $18'' \times 1/2''$	= 18.00 " "
2 flats $4'' \times 1''$	= 8.00 " "
Total	44.19 " "

Pin Plates

Panel point 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 $24 \times 15000 / 1.3054 = 275700$ 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 $275700 / 2 \times 5.5 \times 13500 = 1.225$ 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" of an inch. If both plates be extended the same number
distance above the pin the

of rivets required to connect these will be determined entirely by their bearing value in the channel web, or .705 × .52 × 13500 = 50130 pounds for each rivet. Then the full bearing value however is 0.813 × 5.5 × 13500 = 54810 pounds. The number of rivets required is then

$$\frac{54810}{50130} = 11$$

Panel point C

At the upper panel point take the bearing value of pin plates as 13500 lbs per sq. inch see Cambria pp. 323. Stress in vertical is 15994 say 16000 lbs. Then from pp. 323 the bearing value of a 5.5 in pin in a 1" plate is 74250

$$\therefore \frac{160000}{74250} = 2.15 \text{ in of thickness of}$$

plates and web. .52 thickness of web.

1.075 − .52 = .55 in. Hence .55 is thickness of two pin plates required, but by specifications the outer plate cannot be less than 7/16" for counter sunk rivets and the ...

to be 3/8 in thick which gives 13/16" or .813" which is larger than .55" A 7/16" plate will be used outside and 3/8" used inside of web.

Bearing value for 5.5" pin 1" plate = 67500 lbs ∴ for .813" plate = 54878 lbs bearing value of a 7/8" rivet in web is

$.875 \times .52 \times 13500 = 6142$ lbs.

$\therefore \frac{54878}{6142} = 8.9$ rivets or 9 rivets

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

Pin plates C

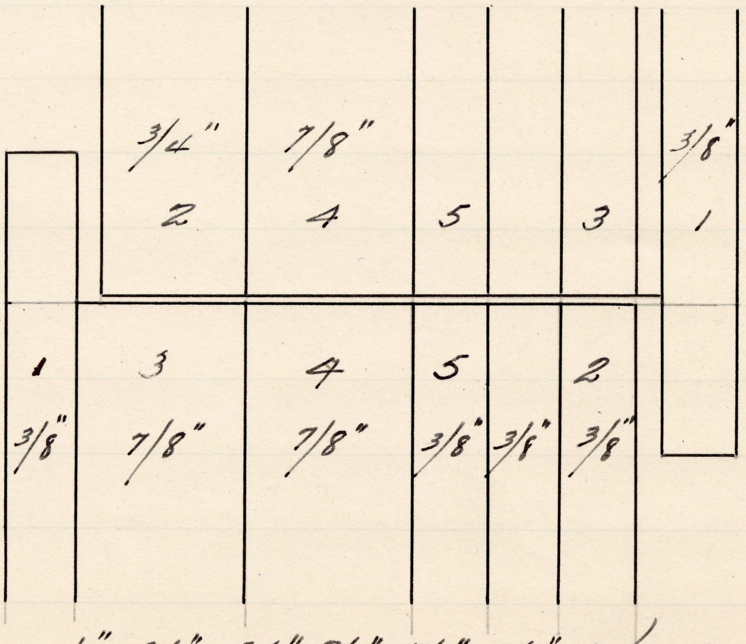
Pin bearing at point C in upper chord is to be designed to take the horizontal component of the full tensile strength of the diagonal Cd or $195630 \times \sin 40° = 125751$ Therefore thickness required for 5½ in

pin is $\frac{125751}{74250}$ = 1.70 in. Now
thickness of plates is 1/2 of 1.7 =
.85 in = linear bearing on
each side. As the net
plate is 1/2 in we will use
one 1/2" plate on outside
of net.
Bearing value of 1 in plate
of 5.5 in. pin is 74250
∴ 1/2 in plate the bearing value
is 37120 lbs which is pin
plates share of bearing and
this must be transferred to
net plate. Bearing value
of 7/8" rivet in net is
$.875 \times 1/2 \times 13500 = 5900$ lbs.
∴ $\frac{37120}{5900} = 6.3$ rivets It will
take 7 rivets. Most of these
must be placed on the
right hand side but will
make other side symmetrical
for appearance sake. Use
minimum spacing in pin
plates about 3 inches.

