SPECIFICATIONS AND DESIGN

OF A

288 SPAN THROUGH, PIN-CONNECTED HIGHWAY BRIDGE.

9 Panels 32 feet, divided by Sub-verticals. Curved Upper Chord.

SPECIFICATIONS

General Specifications: Circular No. 100, U. S. Department of Agriculture, Office of Public Roads. Filed herewith.

ADDITIONAL SPECIFICATIONS

In case the maximum stresses due to wind added to the maximum stresses due to vertical loading (including impact) shall exceed 19000 pounds per square inch, properly reduced for compression, addition must be made to such sections until this limit is not exceeded. The permissible stresses for the connections shall be increased proportionally. Should the stresses be reversed in any possible case, proper provision must be made for such stresses in the opposite direction.

The shearing stress on rivets, bolts, or pins shall not exceed 11000 pounds per square inch of section; and the pressure upon the bearing surface of the projected semi-intrados (diameter times thickness) of the rivet, bolt, or pin hole shall not exceed 22000 pounds per square inch. In field riveting, the number of rivets thus found shall be increased 25 percent if driven by hand, and 10 percent if satisfactory power riveters are used. The amount of field riveting must be reduced to a minimum, without, however, diminishing the number of rivets requisite for strength and rigidity. Whenever it is practicable, all designs are to be so made that the field rivets can be driven readily. For members of any importance, more than two rivets are to be used for each connection. Rivets are not to be used in direct tension.

If the extreme fiber stress resulting from the bending due to the weight only of any member does not exceed 10 percent of the specified unit stress, the effect of such bending may be ignored; but if it does so exceed, its effect must be combined with those of the other stresses, using, however, for determining the sectional area, a unit stress 10 percent greater than that specified.

In general, all trusses shall have main end posts inclined. The effective length of pin-connected spans shall be the distance between centers of end pins of trusses. The effective depth shall be the perpendicular distance between gravity lines of chords, which lines must pass through the centers of pins.

In all main members having an excess of section above that called for by the greatest combination of stresses, the entire detailing is to be proportioned to correspond with the utmost working capacity of the member, and not merely for the greatest total stress to which it may be subjected. In this connection, though, the reduced capacity of single angles connected by one leg only must not be forgotten.

General Principles of design from Waddell's Specifications.

Floor-Beams: Effective length to be the distance between center lines of trusses.

Sections of Intermediate Posts: The effective length shall be the greatest length between points of the axis that are rigidly held in the direction in which the strength is being considered. The least width of posts in pin-connected trusses shall be limited to 10 inches.

Sections of Diagonals and Suspender: Counter-stresses must be provided for wherever caused by the increased live load; and in case of reversal of stress the member must be designed to resist such reversal. The use of more than a single system of cancellation in bridges shall be confined entirely to lateral systems and sway bracing, except that in the middle panels of trusses two rigid diagonals connected at their intersection may, for appearance, be employed, provided that either diagonal have sufficient strength to carry the entire shear in either tension or compression, and that the adjacent vertical posts be figured accordingly. All through spans shall have stiff end vertical suspenders.

Diameter of Pins: The stress in the outer fibers of pins shall not exceed 25000 pounds per square inch, the points of application of the stresses in the connecting members being taken at the certers of bearings. In designing all pin-connected work ample clearance for packing must be provided, and ample room must be left for assembling members in confined spaces. Lower chords are to be packed as closely as possible, and in such a manner as to produce the least bending moment on the pins, but adjacent eye-bars in the same panel must never have less than a one-half inch space between them, in order to facilitate painting. The various members attached to any pin must be packed as closely as practicable, and all interior vacant spaces must be filled with steel fillers, where their omission would permit of the motion of any member of the pin. All bars are to lie in planes as nearly as possible parallel to the central plane of the truss, no divergence exceeding one-eighth of an inch to the foot being permitted.

Upper Chord Sections: In members subject to compression, rivets shall be so spaced that they shall not be farther apart in the direction of the stress than sixteen times the thickness of the thinnest external plate connected, and not more than fifty times that thickness at right angles to the direction of the stress.

Section of Inclined End Post: The inclined end post must be so proportioned that the algebraic sum of the stresses per square inch resulting from the direct compression and the maximum bending moment due to the wind pressure shall not exceed 19000 pounds per square inch. Every column that acts as a beam also must have solid webs at right angles to each other, as no reliance shall be placed on lacing to carry a transverse load down the column.

Lateral Bracing: All lateral bracing shall be made of shapes which can resist compression as well as tension. In detailing struts composed of four angles with a single line of lacing, the clear distance between backs of angles shall never be made less than three-quarters of an inch, in order to permit the insertion of a small paint-brush. The stiff diagonals of the lower lateral system, of which there shall be two in each panel, shall be riveted rigidly to the stringers where they cross them, so as to transfer in an effective manner the thrust of braked trains to the truss posts without causing the floor beams to bend horizontally. In designing short members with riveted connections the sectional area of the piece shall be increased from 10 percent for 6 inches x $3\frac{1}{2}$ inches angles to 25 percent for equal-legged angles beyond the theoretical requirements for the direct stresses, so as to compensate for the secondary stresses due to the eccentric grip of the rivets.

Portal Bracing: All through spans shall have stiff portal bracing at each end, connected rigidly to the end posts. The bracing shall be made as deep as the specified clear head room will allow. When the height of the trusses es great enough to permit it, there shall be used at each panel point a rigid bracing frame riveted to the top lateral strut, and to the posts, and carried down to the clearance line. When the truss depth is not great enough for this detail, corner brackets of proper size, strength, and rigidity are to be riveted between the posts and the upper lateral struts.

Pin Plates: Rivets shall not be countersunk in plates less than seven-sixteenths of an inch in thickness.

Pin plates shall be used at all pin holes in built members for the double purpose of reinforcing for the metal cut away and reducing the unit pressure on pin and bearing to or below the specified limit. They shall be of such size as to distribute properly, through the rivets, the pressure carried by such plates to both flanges and web of each segment of the member; and they shall extend at least six inches within the tie plates of said member, so as to provide for not less than two tranverse rows of rivets there.

It is always better, whenever practicable, to avoid cutting away the ends of channels, but if they must be trimmed, the ends must be reenforced so that the strength of the member shall not be reduced by the trimming.

In riveted tension members, the net section through any pin hole shall have an area 40 percent in excess of the net sectional area of the body of the member. The net section outside of the pin hole along the center line of stress shall be at least 70 percent of the net section through the pin hole.

Tie Plates and Lacing: At the ends of compression members the pitch of rivets shall not exceed four diameters of the rivet, for a distance equal to twice the greatest width of the member.

All segments of compression members connected by lacing only, shall have tie plates placed as near the ends as practicable. The tie plates shall have a length not less than the greatest width of the member, and a thickness not less than one-fortieth of the distance between the lines of connecting rivets, measured at right angles to the length of the member.

Single lattice bars shall have a thickness of not less than one-fortieth, and double bars connected by a rivet at the intersection of not less than one-sixtieth of the distance between the rivets connecting them to the members; and their width shall be:

For 15" channels, or built sections with 3½" or 4" angles - 2½ inches (seven-eighth inch rivets).

For 12" and 10" channels, or built sections with 3" angles - 2¼ inches (three-fourth inch rivets).

For 9" and 8" channels, or built sections with 2½" angles - 2 inches (five-eighth inch rivets).

The distance between connections of lattice bars shall not exceed eight times the least width of the segments connected.

End Bearings: The greatest allowable pressure upon expansion rollers of fixed spans, when impact is considered, shall be determined by the equation $p = 600\ d$, where p is the allowable pressure in pounds per linear inch of roller, and d is the diameter of the roller in inches. The least allowable diameter for expansion rollers is four inches. The bearing shall be so designed as to permit a free movement of the rollers in the longitudinal direction of the span sufficient to take up the extreme variations in length due to temperature changes and deflections, and at the same time prevent any tranverse motion of the end of the span.

All shoe plates, bed plates, and roller plates are to be so stiffened that the extreme fiber stress under bending, when impact is included, shall not exceed 16000 pounds per square inch. Bed plates shall be so proportional that the pressure upon masonry (including impact) will not exceed 400 pounds per square inch.

Pedestals shall be either of cast steel or built up of plates and shapes. In built pedestals, all bearing surfaces of the base plates and vertical bearing plates must be planed. The vertical plates must be secured to the base by angles having at least two rows of rivets in the vertical legs; and the said vertical plates must bear properly from end to end upon the base. No base plate, vertical plate, or connection angle shall be less in thickness than three-quarters of an inch. The vertical plates shall be of sufficient height and must contain enough metal and rivets to distribute properly the loads over the bearings or rollers. The bases of all cast-steel pedestals shall be planed, so as to bear properly on the masonry or rollers. All rollers and the faces of base plates in contact therewith are to be planed smooth, so as furnish perfect contact between rollers and plates throughout their entire length. All pedestals, whether built or cast, must have one or more diaphragms between webs, carried up as high as the general detailing will permit, so as to transmit any transverse horizontal thrust to the base without overstraining the webs by bending in their weakest direction.

