Being a candidate for the Degree of Bachelor of Science in Civil Engineering I submit the following design of a Bowstring Truss Highway Bridge as a Thesis to Prof. D. C. Humphreys.

Respectfully submitted
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The design of a Bowstring truss highway bridge.

The span is taken as 120 feet

The economic depth was calculated from Merriman and Jacoby Part I and it was found to be 30.6 feet.

I took it 29 ft at the middle of the truss. The circular cord was used.

The number of panels is 6

The length of each panel is 20 feet.

Loads.

By the formula $w = 140 + 12b + 0.2bl - 4l$, (taken from Merriman and Jacoby Part II), I calculated the dead load and found it to be 740 lbs per linear foot.

$740 \times 120 = 88800$ lbs total load for both trusses. $88800 \div 2 = 44400$ lbs dead load for one truss.

$44400 \div 6 = 7400$ lbs dead load per panel.

The live load was taken as 80 lbs per sq. ft. of floor space.

$\therefore 80 \times 120 \times 18 = 172800$ lbs total live load $172800 \div 2 = 86400$ lbs per truss

$86400 \div 6 = 14400$ lbs per panel for the live load. $= 14.4$ kips

Snow load

The snow load is taken at 15 lbs per sq. ft. of floor surface.

$\therefore \dfrac{18 \times 20 \times 15}{2} = 2700$ lbs per panel load

Taken in kips it is 2.7

The ratio of the snow load to the uniform live load is $\dfrac{2.7}{14.4} = .1875$.

Wind load.

When the area of the side elevation of the bridge is not known the approximate method is to take the number of linear feet in the truss and multiply by 40 lbs per linear ft.

$\therefore$ Wind load $= 497.2 \times 40 = 19888$ lbs

$\therefore$ Pane load $= 19888 \div (6+4) = 1988.8$ lbs

This will be taken as $= 2.$ kips per panel applied at the top and bottom.

Stresses

The stresses for the truss members were obtained graphically by method in Merriman and Jacoby Part III

Loads were taken at the lower chord.

Lower Chords

Stress	RB	RC	RE
Dead Load	+ 20.94 Kips	+ 20.94 Kips	+ 22.6 Kips
Uniform L.L.	+ 42.2 "	+ 42.2 "	+ 45.1 "
Snow Load	+ 7.9 "	+ 7.9 "	+ 8.45 "
E Wind overturning	+ 7.2	+ 7.2 "	+ 6. "
W " "	− 7.2	− 7.2 "	− 6. "
E " on truss	+ 5.6	+ 8.9 "	+ 10. "
W " " "	− 5.6	− 8.9 "	− 10. "
Max.	+ 83.84 "	+ 83.84 "	+ 92.15 "
Min.	+ 8.1 "	+ 8.1 "	+ 6.6 "

Upper Chords

Stress	AB	AD	AF
Dead Load	− 28.1 Kips	− 24.72 Kips	− 23.4 Kips
Uniform L.L.	− 56.2 "	− 49.28 "	− 46.32 "
Snow Load	− 10.59 "	− 9.2 "	− 8.68 "
E. Wind Overturning	− 9.6 "	− 6.6 "	− 5.6 "
W. " "	+ 9.6 "	+ 6.6 "	+ 5.6 "
E " on truss	0	+ 3.3 "	+ 4.4 "
W " " "	0	− 3.3 "	− 4.4 "
Max.	− 104.49 "	− 93.10 "	− 88.40 "
Min.	− 18.5 "	− 21.42 "	− 22.2 "

