Design 70ft Plate Girder Bridge.

Depth = 6' 4", Width 8ft.

Cooper's Class E-50 Loading.

Specifications as per plate girder bridge, pp. 146, Merriman's "Roofs & Bridges" part III.

Maximum Shears -

Section	0	5'	10'	15'	20'	25'	30'	35'
L. Load	139	119.6	103.8	89	75.4	62.5	50.	38.1
Impact	98.	84.	72.8	62.5	52.9	43.9	35.1	26.8
D. Load	28.	24.	20.0	16.0	12.0	8.0	4.0	0
Maximum	265.	227.6	196.6	167.5	140.3	114.4		

Maximum Moments -

Section	0	5'	10'	15'	20'	25'	30'	35'
wheel	2	2	2	3	3	4	12	13
Live L.	0	600	1067	1463.	1747	1940	2090	2133.
Impact	0	421	748	1028.	1230	1360	1466	1496.
Dead L.	0	130	240.	330	400.	450.	480.	490.
Maximum	0	1151.	2055.	2821	3377	3750	4036	4119. kip feet.

Dead Load:

Wt. $\frac{1}{2}$ girder + $\frac{1}{2}$ lateral bracing = 39560 lbs

" track @ 235 lbs per lin. ft. = 16450

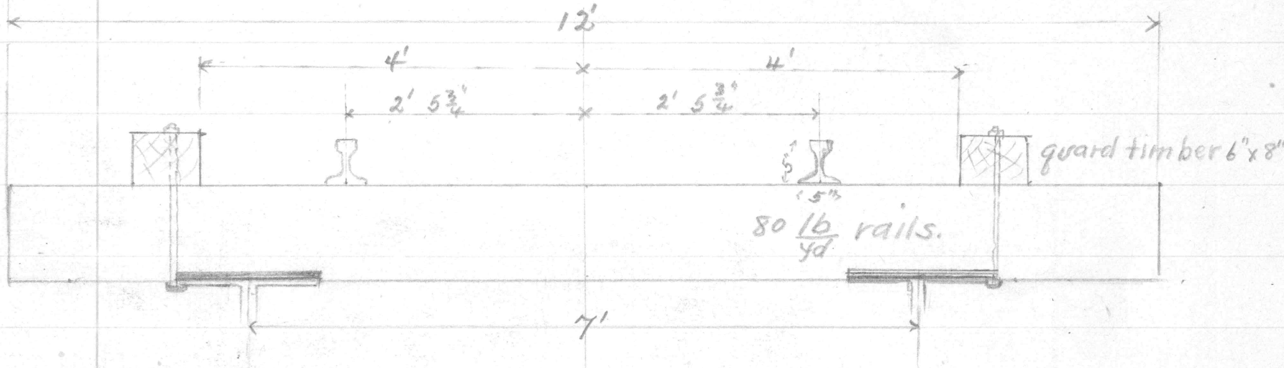
$\overline{56000}$ lbs

Coefficient of impact = 0.702

Wooden Floor:

Ties notched over flanges not less than $\frac{1}{2}$".

Alternate ties bolted to girder and guard rail.



Ties 12' long, breadth 9", depth = 10".

Greatest concentrated load per wheel = 30000 lbs - distributed over 3 ties; per tie = 10,000 lbs

Assume track 450 lbs per linear foot.

Each tie carries $1\frac{1}{4}$' of track.

$\therefore$ Live = 10,000

Impact = 8000

Dead = $\underline{280}$

18280 lbs per track, per tie.

Max. moment = $18280 \times 42" - 18280 \times 29\frac{3}{4}" = 223000$ lbs-inches.

b = breadth, d = depth of ties. Take $S = 2000 \frac{lbs}{in^2}$.

$223000 = 2000 \frac{bd^2}{6}$ $\quad \therefore bd^2 = 670$

Take bearing at 400 $\frac{lbs}{in^2}$.

$\therefore$ bearing area = $\frac{18280}{400}$ = 45.7 in^2.

Use 80 lb rail, $d' = 5"$, $b' = 5"$

45.7/5" = 9" = width of ties.

Since $bd^2 = 760$ $d = \sqrt{\frac{760}{9}}$ = 9.2 net depth of tie.

Use gross depth of 10", & 11" ties at center of girder.

Space ties in clear 5".

At $3\frac{3}{4}$ lbs board measure, wt. per tie = 370 lbs., giving 318 lbs per foot, for ties.

Assume rails, splices, etc at 160 lbs per foot.

— Dead load —

Wt. 1 girder (assumed) + lat. bracing	= 39600 lbs
Track at 235 lbs. per ft =	16450
	56050

Shear at ends, = 28000 lbs.