Design -

Approximation of weight:

$w = 230 - 0.75l + 0.0153l^2 = 1291$ lbs per ft for trusses alone

Also $w = 262 + 0.96l + 0.006l^2 = 1037$ lbs - per ft for two trusses alone.

Use ~~Second~~ formula, wt. trusses = 1037 lbs. per ft = 1037.0

Try 24" 100 lb. I-Beam for Floor Beam $= \frac{100 \times 20}{16}$ = 125.0

16" 25 .. .. Stringers at 3' (approx) = 7 × 25 = 175.0 ~~294.0~~

Concrete Paving average thickness 4½ $= 17 \times \frac{4\frac{1}{2}}{12} \times 288 \times 150$ = 274.5

" Sidewalks " " 3½ $= 10 \times \frac{3\frac{1}{2}}{12} \times 288 \times 150$ = 120.0

Gravel covering for roadway 6" $= 17 \times \frac{6}{12} \times 288 \times 100$ = 244.8

1976.3

Ck by formula for total weight.

$w = 300 + l + 22b + \frac{1}{15}bl\left(1 + \frac{l}{1000}\right) = 1930$ lbs per ft

∴ Use 2000 lbs. per ft. for dead load for ~~each~~ both trusses

Dead panel load =

244.8
120.0
274.5
125
175
1.037
640.637

$\frac{640.637}{288} = 2.22$ Kips-ft

Design –

Approximation of weight.

$w = 230 - 0.75l + 0.0153l^2 = 1291$ lbs. per ft. for trusses alone

Floor Beams – 24" 100 lb I-Beam. $\frac{100 \times 20}{16} = 125$ " " "

Stringers – 10" 25 lb, I-Beam. $7 \times 25 = 175$ " " "

Concrete paving, assume 4" – $\frac{1}{3} \times 17 \times 150 = 850$ " " "

" " (sidewalk) " 3" – $\frac{1}{4} \times 12 \times 150 = 450$ " " "

Gravel Cushion " 6" – $\frac{1}{2} \times 17 \times 100 = 850$ " " "

$2 \,\underline{|\,3741}$ " " "

1870.5 " " "

assume D.L. 2000 lbs per running ft.

Ck formula for total weight $w = 300 + l + 22b + \frac{1}{15} bl(1 + \frac{l}{1000}) = 1930$ lb-ft running

Panel load D.L. = $2000 \times 16 = 32$ Kips.

(assume 100 lbs sq. ft.) (see following) L.L = $16 \times 20 \times 100 \times \frac{1}{2} = 16$ " . } per panel for 1 Truss.

Stringers –

D.L. assumes 10" 25 lb I-beams spaced 3'

Floor and cushion per width 3', per ft. = $\frac{2150}{17} \times 3 = 380$ lbs (see above.)

L.L. Roller (see Plate). Since $\frac{10}{16} > 0.586$, put Heavy wheel at center, max. moment = $\frac{1}{3} \times \frac{10000}{2} \times 16 \times \frac{12}{2} = 160000$ lbs-in. = 160 Kips-in.

L.L. 100 lbs per ft², Max M. = $\frac{1}{8} wl^2 = \frac{1}{8} \times 300 \times 16 \times 16 \times 12 = 115{,}000$.

∴ Use Roller results.

D.L. Moment = $\frac{1}{8} wl^2 = \frac{1}{8} \times 380 \times 16 \times 16 \times 12 = 146$ Kips-in.

$\underline{160}$ " "

306 " "

Allow 13,000 lbs-in², tension, $\frac{306}{13} = 23.5$ in² required.

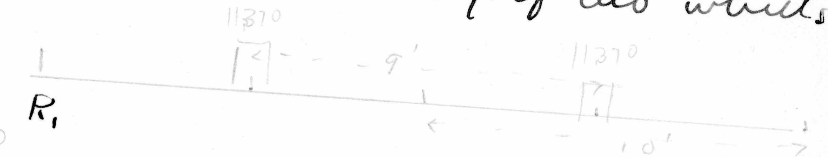
10' 25 lb. I-Beam gives area 24.4 in² ∴ Use.

Floor Beams.

Floor and cushion = 2150 × 16 = 34.4 Kips per ft. (see above.)
stringers = 7 × 25 × 16 = 2.8 "
Floor Beam 20 × 80 = 1.6 "
38.8

D.L. Max Moment = $\frac{1}{8}$ × 38.8 × 20 × 12 = 1,164.0 Kips-in.

L.L. Roller small wheel transfers to large $\left(\frac{16-10}{16}\right) \times 5{,}000 = 1{,}370$
placing large wheels over Beam, with c.g. of two wheels at ℄ of Floor Beam.

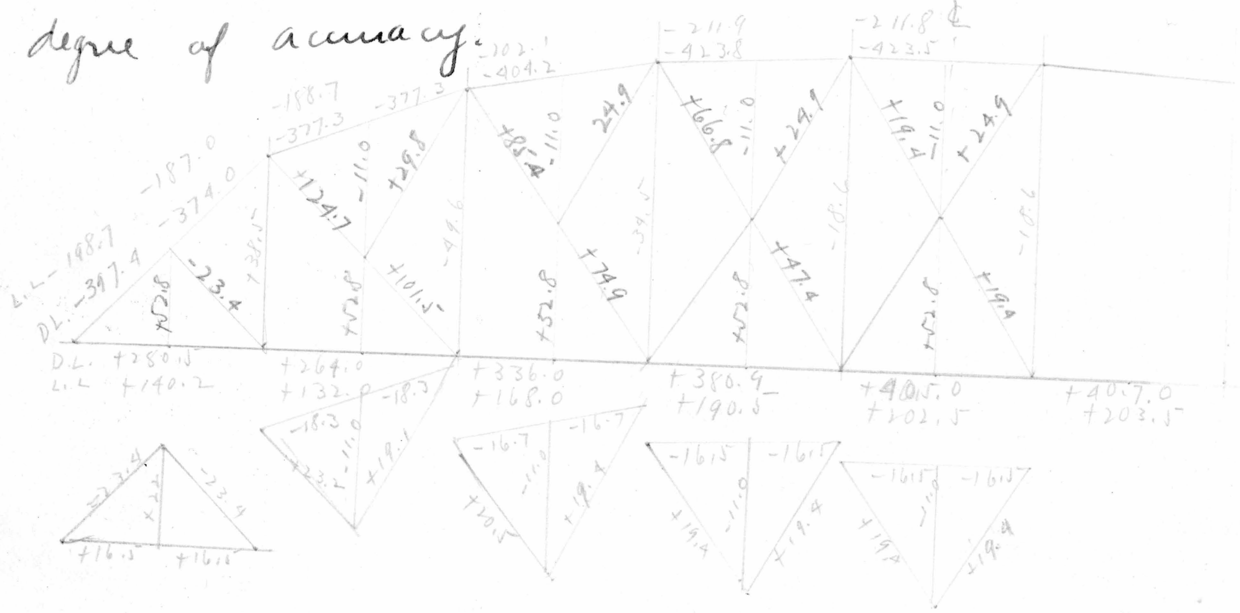


$R_1 = \left(\frac{14.5}{20} + \frac{8.5}{20}\right) \times 11370 = 13{,}100$
$R_2 = 9640$
M = 9640 × 8.5 × 12 = 983.000 lbs in.
L.L. (100 lbs ft²) $\frac{1}{8}$ × 100 × 18 × 16 × 16 × 12 = 692,000 lbs in.
∴ Use Roller Load M = 983 Kips in.
D.L. Mom't = 1164 "
2147 "

∴ Use 24" 80 lb. I Beam, $\frac{I}{c}$ = 174.0 $\frac{2147}{13}$ = 165.2 ~~in~~ = $\frac{I}{c}$

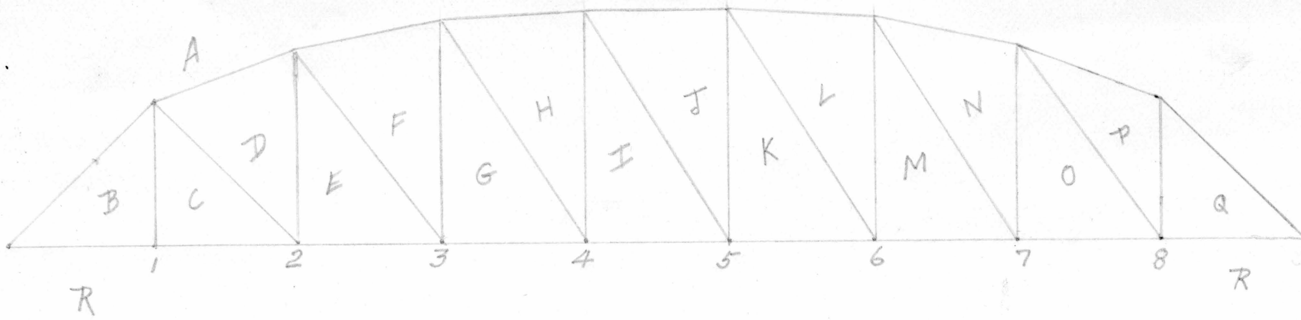
Stresses –

By Method of Moments, measuring lever arms, the D.L. stresses were found as in below figure. These were checked, by graphical method, to a reasonable degree of accuracy.