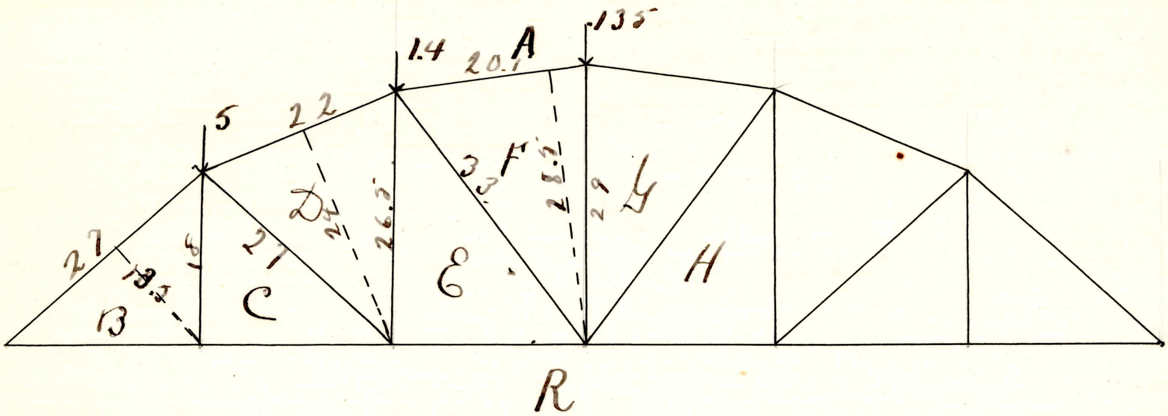
Verticals

Stress	B C	D E	F. G.
Dead Load	+ 7.4 Kips	− 6.1 Kips	− 6.1 kips
L.L. at apex 1	+ 14.4 "	− 5.92 "	− 3.33 "
2	0	− 11.84 "	− 6.66 "
3	0	+ 2.76 "	− 9.99 "
4	0	+ 1.84 "	+ 3.6 "
5	0 "	+ .92 "	+ 1.8
+ total	+ 14.4 "	+ 5.52 "	+ 5.4 "
− total	0 "	− 17.76 "	− 19.98 "
Uniform L.L.	+ 14.4 "	− 12.00 "	− 13.0 "
Snow Load	+ 2.7 "	− 2.25 "	− 2.43 "
max	+ 24.5 "	− 26.11 "	− 28.41 "
min	+ 74 "	− 0.	0

Diagonals

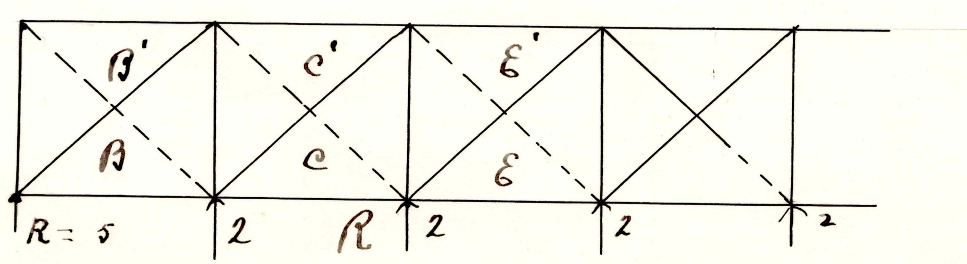
Stress	CD	EF	GH
Dead Load	+2.1 Kips	+ 6. Kips	- .74 Kips
L.L. at Apex 1	- 8.9 "	- 4.1 "	- 2.6 "
2	+ 5.92 "	- 8.2 "	- 5.2 "
3	+ 4.44 "	+ 6.66 "	- 7.8 "
4	+ 2.96 "	+ 4.44 "	+ 5.18 "
5	+ 1.48 "	+ 2.22 "	+ 2.59 "
+ total	+ 14.80 "	+ 13.32 "	+ 7.77 "
- total	- 8.9 "	- 12.3 "	- 15.64
Uniform W.L.	+ 4.1 "	+ 1.2 "	- 1.2
Snow Load	.75 "	+ .23 "	- .23
Max.	+ 17.65 "	+ 14.45	- 16.21
Min.	- 6.8 "	- 11.7	+ 7.7

By the above stresses it will be seen that counters are needed in all the panels, therefore they will be used.



Each panel wind load is 2 kips
Therefore at the middle panel the overturning moment is $\frac{1 \times 2.5}{18} = .135$ kips.
At the 2nd and 4th verticals the moment caused by overturning effect of the wind is $\frac{(2+1)\ 8.5}{18} = 1.4$ kips. Now at the end verticals it is $\frac{5 \times 18}{18} = 5$ kips.
The reaction at the end due to these downward loads is $5 + 1.4 + .07 = 6.47 = R_1$
Stresses due to these load on the chords are obtained as follows.
Lower chords.
Take C of M at upper vertex for R B.
Then $6.47 \times 20 - RB \times 18 = 0$ ∴ $RB = +7.2$ kips
$RC = RB = +7.2$ kips overturning stress.
For R E take C of M at apex 2.
Then $6.47 \times 40 - 5 \times 20 - RE \times 26.5 = 0$
∴ $RE = +6.$ kips.