Hip joint B

At the hip joint B the entire 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.

For this joint we will use the limit of a bearing value of 15000 $\frac{lb}{in^2}$. The full strength of upper chord is $39.69 \times 13550 = 537800$ 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 find in table 82500 $\therefore \frac{532500}{82500} = 6.45$ in.
$\therefore$ thickness required on each web is 3.22 in



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

Chord BC the bearing of plates

is as follows.

(1) $5.5 \times 3/8 \times 15000 = 30900$ lbs

(2) $= 30900$ "

(3) $= 72150$ "

(4) $= 72150$ "

(5) filler $= 30900$ "

web. $= 30900$ "

Total 267900 "

Directly from pin bearing web takes 30900 lbs. ∴ Web will take 89500 lbs.

∴ $89500 - 30900 = 58600$ lbs more stress than it gets directly from bearing.

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

	Gross Area.	Stresses
1/2 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 $5.5" \times 3/8" \times 15000 = 30900$ 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 web ~~from~~ other plates which ~~is~~ 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 $7/8" \times 3/8" \times 13500 = 4350$ lbs. which is less than the shear for a 7/8" 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 between bearing value left over as web is one of the thinnest plates.

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.

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

Plate (5) No 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.

End Post Joint B.

Full strength of end post is $44.19 \times 12500 = 552400$ lb. Full strength required in end post is 527400 lb say 540000 (i.e. a mean)

$\therefore \frac{540000}{82500} = 6.54$ in.^{2} $\therefore$ for each web $= 3.27$ in

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

Distance out to out of web of end post is 19"

$\therefore$ 19" − (2 webs 1/2" + 4 plates 3/8") = 19" − 2.5" = 16.5"

Distance out to out of diagonal BC is 16.4"

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

	Gross Area	Stresses
1/2 of 3/8" cover plate	4.87 sq in	
1 upper angle	2.11 "	
	6.98	84500
1 web plate	9.00 "	111500
1 lower angle	2.11 "	
1 flat	4.00 "	
	6.11	74000
Total	22.09	270000

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 filler will also transfer 30900 to web leaving 69250 - 30900 = 38350^{lb} 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" × 3/8" × 15000	= 30900 lbs.
(2) = 5.5 × 3/4" × 15000	= 61800 "
(3) =	= 30900 "
(4) = 5.5 × 7/8" × 1500	= 72150 "
(5) = filler	= 30900 "
web 5.5 × 1/2 × 1500	= 41250 "
Total	267900 "

Bearing value of 7/8" rivet in 1/2" web is 7/8" × 1/2" × 13500 = 5900 lbs which is less than the shear for a 7/8" rivet so will use it

Plate (1)

Bearing value of 7/8" rivet in 3/8" plate is 7/8 × 3/8 × 13500 = 4350 lbs and will be used here as plate is thinner than web.

∴ $\frac{30900}{4350}$ = 7.1 ∴ 8 rivets will be used 4 being in shorter angle as stresses must be divided equally between angles. These rivets will also take 5900-4350 or 1550 lbs per rivet in plate ② ∴ Total bearing value due to these in plate ② is 1550 × 8 = 12400 lbs.

Plate ② ∴ plate ② will require
$\frac{61800-12400}{5900} = 8.4$ rivets or 9 rivets 4 to the angle beyond the extremity of first plate. All the bearing value of the rivet is used up here.

Plate (3) $\frac{30900}{4350} = 7.1$ or 6 rivets or 3 rivets to the angle will be required beyond the extremity of plate ② taking off one rivet for web. These rivets will also take 5900-4350 or 1550 lbs per rivet in plate ④

1550 × 8 = 12400 lbs.

Plate ④ ∴ $\frac{72150-12400}{5900}$ = 10 rivets will be used beyond end of plate ③ i.e. 5 rivets to angle.

Filler (5) No of rivets required to carry stress from filler plate to web is 8 but many more than this number will be used to keep plate in contact and to give necessary stiffness in compression.

Diagonal at joint B

8 in bar used as a diagonal Bc connected to a 5.5 in pin require a head of 17" in diameter See Cambria pp. 339.