The wind loads were then calculated, for upper and lower lateral system, Dead and L.L. allowing 150 lbs. per running ft. ~~day~~ and the live load calculated for the bridge and a stress sheet made out as per following pages.

Live Load Stresses.



	Main Diagonals				Counters		
	CD	EF	GH	IJ	KL	MN	OP
LL of 1	−7.3	−3.9	−2.6	−2.3	−2.1	−2.0	−2.4
2	+14.0	−7.9	−5.3	−4.5	−4.3	−4.0	−4.7
3	+12.0	+10.8	−7.9	−6.8	−6.4	−6.0	−7.1
4	+10.0	+9.0	+10.6	−9.0	−9.7	−8.0	−9.5
5	+8.0	+7.2	+9.7	+9.5	−10.6	−10.0	−11.9
6	+6.0	+5.4	+6.4	+7.1	+8.4	−12.0	−13.0
7	+4.0	+3.6	+4.3	+4.7	+5.6	+9.0	−16.6
8	+2.0	+1.8	+2.1	+2.4	+2.8	+4.5	+8.8
* +Total	112.0	75.6	63.0	48.0	33.6	27.0	17.6
−Total	14.6	23.6	31.2	46.0	63.0	84.0	134.4
Max. L.L.	+97.4	+52.0	+31.8	+2.0	−29.4	−79.5	−116.8
D.L.	+124.7	+85.4	+66.8	+19.4	−1.85		

* Totals are 2 X addition of columns on account of error.

Verticals

	BC	DE	FG	HI	JK	LM	NO	PQ
1	+16.0	+5.2	+3.2	+2.2	+2.0	+1.8	+1.7	+1.9
2	0	+10.2	+6.4	+4.4	+4.0	+3.6	+3.4	+3.8
3	0	−8.4	+9.6	+6.6	+6.0	+5.4	+5.1	+5.7
4	0	−7.0	−7.5	+8.8	+8.0	+7.2	+6.8	+7.6
5	0	−5.6	−6.0	−7.2	+10.0	+9.0	+8.5	+9.5
6	0	−4.2	−4.5	−5.4	−6.0	+10.8	+10.2	+11.4
7	0	−2.8	−3.0	−3.6	−4.0	−4.8	+11.9	+13.3
8	0	−1.4	−1.5	−1.8	−2.0	−2.4	−3.8	+15.2
* +Total	32.0	+30.8	38.4	44.0	60.0	75.6	115.5	136.7
−Total	0	−58.8	45.0	36.0	24.0	14.4	−7.6	0
Max. L.L.	+32.0	−28.0	−6.6	+8.0	+36.0	+61.2	+107.9	+136.7
D.L.	+38.5	−49.6	−34.5	−18.6	−18.6			

* Totals are 2 X addition of columns on account of error.

The values in the above tables were gotten by the graphic method.

Stress Sheet.

	End Post	Upper Chord				Lower Chord					
	AB	AD	AF	AH	AJ	RB	RC	RE	RG	RI	RK
Dead Load	−397.4	−377.3	−404.2	−423.8	−423.5	+280.5	+264.0	+236.0	+380.9	+405.0	+407.0
Live Load	−198.7	−188.7	−202.1	−211.9	−211.8	+140.2	+132.0	+168.0	+190.5	+202.0	+203.5
Impact	−33.8	−32.0	−34.4	−36.0	−36.0	+23.9	+22.4	+28.6	+32.4	+34.3	+34.6
Wind over. on truss E	−76.0	−30.7	−30.7	−30.7	−30.7	+30.7	+30.7	+30.7	+30.7	+30.7	+30.7
Wind over. on truss W	+76.0	+30.7	+30.7	+30.7	+30.7	−30.7	−30.7	−30.7	−30.7	−30.7	−30.7
Wind. on truss E	−13.5	−7.2	−19.2	−26.4	−28.0	+9.6	+26.4	+38.4	+45.6	+48.0	+48.0
Wind. on truss W	+13.5	+7.2	+19.2	+26.4	+28.0	−9.6	−26.4	−38.4	−45.6	−48.0	−48.0
Wind load E						+9.6	+26.4	+38.4	+45.6	+48.0	+48.0
Wind load W						−9.6	−26.4	−38.4	−45.6	−48.0	−48.0
Max	−719.4	−635.9	−690.6	−728.8	−730.0	+494.5	+501.9	+540.1	+725.7	+768.0	+771.8
Min	−308.0	−339.4	−354.3	−366.7	−364.8	+230.6	+180.5	+128.5	+259.0	+278.3	+280.3

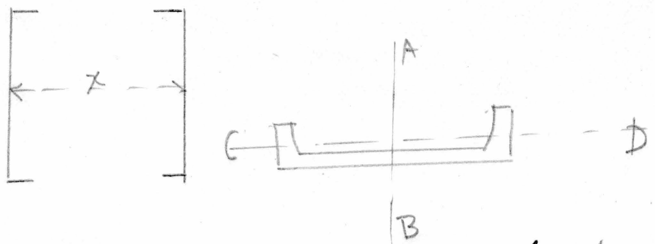
	Main Diagonals				Counter	Verticals				
	CD	EF	GH	IJ	KL	BC	DE	FG	HT	JK
Dead Load	+124.7	+92.5	+69.4	+19.4	+69.4	+38.5	−49.6	−34.5	−18.6	−18.6
Live load +	+112.0	+75.6	+63.0	+48.0	+33.6	+32.0	+30.8	+38.4	+44.0	+60.0
Live load −	−14.6	−23.6	−31.2	−46.0	−63.0	0	−58.8	−45.0	−36.0	−24.0
Impact ±	+21.4 −2.8	+15.4 −4.8	+13.7 −6.8	+11.2 −10.8	+8.5 −16.0	+61.0 0	+5.9 −11.3	+7.4 −8.6	+8.4 −6.9	+11.5 −4.6
Max.	+258.1	+183.5	+146.1	+78.6	0	+76.6	−12.9	+11.3	+33.8	+52.9
Min.	+107.3	+57.0	+28.8	+36.4	−9.6	+38.5	−119.7	−88.1	−61.5	−47.2

Intermediate Posts. (Cambria)

2nd Try 12" 30 lbs. (C41)
-119.7 Allowable stress $= 16000 - \frac{70l}{r} = 16000 - \frac{70 \times 44 \times 12}{4.28}$
$= 7370$

$\frac{-119.7}{7.37} = 16.3$ sq. in. 12" 30 lbs gives area of 8.82
$8.82 \times 2 = 17.62$ ∴ safe

Spacing.



Make Moments of two channels about two axes ⊥ to each other equal. $2 \times 103.2 = 2\left[3.9 + 8.82\left(\tfrac{1}{2}x - 0.651\right)^2\right]$, $x = 8"$.

3rd Try C41 12" 25 lbs.
-88.1 Allowable stress $= 16000 - \frac{70l}{r} = 16000 - \frac{50 \times 12 \times 70}{4.43} = 6.15$

$\frac{88.1}{6.15} = 14.3$ $2 \times 7.35 = 14.7$ ∴ use.

Spacing, calculated as above $= 10.07$.

4th Try 12" 20.5 lb. (C41.)
-62.0 allowable stress $= 16000 - \frac{52 \times 12 \times 70}{4.61} = 6152$

$\frac{62}{6.52} = 9.52$ $2 \times 6.03 = 12.06$.

Spacing calculated as above $= 10.44$

Adopted Spacing $10\frac{3}{8}$".

1st tension +77. Allowable stress 12,500 Use C41 12" 25 lbs, $a = 7.35$, $t = 0.39$
net section at pin, 7.35
$0.39 \times 6\frac{3}{4}$ $= 2.64$
4.71 $4.71 \times 2 = 9.42$ in² $\frac{77}{12.5} = 6.16$ in² ∴ use.

See design pin plates.