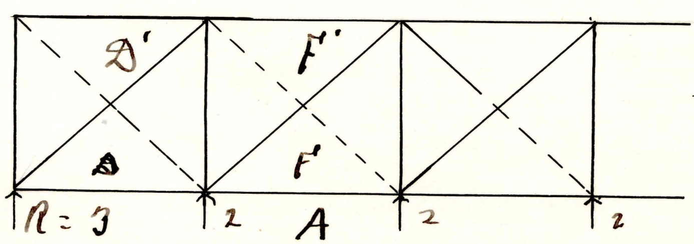
Upper Chords are obtained from the same figure.
For AB take C of m at lower vertex and measure leaver arm of AB.
Then $AB \times 13.5 + 6.47 \times 20 = 0$
$\therefore AB = -9.6$ kips overturning stress.
For AD take C of m at 2nd vertex at bottom
$AD \times 24 = -6.47 \times 40 + 5 \times 20 = -158.8$
$\therefore AD = -6.6$ Kips overturning stress.
For AF take C of m at 3rd vertex at bottom.
Then $AF \times 28.5 = 6.47 \times 60 - 5 \times 40 - 1.4 \times 20 = -168.2$
$\therefore AF = -5.6$
For wind on the opposite side the stresses are the same except the sign is changed.



Stresses due to wind on truss for the Lower Lateral system.

For R B take c of m at ~~ist~~ 1st vertex
Then $RB \times 18 = 5 \times 20$ $\therefore RB = 5.6$ kips
For R C take c of m at 2nd vertex
Then $RC \times 18 = 5 \times 40 - 2 \times 20 = 160$
$\therefore RC = + 8.8$ Kips.
For R E take C. of m. at 3rd vertex
Then $RE \times 18 = 5 \times 60 - 2 \times 40 - 2 \times 20 = 180$
$\therefore RE = 10$ kips.

Stress in diagonals is $V \sec \theta$
$V_1 = 5000$ $\sec \theta = 1.494$
Therefore $BB' = 5000 \times 1.494 = 7470$ lbs,
In $CC' = 3000 \times 1.494 = 4482$ lbs.
In $EE' = 1000 \times 1.494 = 1494$ lbs.
In the struts the stress is very small.



For the Upper Lateral system the stresses due to wind on the truss are obtained anylitically as follows.

For AD take c of m at opposite vertex
Then $AD \times 18 = 3 \times 20$ $\quad \therefore AD = 3.3$ kips
For AF take c of m at opposite vertex
$\therefore AF \times 18 = 3 \times 46 - 2 \times 20 = 80$
$\therefore AF = 4.4$ kips.

The diagonals are obtained by multiplying the vertical shear by $\sec\theta$.
$\therefore DD' = 3. \times 1.494 = 4.482$ kips
$FF' = 1. \times 1.494 = 1.494$ kips.

When the wind is blowing in the opposite direction the stresses are the same but the signs are changed.

Design of Pieces.

Stringers for Roadway.

The dead load for these stringers consists of only the weight of the floor covering which is $2 \times 4 \times 4\frac{1}{2} = 36$ lbs per linear foot.

Bending moment due to this weight is $M_1 = \frac{1}{8} wl^2 = \frac{36 \times 400 \times 12}{8} = 21600$ lb-in dead load moment.

Live load moment is $M_2 = \frac{80 \times 400}{8} \times 12$

$\therefore M_2 = 48000$ lb-in.

$\therefore$ total bending moment is $48000 + 21600 = 69600$ lb-in.

This requires a section modulus of $\frac{69600}{7400} = 9.42$ in.^{3}

A 7" 15# I beam has a section modulus of 10.4 in.^{3} Therefore this size will be used throughout the bridge.

The moment due to its own weight is $\frac{15 \times \cancel{400}^{50} \times \cancel{12}}{\cancel{8}} = 750$ lbs and as this is smaller than $\frac{1}{10}$ of 69600 it will be neglected.