Pin Plate Bb Net section area of 2 channels 12 in. 25 lb. = 14.7 in $t = .39$ in. Since suspender Bb is a tension member its net sectional area at the pin hole must be 40 per cent 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" × 7/16" on out side and 9½" × 3/4" on inside. Net area of these pin plates are 2.62 sq. in and 2.62 sq. in. Net area of channel

pin hole is $7.35 - (.39 \times 5.5) = 5.20$ 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 $\times 12" = 17200$ "
" 3/4" " $\times 9\frac{1}{2}" = \underline{17200}$ "
Total 68400 "
Bearing value of rivet in web of channel is $7/8" \times .39 \times 13500 =$ 4607 lbs. which is less than single shearing value of rivet.
$\therefore$ 3/4" $\times 9\frac{1}{2}$" plate will require $\frac{17200}{4607} =$ 3.7 or 4 rivets to transfer stress to web of channel & likewise (7/16 $\times$ 12") plate will require $\frac{17200}{4607} = 3.7$ or 4 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.
Net section through pin is 10.44 sq. in. Net section outside of pin along center line

of stress shall be 70% of this ∴ $10.44 \times .70 = 7.3$ sq. in. Now thickness of plates and web is $7/16'' + 3/4'' + .39'' = 1.57''$
$7.3 \div 1.57 = 4.65$ in. ∴ $4.65 + \frac{5.5}{2} = 7.4$ in from center of pin hole to end of suspender say $7\frac{1}{2}$ in.

Now since allowance was made for 2 rivet holes in flanges and 2 rivet holes in web of each channel, only two rivets will be placed in any one section of pin plates after coming below tie plates shown on sides of member.

Portal—

Net strength of sections making up the portal is 4 angles $3'' \times 2\frac{1}{2}'' \times 3/8'' = 7.72$ sq. in deducting for two rivet holes we have

$7.72 - 1.4 = 6.32$ sq. in.

$6.32 \times 13500 = 86500$ lbs.

hence connections will require $\frac{86500}{6013} = 14$ rivets to section

Connection plates at portal will require a working sectional area of 6.32 sq. in. to transfer net strength of portal bracing.

Net strength of sections making up the top laterals 2 angles $3\frac{1}{2}" \times 2\frac{1}{2}" \times \frac{1}{2}"$ Area $= 5.5 \times 2 = 11$ sq. in

deduct for 2 rivets $\frac{7}{8}" \times \frac{1}{2}" = .9$ or 1 sq. in.

$\therefore 11.0 - 1.0 = 10.0$ sq. in.

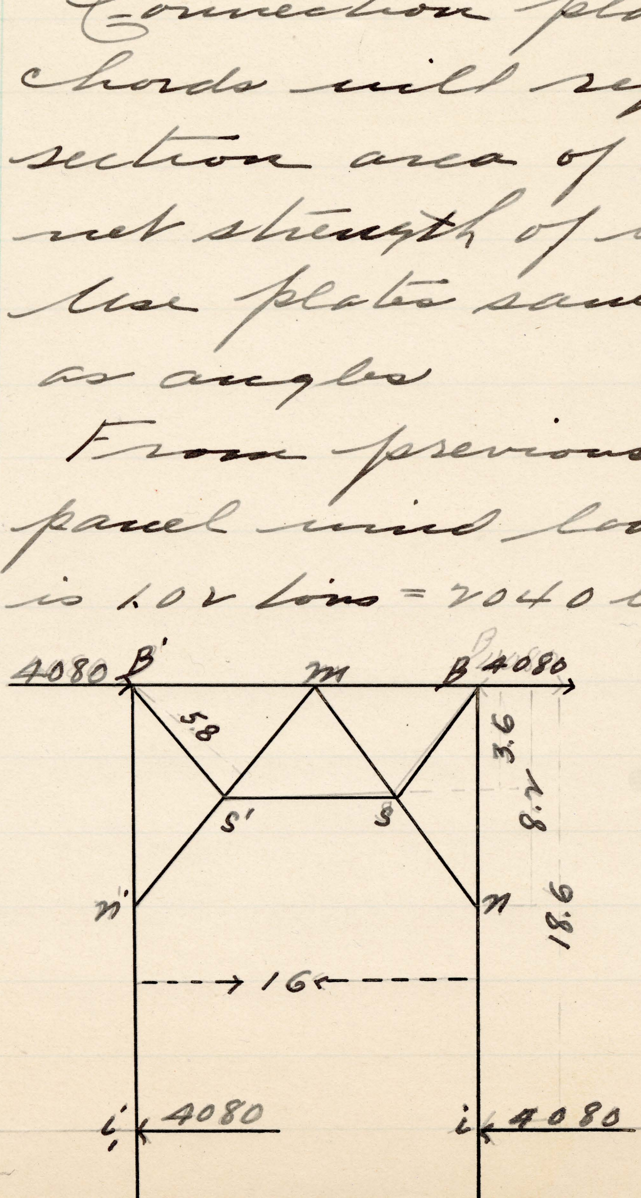
$\therefore$ Net strength $= 10.0 \times 13500 = 135000$ lbs.