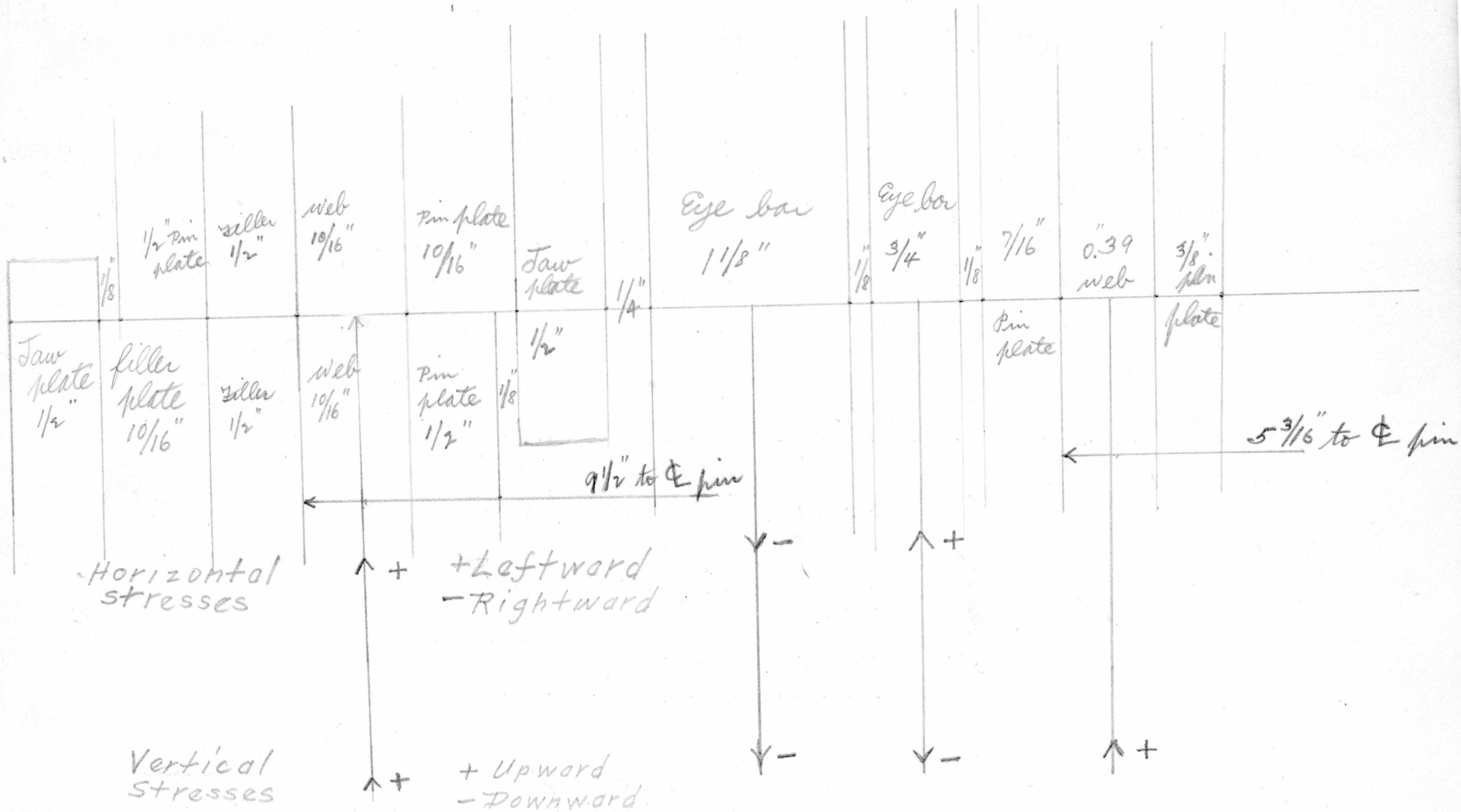
Diagonals and Suspenders.

Member	P (Kips.	No. Bars.	Area Reqd.	Bars Used.	Size Head.
$L_0 L_1$	494.5	4	31.0	8 x 1	18"
$L_1 L_2$	501.9	4	31.4	8 x 1	18"
$L_2 L_3$	549.9	4	34.4	9 x 1 1/8	19 1/2"
$L_3 L_4$	725.7	4	48.3	10 x 1 3/8	24 1/2"
$L_4 L_5$	768.0	4	48.0	10 x 1 3/8	24 1/2"
$U_1 L_2$	238.1	2	14.9	8 x 1	18
$U_2 L_3$	176.4	2	11.1	7 x 1 5/16	17
$U_3 L_4$	143.5	2	9.0	5 x 1	13
$U_4 L_5$	78.6	2	5.0	4 x 3/4	11 1/2
$U_5 L_6$	42.1	2	5.1	4 x 3/4	11 1/2
$M_1 U_2$	52.8	2	3.3	4 x 3/4	11 1/2
$M_2 U_3$	29.8	2	1.9	4 x 3/4	11 1/2
$M_3 U_4$	34.9	2	2.2	4 x 3/4	11 1/2

Size of Bars used must conform to pins used, as largest hole allowable in Bar must be as large as pin used.

Design of Pins.

The design of one pin will be given in detail and only the results will be given for the others.



shear in table below = resultant or component of full strength of member.

Horizontal Forces on Pin.

Shear	Dist.	Increment	Moment
+ 60.3	2.28	+ 137.3 max	+137.3 max.
− 53.4	0.92	− 49.1	+ 88.2
+ 6.9	5.98	+41.3	+129.5

Vertical Forces on Pin

+ 45.8	2.28	+104.5	+ 104.5
− 71.2	0.92	− 65.5	+ 39.0
− 12.1	0.98	− 11.9	+ 27.1
+159.9	5.00	+299.0	+326.1 max.

$$M = \sqrt{M_h^2 + M_v^2} = \sqrt{13.23^2 + 326^2} = 431$$

$$\frac{I}{c} = \frac{M}{S} = \frac{431}{25} = 17.2 \qquad \left\{ \frac{I}{c} = \frac{0.049d^{3}}{\frac{d}{2}} = 0.098\,d^3 \right\}$$

$$\therefore 0.098d^3 = 17.2 \qquad d^3 = \frac{17.2}{0.098} = 176 \qquad d = 5.6 \quad \therefore \text{use } 6'' \text{ pin}$$

Since we have chosen a 6" pin we must provide for sufficient bearing on the pin for upper chord.

$$S_{2\text{-}3} = 16000 - \frac{70 \times 32.567 \times 12}{7.77 = r} = 12480$$

$$S_{1\text{-}2} = 16000 - \frac{70 \times 34.176 \times 12}{7.77} = 12310$$

Full strength $U_2U_3 = 12480 \times \underset{=\text{area}}{58.5} = 730{,}000$

$U_1U_2 = 12310 \times 58.5 = 720{,}000$

$$\frac{730000}{22000} = 33.2 \qquad \frac{33.2}{12} = 2.77''$$

$$\frac{720000}{22000} = 32.7 \qquad \frac{32.7}{12} = 2.72''$$

Where 2.77" and 2.72" are dists. along pin on each side.

Size of other pins are shown on plates accompanying these notes.

Upper Chord Section

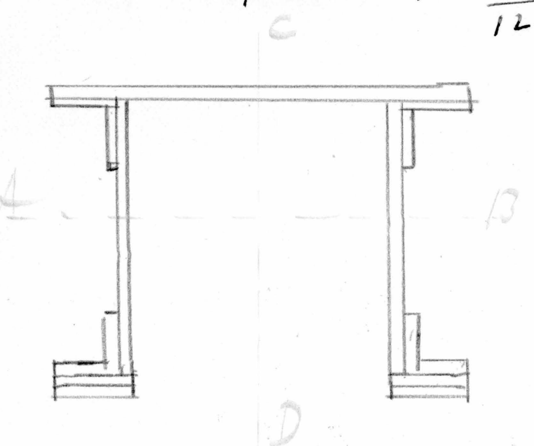
Estimated clearance for verticals and Eye-Bars 19"
Estimated depth .. Heads of Eye-Bars. 18"

Radius of gyration about Horizontal axis, $r = 0.4 \times$ depth (20 approx.)

$r = 0.4 \times 20 = 8.00$ allowable stress $= 16000 - \frac{70 l}{r}$

$= 16000 - \frac{70 \times 32 \times 12}{8} = 12{,}620$

Middle panel, $\frac{730}{12.62} = 57.8$.



Composition:

1 cover plate $28" \times \frac{1}{2}"$	$= 14$ in^2
4 ∠s $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{10}{16}"$	$= 13$ "
2 plates $18" \times \frac{10}{16}"$	$= 22.5$ "
4 flats $5" \times \frac{1}{2}"$	$= 10.$ "
	59.5 "

Radius of gyration about AB evidently least. (See Below.)

Calculation. Moment of:

1 cover plate $= \frac{1}{12} \times 28 \times (\frac{1}{2})^3$	$= 0.28$	in^4
$Ax^2 = 14(9.0 + 0.125 + 0.25)^2$	$= 1141.92$	"
4∠s $= 4 \times 3.64$	$= 14.55$	"
$Ax^2 = 13(9.125 - 1.06)^2$	$= 844.99$	"
2 webb plates $= 2 \times \frac{1}{12} \times \frac{5}{8} \times 18^3$	$= 607.50$	"
4 flats $4 \times \frac{1}{12} \times 5 \times \frac{1}{2} (\frac{1}{2})^3$	$= 0.83$	"
$Ax^2 = 10(9.125 + 0.5)^2$	$= 926.4$	"
	3536.47	in^4
Ecc. corr of $0.2" = 58.5$	$- 2.58$	
	3533.9	

True radius of gyration $r = \sqrt{\frac{M}{A}} = \sqrt{\frac{3533.9}{59.5}} = 7.77$.