They will be spaced 2 feet apart therefore it will require 48 I in the whole span.

Floor Beams.

The dead load concentration under a roadway stringer is as follows

Weight due to flooring $= 2 \times 4 \times 4\frac{1}{2} \times 20 = 720$

" " " stringer $= 15 \times 20 \quad = 300$

Total for roadway stringer $= 1020$ lbs.

Left reaction due to this is

$\frac{1020}{90}(16+14+12+10+8+6+4+2) = R_1$

$\therefore R_1 = 3860$ lbs.

$\therefore M = R \times 9.5 \times 12 = 3860 \times 9.5 \times 12 = 439080.$

The section modulus is $\frac{439080}{7400} = 59.3$.

A 15" 45 lb I satisfies this condition and will be used.

There are 5 of these I beams needed.

Tension members.

Lower Chords.

For the tension members I used an allowable unit stress of 10000 lbs per sq. in.

R.B. $S = \frac{P}{a}$ where P = stress in member S = unit stress and a = area of cross section. Maximum stress is 83840 lbs. I will try 2 eye bars. $\therefore P = \frac{83840}{2} = 41920$

$\therefore a = \frac{41920}{10000} = 4.2$ sq in area cross section that is needed.

From Cambria pp. 339
I will use a 5"×1" eye bar this will satisfy the conditions close enough.
Use the same for RC.

RE. The maximum stress in RE is 92150. For this I will try 2 eye bars. Therefore $P = \frac{92150}{2} = 46075$ lbs
$\therefore a = \frac{46075}{10000} = 4.6$ sq. in. area.
The same eye bars as in RB will be used as their area of cross section is plenty large enough.
For both trusses there will be needed 24 eye bars of this size.
The length of each will be $24\frac{7}{12}$ ft.

Vertical Post

BC For BC the maximum stress in this member is 24500 lbs.
Try 2 eye bars. Then $P = \frac{24500}{2} = 12250$
$\therefore a = \frac{12250}{10000} = 1.23$ sq. in.
A bar 3"×3/4" will be used for this as it gives plenty area.
The length of this bar is $18' + 3\frac{11}{12}' = 21\frac{11}{12}'$

Diagonals.

CD The maximum stress in CD is +17650 lbs. $\therefore a = \frac{17650}{10000} = 1.77$ sq. in.
For this I will use one eye bar 3" x $\frac{3}{4}$" this satisfies the condition close enough.
The length is $27' + 2\frac{11}{12}' = 29\frac{11}{12}$

EF. The maximum stress in EF is +14450 lbs.
Then $a = \frac{14450}{10000} = 1.45$ sq. in.
I will use one eye bar same size as above.
Its length is ~~the same as the above~~
$33' + 2\frac{11}{12}' = 35\frac{11}{12}'$.

By the conditions of the maximum and minimum stresses counters are needed in all the panels.
Therefore loops will be used
For the first panel the size will be taken as 1" x 1".
In the 2nd it will be taken
$1\frac{1}{4}$" x $1\frac{1}{4}$".

Posts.

End Post $\underline{AB}$

The section will consist of cover plate, two flats and two channels flanges turned out. The flats will be riveted to the lower flanges with $\frac{3}{4}$" rivets spaced 4" apart.

The distance from back to back will be taken as 6" + 1.5" = 7.5". The 1.5" is added for rivet heads and clearance.

A 10", 20 lbs channel will be tried, the width is 2.6". ∴ The width of the cover plate would have to be 12.7". It will be taken 13" and $\frac{7}{16}$" thick. The two flats will be taken as 2.6" × $\frac{1}{2}$".

Piece	A	l	Al
2 Channels	8.92	5.218	46.544
1 Plate	5.7	0	0
2 Flats	2.6	10.218	26.567
Total	17.22		73.111

Distance to center of gravity from center of cover plate is $g = \frac{\Sigma A l}{\Sigma A} = \frac{73.111}{17.22} = 4.246$ in.