$\frac{135000}{6013} = 22$ rivets 11 rivets each side of chord.

Connection plates on top chords will require a working section area of 10 sq. in. to transfer net strength of laterals.

Use plates same thickness as angles

From previous calculation panel wind load at top chord is 1.02 tons $= 2040$ lbs. $\therefore R = \frac{2040 \times 4}{2} = 4080$ lbs.



$w = 4080$ lbs. $= W'$

$H = W = 4080$ lbs.

Moments about i; to get R

$(w + W') \times 18.6 - R \times 16 = 0$

$$\therefore R = \frac{8160 \times 18.6}{16} = 9500 \text{ approx.}$$

$R = R_1 = 9500$ lbs.

Since the moment is zero in the middle section through ~~in~~ there is no stress in S'S and hence there can be no stress in B'S' or BS. The object of inserting them is to hold the diagonals in line and improve the appearance. Same sections will be used for these as for the diagonals.

Passing section cutting B'm and n'm and taking moments about B' we have

$$-S_{n'm} \times (\perp \text{ distance to } mn') + H \times 18.6 = 0$$

$$S_{n'm} = \frac{4080 \times 18.6}{5.8} = 13000 \text{ lbs.}$$

Taking moments about n' then

$$S_{B'm} \times 8.2 + 4080(18.6 + 8.2) = 0$$

$$S_{B'm} = -\frac{4080 \times 26.8}{8.2} = 13350 \text{ lbs.}$$

No angles less than 3" x 2½" shall be used for bracing

From Cooper's specification ~~P. 3000~~

$P = 13000 - 60\frac{l}{r}$ - dead load and 2/3 of this for live load.

$\frac{l}{r}$ shall not exceed 120 for laterals. Let $\frac{l}{r} = 100$ in this case. length mn = 11.5 feet = 138 in.

then $r = \frac{100}{l} = \frac{100}{138} = 0.725$

$\therefore P = 13000 - 60 \times 100 = 7000$ lbs.

Stress in mn = 13000 lbs.

$\therefore$ Section area of mn $= \frac{13000}{7000} = 2$ sq in.

Area of 2 ~~angles~~ angles $3'' \times 2\frac{1}{2}'' \times \frac{3}{8}'' = 3.86$

- deduct one rivet $= \frac{7}{8}'' \times \frac{6}{8}'' = \frac{42}{64} = .7$

$\therefore 3.86 - .7 = 3.16$ in which is much in excess of area required. but least radius of gyration of a strut built up of 2 pairs of these angles with one system of lacing between angles placed $\frac{3}{4}''$ apart is 1.63

$\therefore \frac{l}{r} = \frac{138}{1.63} = 85+$ which is within limit also $\frac{l}{r}$ for B'm will fall in the limit.

Now the diagonals in upper laterals are 37 feet = 444 in but considering them stayed at middle l in this case would be $\frac{444}{2} = 222$ in.

$\therefore l/r = \frac{222}{1.63} = 136$ This does

not come in limits
therefore will have to use
angles $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{1}{2}''$ $r = 1.92$
$\therefore l/r = \frac{222}{1.92} = 115$ which is in
limit of 120. Therefore we
will use it.