True allowable stress $= 16000 - \frac{70 l}{r} = 16000 - \frac{70 \times 32 \times 12}{7.77} = 12{,}54$

$\frac{730}{12.54} = 58.25$ ∴ use this section for chord 4-5

3 to 4 -728.8 allowable stress $= 16000 - \frac{32.06 \times 12 \times 70}{7.77} = 12.53$

$\frac{728.8}{12.53} = 58.2$ ∴ use.

2 to 3 -690.6 allowable stress $= 16000 - \frac{32.50 \times 12 \times 70}{7.77} = 12.48$

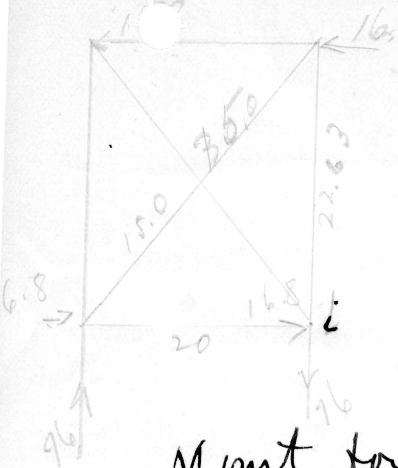
$\frac{690.6}{12.48} = 55.35$ ∴ use.

1 to 2 -635.9 $16000 - \frac{70 \times 12 \times 34.18}{7.77} = 12.295$ $\frac{635.9}{12.295} = 51.75$

End Post. Because of stresses brought on by effect of wind pressure, cover plate was increased in end post. Different thickness were tried and 3/4" found necessary. Corrected moment for this section about Horizontal neutral axis is 5831.8

assume i at middle of End Post.

$$M = \tfrac{1}{2}Pl = \tfrac{1}{2}\times 25\times 12\times 16.8 = 2520$$



$$S_1 = \frac{Mc}{I - \frac{Pl^2}{6E}} = \frac{2520\times 13}{5832 - \frac{719.4\times(12\times 19)^2}{6\times 26\times 10^6}}$$

$$S_1 = 5.74$$

A of Sect = 65.0

$\frac{719.4}{65.0} = 11.060$ allowable stress.

5.74

16.80

∴ OK. as limit of combined stresses must give unit stress < 19 Kips.

Mom't for this section calculated as above $= 9128.4$ in^4.

$r_1 = \sqrt{\frac{9128.4}{65}} = \cancel{63.6}\ 7.99.$ allowable stress $= 16000 - \frac{70\times 12\times 45.25}{7.99}$

$= 11.125$ Kips/in^2.

Total strength $= 11.125\times 65 = 723$ Kips. ∴ use.

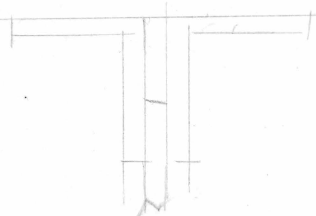
Mom't of section of upper chord ~~for~~ about CD (see Fig above). was found to be $I' = 5465.6$. ∴ First Mom't used.

Portal Bracing.

Each brace takes ½ the shear, or 38 Kips.

Used 2 ∠s $3\tfrac{1}{2}'' \times 3\tfrac{1}{2}'' \times \tfrac{1}{2}$ Back to Back laced by $\tfrac{1}{2}'' \times 2''$ lacing

Least $r = 1.66$.



allowable stress $= 16000 - \frac{70\times 12\times 15}{1.66}$

$= 8400$

$\frac{38}{8.4} = 4.54$ in^2.

4 ∠s $3\tfrac{1}{2}'' \times 3\tfrac{1}{2} \times \tfrac{1}{2}''$ gives, $4\times 3.25 = 13$ in^2

∴ safe.

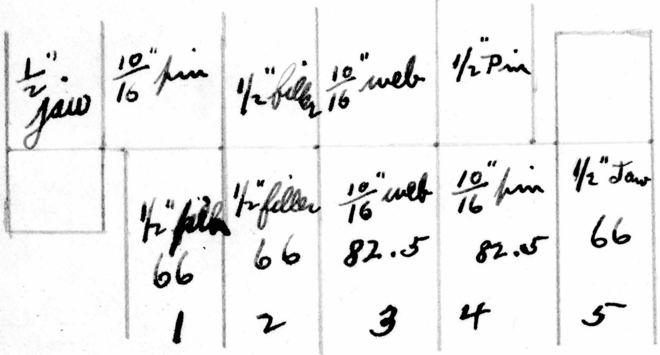
Pin Plates.

The table shown below gives the division of stresses at the different panel points in the upper chord.

	Gross area	Stress U_1-U_2	12.48 U_2U_3	U_3-U_4 12.53	U_4-U_5 12.62
1/2 Cover plate	7 in^2 ×12.29	86.03	87.5	87.7	88.4
1 Upper L	3.25	39.95	40.6	40.8	41.1
1 web plate	11.25	138.30	140.5	141.0	142.2
1 Lower L	3.25	39.95	40.6	40.8	41.1
2 Flats	5.00	61.45	62.4	63.7	63.1
Total	29.75	365.68	371.6	373.0	375.9

The values 12.29, 12.48 &c come from formula for allowable compression $S = 16000 - \frac{70l}{r}$. As the stresses are very nearly the same for each point, we will use the largest, namely that at U_4-U_5, in our discussion for all points, thus making them uniform.

The web takes 142.2 − 82.5 more stress than it gets from pin bearing = 59.7, which comes from the 1/2" filler leaving 66 − 59.7 = 6.3 which comes from all other plates in proportion to their thickness. This adds to these plates (shown in diagram below) from left to right 1.15, 1.15, 1.43, 1.43, 1.15.



Total stresses

In ① = 66 + 1.15 = 67.15
② = 66 + 1.15 = 67.15
③
④ = 82.5 + 1.43 = 83.93
⑤ = 67.15

⑤ requires $\frac{67.15}{4.510}$ = 15 rivets use 7 on bottom 14 thru ① in double shear bear value = 14 × 8.2 = 115.0

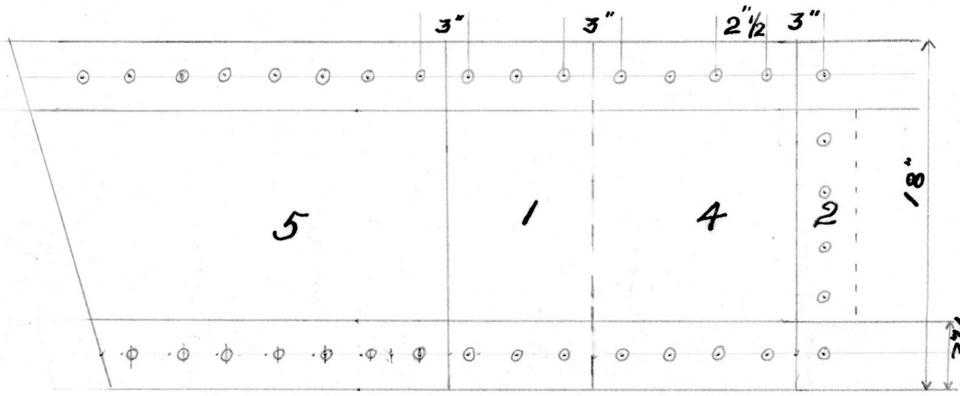
Combined stresses ⑤ and ① = 67.15 + 67.15 = 134.3 − 115 = 19.3 = additional stress carried by rivets in single shear $\frac{19.3}{4.510}$ = 5 rivets through ① ∴ plate ① is lengthened to catch 6 rivets, 3 in each angle ~~in single shear~~.

Combined stresses ①, ④ + ⑤ 67.15 + 67.15 + 83.93 = 218.23
Bear value 20 rivets in double shear = 20 × 8.2 = 164.00

54.23

$\frac{54.23}{4.51} = 12$ rivets $\therefore$ (4) is extended to catch 8 rivets, ~~5~~ 4 in each angle.

Combined stresses filler plate to web $= 59.7 + 1.43 + 1.15 = 62.28$ $\frac{62.28}{4.51} = 14$ rivets. Filler plate is extended for appearance, and has 4 rivets between angles for same reason.



The above diagram gives the location of plates and rivets at panel points in upper chord. The numbers shown on the drawing refer to the same pieces in the diagram on the previous page.

Suspenders.

1st Panel. Top. $\frac{52.8}{12.5} = 4.22$ in² or 2.11 in² for each channel.

Use 8" 16.25 lb. channels. area $= 4.78$ in² $t = 0.21$

Less rivets $3 \times 1 \times 0.21 = \frac{0.63}{4.15}$ $\therefore$ net area.

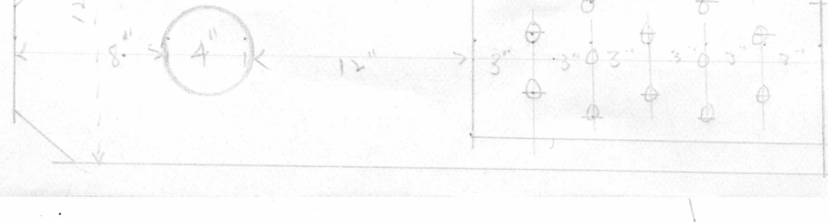
Full strength $= 4.15 \times 12.5 = 51.9$ Kips.