The eccentricity of the section is
$e = \left(\frac{10}{2} + \frac{7}{32}\right) - 4.246 = .972$ inches.
The moment of inertia is now computed, neglecting the I of the plates about their axis parallel to their widths

Piece	A	I'	h	ah^2
2 Channels	8.92	161.74	.972	8.42
1 Plate	5.7	0	4.246	102.94
2 Flats	2.6	0	5.972	92.66
Total	17.22			204.02

$\therefore I = I' + \Sigma ah^2 = 161.74 + 204.02 = 365.76$ in.
The radius of gyration $r = \left(\frac{365.76}{17.22}\right)^{\frac{1}{2}} = 4.54$ in
By column Formula
$P = 10000 - \frac{140 \times 27.1}{4.56} = 6395.6$ lbs in^2.
Stress in chords is 104490
$\therefore a = \frac{104490}{6395.6} = 16.34$ in^2.
Therefore the above will be used.
The end post is also subject to bending due to wind at a point where the knee bracing joins it.
$\therefore 12(27.1 - 10) \times 2000 \times 2.5 = 513000$ lb in.
That is where the portal strut is taken as 5 ft and the knee brace as 5 ft farther down.

The determination of the approximate area required in the section may be found from the formula.

$$\frac{450\,l}{r} + \frac{P}{A} + \frac{Mc}{I} = 10000$$

$\frac{2}{3}$ of the total bending moment is taken to reduce it to live load equivalent.

$$\therefore \frac{450 \times 27.1}{4.54} + \frac{104490}{A} + \frac{342000 \times 6.5}{A(4.54)^2} = 10000$$

Where $c = 6.5$ = half width cover plate and $r = 4.54$ = radius of gyration.

$\therefore$ clearing $A \times 205938.95 = 4476538.9$

$\therefore A = 21.7 \text{ in}^2$ necessary.

Therefore an additional area of 4.48 in^2 has to added to the original to make it strong enough.

I will use $7.5'' \times \frac{3}{8}''$ plates on the outside of the ~~flange~~ channels.

These will be riveted by $\frac{7}{8}''$ rivets spaced 4 in apart.

The cover plate will also be riveted on with $\frac{7}{8}''$ rivets spaced 4 in apart.

Vertical Posts.

DE. The vertical post DE is in compression. By specifications the radius of gyration cannot be less than $\frac{26.5 \times 12}{100} = 3.18$ inches.
The thickness must be $\frac{5}{16}$" thick.
For this I will try 9" 15 lb channels.
The area is $4.41 \times 2 = 8.82$
The radius of gyration is 3.4
$\therefore P = 10000 - \frac{540 \times 26.5}{3.4} = 5780$ lbs.
area $= \frac{26110}{5780} = 4.52$ sq in. area required.
For the spacing of the channels I used the formula given in Part III pp 322-323
$r^2 = r_1^2 + (c-g)^2$. Obtaining from handbook values of r_1 and g and assuming c, I substituted in the formula.
$r^2 = (.66)^2 - (3.25 - .59)^2 = 7.51 \quad \therefore r = 2.74$
Substituting for r in equation
$c = \frac{ar^2}{m}\left(S - \frac{P}{a} - \frac{4K(l-x)x}{lr}\right)$. $a = 8.82$
$m = \frac{2}{3}\left(\frac{2000 \times 15 \times 12}{4}\right) = 60000$ live load equivalent. $\frac{P}{a} = \frac{26110}{8.82} = 2960$ $K = 540$
$l = 26.5'$ $x = 11.5'$ S taken 11000
$\therefore c = 3.41$ this is too much but is so close that it will be used.
$\therefore$ Distance apart of channels $= 6.5$ inches

F.G. The maximum stress in F.G. is 28410 lbs.
The channels used for D E will be tried. The length of this post is 29 ft.
Unit load is $P = 10000 - \frac{54 \times 12 \times 29}{3.4} = 5,400$
area $= \frac{28410}{5400} = 5.26$ sq. in.
This will be used for F.G.
The same spacing as in D E will also be used.
The channels will be connected by lattice bars 2" in width and $\frac{5}{16}$" thick connected by $\frac{3}{4}$" rivets.

Hangers.

The shear at the end of the floor beam is 3860 lbs. This is the maximum stress that will be ~~cond~~ come in the hanger.
$\therefore \frac{3860}{10000} = .386$ in^2 required.
For to be on the safe side I will use two rods 1" in diameter and connected by a flat plate 1" thick connecting them across the ends, and held in place by nuts on the ends of the rods.