Pin bearing $= \frac{51.9}{2 \times 4 \times 22} = 0.295 = t$ $\therefore$ use $\frac{1}{2}$" [4" pin $\frac{1}{2}$" plate on each side of channel see Plate]

~~Bearing~~ Shear 3/4" rivet in single shear $= 2.651$ Kips.

Full strength gives $\frac{51.9}{2.651} = 20$ rivets. Use 12 in each channel.

Bearing of rivets. $= 12 \times 3375 = 40.7$ Kips. $\therefore$ safe.



Floor Beam Con to 1st suspenders & Posts.

Shear at end of floor beam 52.8 7/8" rivets. value in shear = 4550 lbs.

$\frac{52.8}{4550}$ = 120 rivets. Use 8 on each side I-Beam

Bearing in 1/2" metal = 16 × 7.87 = 126 Kips ∴ ok.

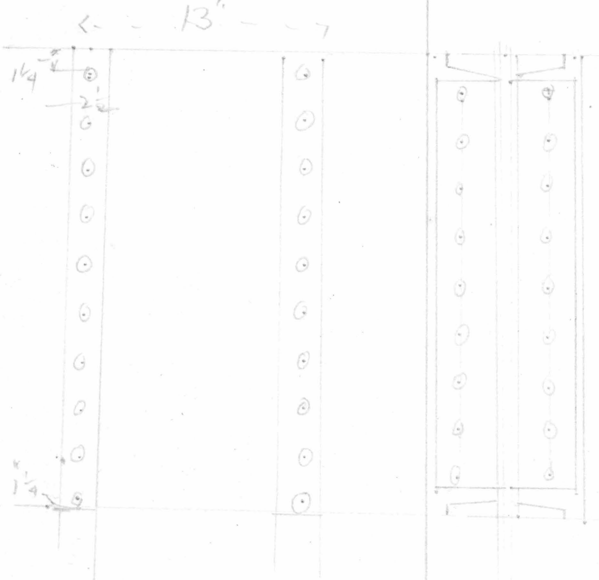
Plates transferring shear between channels, 1/2 of which is carried from floor-beam side to outside channel.

1/4" 26"×13" plate ok. Rivets. 3/4" in single shear = 2651

$\frac{26.4}{2651}$ = 10 rivets. Bearing of rivets 20×3375 = 67.5 Kips. ok.

Strength of ~~channels~~ plates. gross area = 4.78
less holes = .60
4.15 = net area

4.15 × 2 = 9.30   9.30 × 15 = 139 Kips ∴ ok.



Strut to end post figured same section, bearing and no. of rivets as top of 1st suspender ∴ used as shown.

Second Post.

Net area of channel = 5.00 × allowable stress, 5 × 12.5 = 62.5

5 × 12.50 = 32.7

2 pin plates. 7/16 × 6 = 2.62   1.5 × 12.5 = 18.8

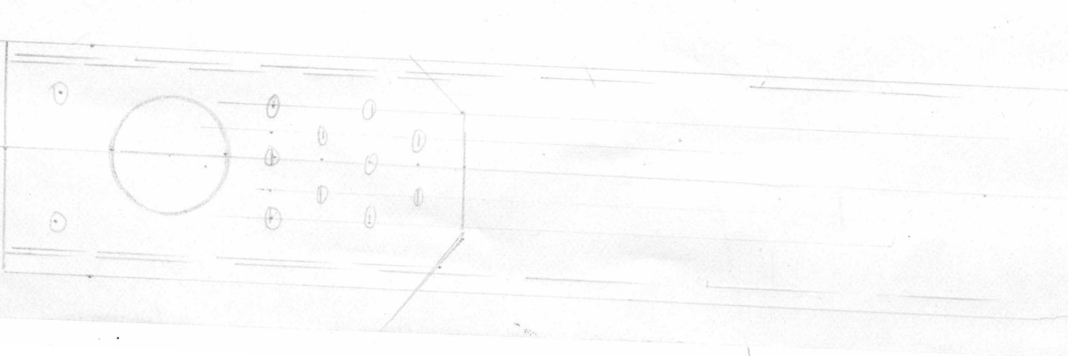
6/16 × 4 = 1.50

Bearing, allowed 22 K per in². $\frac{62.5}{22}$ = 2.84, $\frac{32.7}{22}$ = 1.49, $\frac{18.8}{22}$ = 0.85

Total Bearing = 5.18 in²

~~area channel~~ ~~4.78~~ in² Bearing 3/4" in double shear = 5.301

$\frac{32.7}{5.301}$ = 6 rivets. $\frac{18.8}{5.301}$ = 4 rivets.



Bottom 2nd Post.

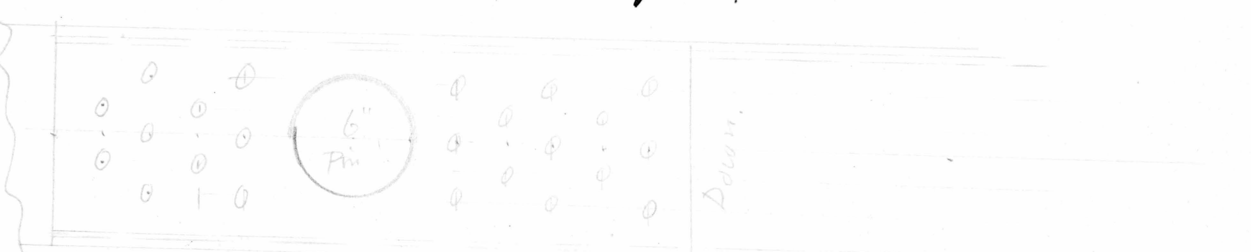
Cross area web. 7.35
2 pin plates.
$\frac{1}{2} \times 12$ 6.00
$\frac{1}{2} \times 12$ 6.00
19.35
Less pin hole (6 × 1.39 = 8.34
net area. 11.01

$11.01 - 7.35 = 3.66$ $\frac{3.66}{7.35} = 50\%$

∴ > 40% in excess of Body of channel.

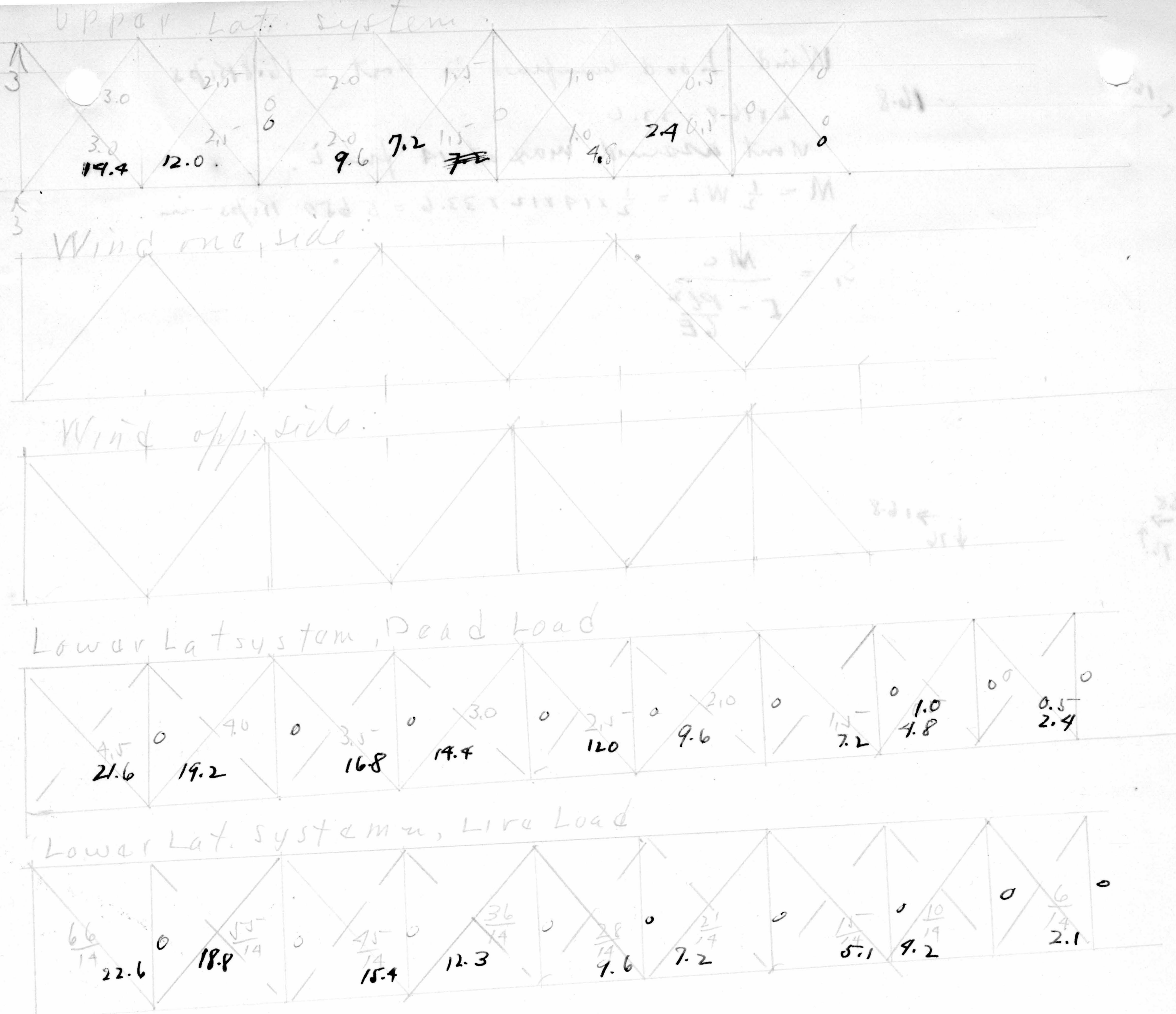
Net area channel = 5.00, Total strength = $5 \times 12.5 = 62.5$
$\frac{1}{2}" \times 6"$ Plate = 3.00 " = $3. \times 12.5 = 37.5$
$\frac{1}{2}" \times 6"$ " = 2.00 " = $2 \times 12.5 = 25.0$

$\frac{62.5}{22} = 2.84$ in $\frac{37.5}{22} = 1.70$ in $\frac{25}{22} = 1.13$ in. ∴ Bearing OK.
$\frac{37.5}{5.301} = 7$ rivets. $\frac{25}{5.301} = 5$ rivets. Used 13 below pin for tension, and above pin same and top for comp.



At Bottom Pin.

Post V_2L_2 (3rd Post.) Allowable stress $= 16000 - \frac{70 \times 12 \times 44}{4.28}$
= 7.38 Kips-in²
Gross area = $2 \times 882 \times S = 2 \times 8.82 \times 7.38 = 130.5$ = Total strength.
$\frac{130.5}{22.000} = 5.93$ □" for bearing. $\frac{5.93}{6} = 1"$ along pin. ∴ t = ½" each channel.
Net area channel = 5.82 in²
∴ only 1 pin plate 7/16"
Plate 7/16 × 6" = 2.625 strength = $5.82 \times 7.38 = 43.0$
$= 2.65 \times 7.38 = 19.4$.
$\frac{19.4}{2.65} = 7.6$ rivets. 8 rivets (3/4") used for top
Bottom same as 2nd post at all panels except suspender. (See 1st Top.)
4th Post. Stress = $16000 - \frac{70 \times 12 \times 50}{4.45} = 6.55$ Kips.
Full strength = $2 \times 6.55 \times 7.35 = 96.2$ K. $\frac{96.2}{22} = 4.36$ in²
$\frac{4.36}{2 \times 6} = 0.363$ in ∴ only 1 pin plate 7/16"
8-3/4" rivets used
Bottom same.
5th Post. Same as previous.



Dead Panel Load $150 \times 32 = 4.8K$
Live " " $150 \times 32 = 4.8K$

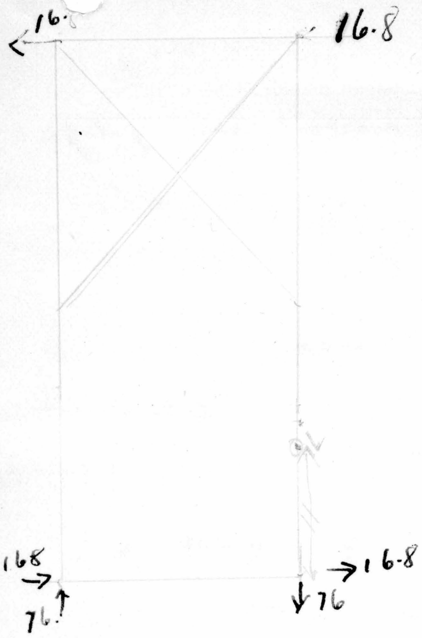
Lower Lat. System. Max comp $= 22.6 + 21.6 = 44.2$ Kips.
Allowable comp $= 16000 - \frac{70 \times 12 \times 10}{1.06} = 8150$

$\frac{44.2}{8.15} = 5.4 \text{ in}^2$ ∴ use 2 $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{1}{2}$ ∠s back to back.

Upper Lat System: Generally adopted system is pair of ∠s back to back as struts and system of single ∠s tied together by lacing ~~and~~ or batten plates. See Plates.

Cambe.

Allowed as per specification.



Wind Load transferred to Post = 16.8 Kips

$2 \times 16.8 = 33.6$.

Mom't assumed max at 14' from i.

$M = \frac{1}{2} WL = \frac{1}{2} \times 14 \times 12 \times 33.6 = 5650$ Kips-in.

$$S_1 = \frac{Mc}{I - \frac{Pl^2}{6E}}$$

End Bearings. (Roller.)

The expansion bearing used is the standard one designed by George S. Morison, shown on page 225, Part III Roofs and Bridges, and also in the drawings accompanying these notes.

$p = 600\,d$ where p is pressure per linear in and d is the diameter of a roller. $\therefore p = 600 \times 6 = 3600$ lbs/lin. in.

We used 6 rollers which are 35" long. Hence $6 \times 35 = 210$"

$210 \times 3600 = 76.7$ Kips pressure upheld by rollers.

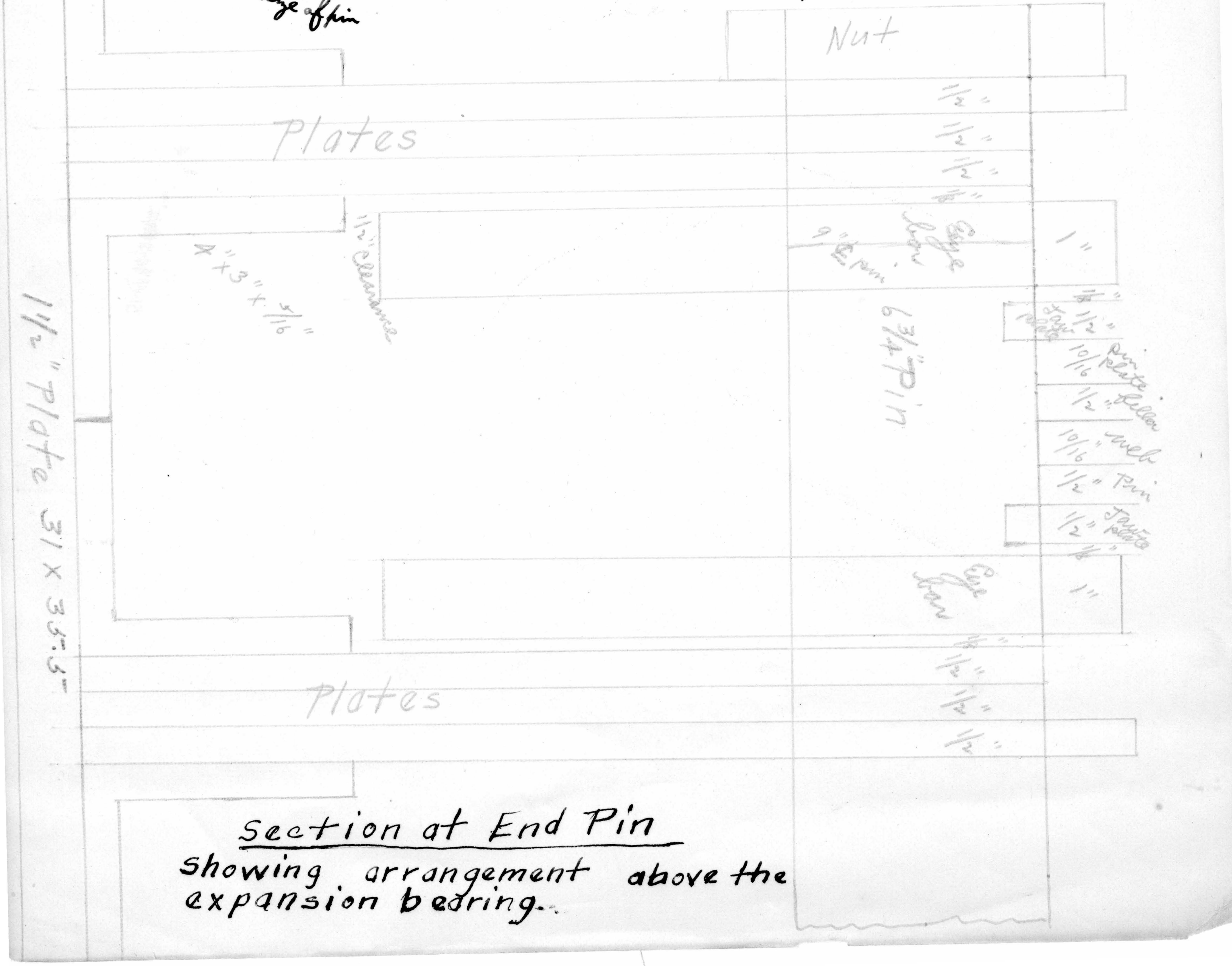
The vertical pressure exerted on these rollers is 587 Kips. $\therefore$ We are safe in using 6 rollers.