Upper Chords.

For the upper chord AD a $\frac{5}{16}$" X 13" cover plate and two 10" 20 lb channels will be used. Then.

Piece	a	l	a l
2 Channels	8.92	5.1563	45.89
1 Plate	4.06	0	0
Total	12.98		45.89

now $g = \frac{\Sigma a l}{\Sigma a} = \frac{45.89}{12.98} = 3.535$ inches.

The eccentricity of the section e is

$\left(\frac{10}{2} + \frac{5}{32}\right) - 3.577 = 1.579 = e$

The moment of Inertia is now calculated, neglecting the I of plate about its own axis parallel to plate.

Piece	A	I'	h	a h²
2 Channels	8.92	161.74	1.579	22.2
1 Plate	4.06	0	3.535	52.
Total	12.98			74.2

$I = I' + \Sigma A h^2 = 161.74 + 74.2 = 235.94$ in⁴

radius of gyration $r = \left(\frac{235.94}{12.98}\right) = 4.35$ in

$P = 10000 - 540 \frac{22}{4.35} = 7288$ lbs

$a = \frac{93100}{7288} = 12.77$ therefore above will be used.

AF

For the upper chord AF I will use the same channels as in AD, the cover plate of the same width but a thickness of $\frac{3}{8}''$.

Piece	a	l	al
2 channels	8.92	5.187	46.27
1 Plate	4.875	0	0
Total	13.795		46.27

$\therefore g = \dfrac{46.27}{13.80} = 3.35$ in

$e = \left(\dfrac{10}{2} + \dfrac{3}{16}\right) - 3.35 = 1.85$ in.

Moment of Inertia is obtained as follows

Piece	A	I'	h	a h^2
2 channels	8.92	161.74	1.85 ~~75~~	28.6
1 Plate	4.875	0	3.35	54
Total	13.795	0		82.6

$\therefore I = I' + \Sigma a h^2 = 161.74 + 82.6 = 244.34$ in^4

radius of gyration $= \left(\dfrac{244.34}{13.80}\right)^{\frac{1}{2}} = 4.2$ in

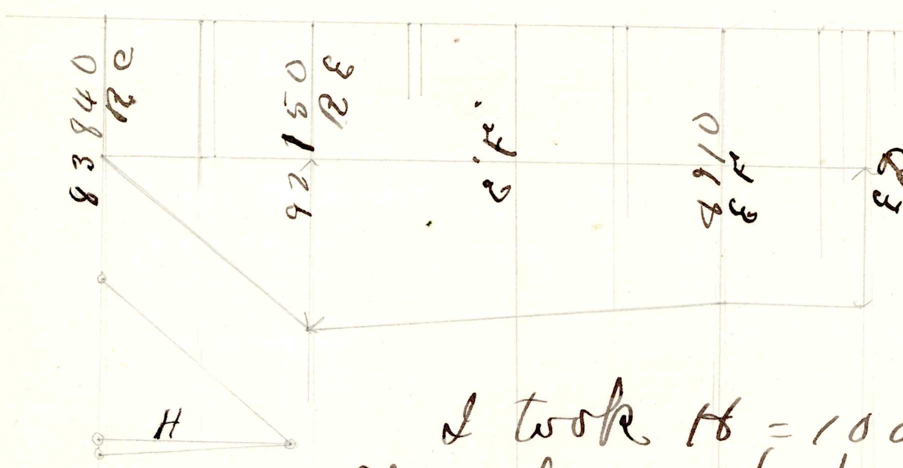
$P = 10000 - \dfrac{540 \times 20}{4.2} = 7440$ lbs

$\therefore a = \dfrac{88400}{7440} = 11.88$ in^2. This is sufficient and the above will be used.

The rivets in the cover plates will be taken $\frac{3}{4}$" in diameter and spaced 4 in apart.

Design of Pins.

For the size of the pin I calculated it graphically by the method given in Part III pp. 266.



This is for panel point 2.

I took $H = 100000$ lbs. $= 1$ in

Then the greatest moment $= Hy$

$\therefore M = 100000 \times .9 = 90000$ lbs.

Taken from the handbook this requires a pin $3\frac{5}{8}$" in diameter.

For safety this pin will be used throughout the truss.

Design of Lower Laterals

For this I took the allowable unit as 18000 lbs. The computed stress divided by this gives the area required. For first panel it is .41 sq in, second it is .25 in². These are all very small and would require a very small angle but they must have ~~an~~ net area of not less than $\frac{3}{4}$" sq in.

One 3" x 2½" x $\frac{5}{16}$" angle will be used, it has a gross area of 1.63 sq in, which is sufficient.

These angles will be connected in the center by $\frac{5}{16}$" plates with $\frac{3}{4}$" rivets and the ends will be connected to plates, $\frac{5}{16}$" thick, which are fastened to the floor beam by $\frac{3}{4}$" rivets.

These same size angles will be used for all the panels and connected by the same plates and rivets.

Where the angles cross one another ~~they~~ one will be cut and riveted to the plate by a sufficient number of rivets to make up for the part cut out.

Upper Laterals.

The computed stresses are all very small requiring less than 3/4 sq. in. A $2\frac{1}{2}'' \times 2'' \times \frac{5}{16}''$ angle will be used this is plenty large enough.
It will be connected to the top chord by $\frac{5}{16}''$ plate and $\frac{3}{4}''$ rivets.
The angles will be fastened together with a plate $\frac{5}{16}''$ thick and with $\frac{3}{4}''$ rivets.
This size will be used for all the panels.

Sway bracing.

By the specifications sway bracing is required.
The computed stress for its lower chord is $\frac{2000 \times 29}{2 \times 11.5} = 2522.$ lbs
The radius of gyration cannot be less than $\frac{12 \times 18}{120} = 1.8$ inches.
Two $3\frac{1}{2}'' \times 3'' \times \frac{5}{16}''$ angles spaced $\frac{1}{2}''$ back to back gives a radius of gyration of 1.8 inches. Therefore these will be used although the area is in excess of that required.

For the upper chord the same size angles will be used.
The two chords will be connected by two panels of latticing consisting of $3'' \times 3'' \times \frac{5}{16}''$ angles.
The upper chord is connected to the post by a plate $\frac{5}{16}''$ thick and with $\frac{3}{4}''$ rivets. The lower chord is connected by a plate riveted to the side of the post with angles.
The thickness of the plate will be taken as $\frac{1}{2}''$ and $\frac{3}{4}''$ rivets will be used in it.
The lattice bars will be fastened to the chords by plates put between the angles and riveted with $\frac{3}{4}''$ rivets.
The same size angles will be used for all the sway bracing but at the other posts it will be of different lengths.
The bars are fastened together at the middle by plates $\frac{5}{16}''$ ~~and~~ thick and with $\frac{3}{4}''$ rivets.

Portal Bracing.

The radius of gyration cannot be less than 1.8 and the thickness of the angles not less than $\frac{6}{16}$".

For the Lower Chord. two $3\frac{1}{2}$" x 3" x $\frac{5}{16}$" angles spaced $\frac{3}{4}$" back to back gives a radius of gyration of 1.8

Then $P = 13000 - 720 \frac{18}{1.8} = 5800$ lbs

$a = \frac{13550}{5800} = 2.34$ in^2 required.

The given area of the above is 3.88 in^2 as this is sufficient it will be used for both chords.

The knee bracing of the portal strut will now be examined. The stress is 15,674 lbs. Using same angles as above

$P = 13000 - 720 \frac{7.1}{.9} = 7320$

$\therefore a = \frac{15674}{7320} = 2.14$ in^2 area required

as 3.88 is the gross area given it will be taken as above.

The diagonals in the portal bracing use 2 ∠s $3\frac{1}{2}$ x 3 x $\frac{3}{4}$".

The stress = 11500 x 3.72 = 42,780 lbs

$a = \frac{42780}{5800} = 7.38$ in^2 area required.

The above gives an area of 8.64 in^2

Therefore it will be used.
The chords are connected to the end post by plates $\frac{1}{16}$" thick and with 7/8" rivets.
The bracing are connected in the middle by a $\frac{3}{4}$" plate and with $\frac{3}{4}$" rivets.
The Ends of the bridge will be rested on plates about 1" thick and 2 ft square and these will be fastened to the masonry by pins in the foundation.