This pressure is transfared from the pin at this point thru 4 sets of plates distributed across the pin and thence on to the rollers. To figure the size of these plates we use full strength of the rollers.

For one set of ~~these~~ plates $\frac{767}{4} = 192$ ~~lbs~~ Kips

$\frac{192}{22 \times 6\frac{3}{4}} = 1.29"$ $\therefore$ use 3 - 1/2" plates.

($6\frac{3}{4}$ = size of pin)



Section at End Pin

showing arrangement above the expansion bearing.

Sidewalk. (See plate)

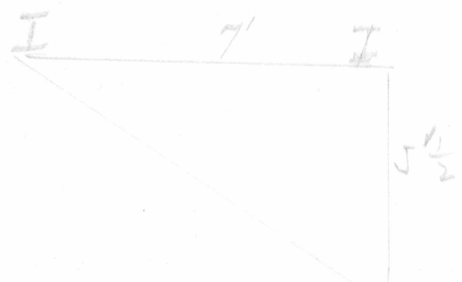
Stringers: 3" slab at 150 lbs. per cu ft. = 37.5 lbs per sq ft.
$3 \times 37.5 = 112.5$ per linear ft.

8" I-Beam = 18.
Channel = 5
$\overline{135.5}$

Live Load at 100 lbs per sq. ft. = $3 \times 100 = 300$.

L.L.M. = $\frac{1}{8}wl^2 = \frac{1}{8} \times 300 \times 16^2 = 9600$ lbs ft = 115,000 lbs-in.
D.L.M = $\frac{1}{8}wl^2 = \frac{1}{8} \times 135.5 \times 16^2 =$ 43.400 " "
158.4 Kips-in = total M.

$\frac{158.4}{13} = 12.2 = \frac{I}{c}$. ∴ Use 8" 18 lb. I-Beam $\frac{I}{c} = 14.2$



½ Load: Floor = 56.25
I-Beam = 18.
Channel = 5
Railing = 10
89.25 lbs.

'C' D.L. Moment about 'C' = $89.25 \times 16 = 1428$ lbs-ft × 7
L.L. " " " = $2\frac{1}{2} \times 100 \times 16 = 4000$ " × 7
Total = 5428 " × 7
= 38000 lbs-ft.

Held by rivets as per plate.
$S_1 \times 6 \times 5\frac{1}{2} + 16 S_1 \times 1 = 49 S_1 = 38000$ ∴ $S_1 = 775$ lbs. per rivets.

1" rivets gives 7060 lbs in single shear ∴ use ok.

Analysis of Weight. 1/2 Truss

Upper Chord.

1 Cover plate $28'' \times 1/2'' = \frac{14}{144} \times 490 =$ 47.6
4 Angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times 1/2''$ From Cambria = 44.4
2 Plates $18'' \times 5/8'' = \frac{18}{144} \times 490 =$ 61.2
2 Flats $5'' \times 1''$ = From Cambria = 34.0
187.2 lbs. per linear foot.

Lengths
Chord 1 = 45.25
2 = 34.17
3 = 32.56
4 = 32.06
1/2 5 = 16.00
160.04 ft

Plates
Jaw $\frac{1}{2} \times 16 \times 27 = 0.154$
Pin $1/2 \times 16 \times 27 = 0.154$
Pin $1/2 \times 16 \times 37 = 0.206$
Filler $1/2 \times 11 \times 40 = 0.153$
0.667 ft³ 1 plate
$0.667 \times 490 \times 18 = 5880$ lbs ~~ft³~~

$160.00 \times 187.2 = 30,000$ lbs ft.
Extra for end post covering plate $= 28'' \times 1/4 \times 45.25$
= 1075 lbs ~~ft~~

Tie plates $9 - 1/2 \times 16 \times 28 = 1.36$
$3\frac{1}{2} - 1/2 \times 28 \times 28 = 0.95$
Lacing $= 2\sqrt{2} \times 160 \times 1/2 \times 2 = 0.55$
$2.86 \times 490 = 1400$ lb. ~~ft³~~

Pins $4\frac{1}{2} \times 30 \times \pi \times 9 \times 0.353 = 1345$ lbs ~~ft³~~

Main Compression member = 30,000 lbs
Extra for End Post Covering plate = 1075 "
Plates = 5880 "
Ties plates = 1400 "
Pin = 1345 "
Total = 39,700 "

Lower Chord.

Eye bars $8 - 8'' \times 1'' \times 32' \times 12 = 24600$
$4 - 9'' \times 1.12 \times 32 \times 12 = 14680$
$6 - 10'' \times 1.37 \times 32 \times 12 = 30680$
$69960 \times 0.353 = 24700$ lbs
Pins $5\frac{1}{2} \times 35 \times \pi \times 12 \times 0.353$ = 2540 "
27940 "

Verticals

$36' + 54' + 56' = 146 \times 25 \times 2 = 7300$ lbs
$48' \times 30 \times 2$ = 2880 "
Lacing $\frac{2\sqrt{2} \times 178 \times \frac{1}{2} \times 2}{1728} \times 490 = 299$ "
Plates $4 \times 4 - 12 \times 1/2 \times 27 = 2590$
$4 \times 2 - 12 \times 1/2 \times 56 = 2690$
$4 \times 2 - 12 \times 1/2 \times 22 = 1050$
$4 \times 2 - 12 \times 1/2 \times 35 = 1680$
$8010 \times 0.353 = 2830$ "
13309 "

Sub Verticals.

8"-16.25 lb. channels.
24' (End Post)
20'
20'
26'
29'
15' (1/2 counter)
134'
$134 \times 2 \times 16.25 = 4360$ lbs.
2470 "
6830 "

6" 13.00 lb chanls.
24
27
29
15
95'
$95 \times 13 \times 2 = 2470$ lbs.

~~Total~~

Lacing $\frac{3}{32} \times \sqrt{2} \times 134 \times \frac{1}{2} \times 2 = 150$ lbs.
$\frac{1}{2} \times 2\sqrt{2} \times 95 \times \frac{1}{2} \times 2 =$ 80 "
230 "

Plates. $3\frac{1}{2} \times 2 \times \frac{1}{2} \times 13 \times 25 = 1140$
$3\frac{1}{2} \times 2 \times \frac{1}{2} \times 12 \times 25 = 1100$
2690 see above
1050 " "
1680 " "
7660
$7660 \times .353 = 2700$

Total weight Sub verts. $6830 + 230 + 2700 = 9760$

Total " Verticals. 13309

23069

Plus pins. $4\frac{1}{2} \times 26 \times \pi \times 4 \times 0.353 =$ 520

23589

Diagonals.

4 - 8" x 1" x 21' = 672 lbs.
2 - 4" x ¾" x 37' = 222
4 - 7" x 1" x 31' = 870
2 - 4" x ¾" x 37' = 222
6 - 5" x 1" x 34' = 1020
2 - 5" x 1" x 36' = 360
2 - 5" x 1" x 36' = 360
2 - 4" x ¾" x 35' = 210

3946 x 12 x 0.353 = 16,750 lbs.

Floor Beams and Stringers.
(½ system)

Floor Beam - $8\frac{1}{2} \times 9\frac{1}{2} \times 80 = 6450$

$3\frac{1}{2} \times 25 \times 144 = 12600$

19050

Paving.

Roadway

$8\frac{1}{2} \times \frac{4\frac{1}{2}}{12} \times 144 \times 150 = 68700$

$8\frac{1}{2} \times \frac{6}{12} \times 144 \times 100 = 61200$

129900

Sidewalk.

$5 \times \frac{3\frac{1}{2}}{12} \times 150 \times 144 = 30,000$

Gussett Plates Sidewalk

$8\frac{1}{2} \times \frac{1}{2} \times 15 \times 0.353 \times 12 = 2740$

I Beams $2 \times 144 \times 18 = 5180$

7920

½ Upper Lateral System.

Portal. 1 x 30 x 11.1 = 333
Panels. 7 x 26 x 11.1 = 2000
Struts. 8 x 20 x 19.6 = 3130
Plates 8 x ½ x 17 x 24 = 575
9 x ½ x 11 x 22 = 384

½ L.L.S. 9 x 20 x 19.6 = 3530
plates. 9 x 4.5 = 1710
Total 11,662

Total Weight.

Upper Chord.	39700 lbs.
Lower "	27940
Verticals.	23589
Diagonals	16750
Floor Beams + Strs.	10058
Paving.	160900
Sidewalk.	7920
Lateral Systems	11662
	298519

Total Bridge.

4 x 298519 = 1,194,076 lbs.

Estimated. 3995 x 288 = 1,155,000

too high by 3.26%.

Impact Factor over 5% of total load. ∴ ok.

Respectfully submitted.

Bm H Farquhar