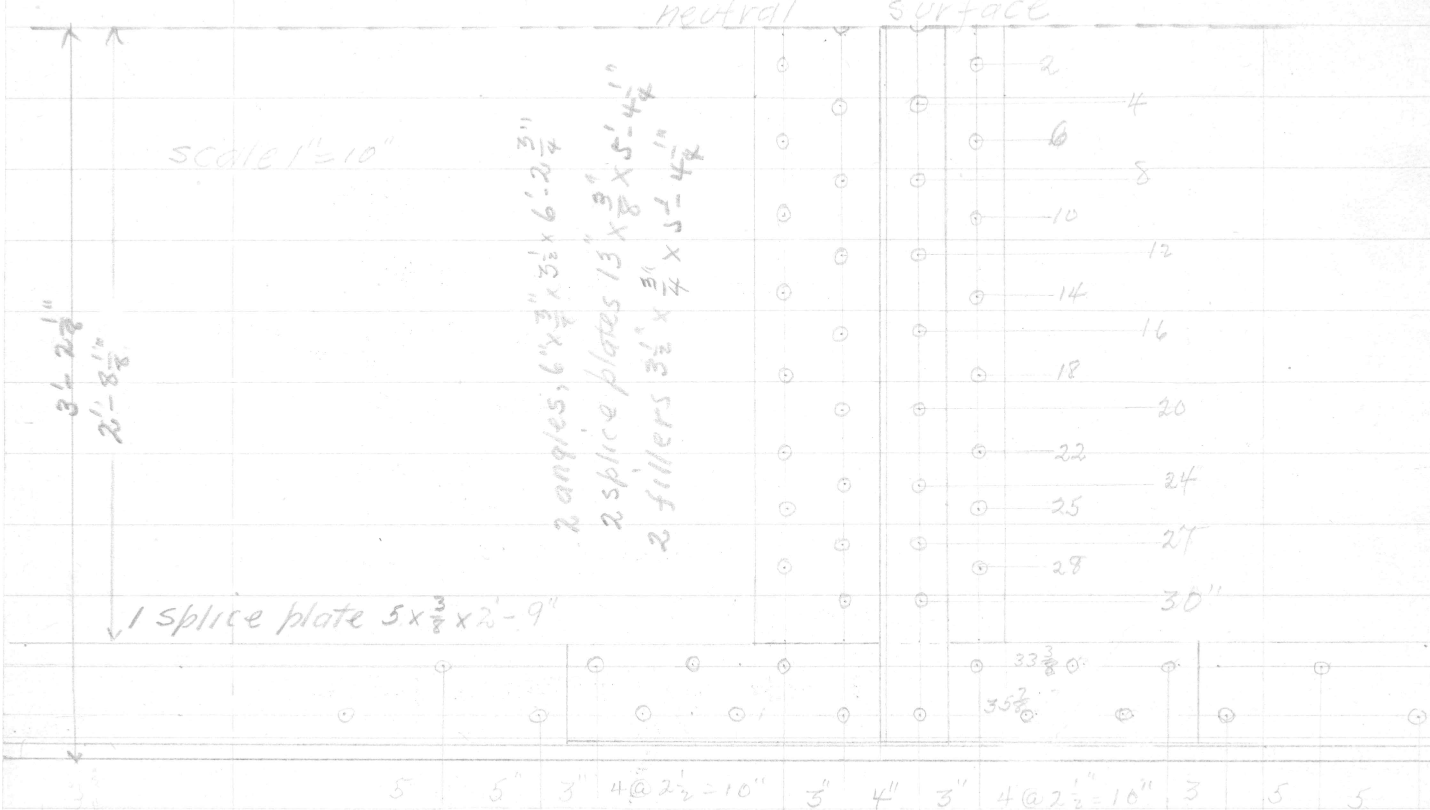
Web:

- Shearing stress @ 12000 $\frac{lbs}{in^2}$

$\frac{265000}{12000} = 22.08^{\square''}$ = Required area of web, net.

A $\frac{7}{16}''$ web will allow 25 rivets, pitch 3.04"
Use 2 plates, 76" x $\frac{7}{16}''$; each 430" long.
giving one web splice, at center of girder
and allowing 10" beyond center of bearings at
each end of girder.

- Web Splice -



The web will be made up of 2 plates, each 430" long x $\frac{7}{16}$" x 76", and spliced at center of girder. The splice plates and rivets will be designed to develop full strength of web to resist bending moment at section, only.

Unit stress outer fiber = 17000 $\frac{lbs}{in^2}$, and since web resists $\frac{1}{6}$ of moment, for gross web:

$$\frac{1}{6} \times 17000 \times \frac{7}{16}" \times 76" \times 76" = 7160000 \text{ lbs in}$$ resisting moment gross section of web.

+ for lower half $\frac{7160000}{2}$ = 3580,000 lbs-in.

To deduct tensile stress on acct. rivet holes:

At outer fiber, for each rivet hole

$17000 \times 1'' \times \frac{7''}{16} = 7440$ lbs., and for one at distance y, deduction $= 7440y/38''$

Moment this stress about neutral axis $= 7440\,y^2/38$, & for all holes $= \frac{7440}{38} \Sigma y^2$.

Per Figure; inner row of rivets:

$4^2 = 16$

$8^2 = 64$

$12^2 = 144$

$16^2 = 256$

$20^2 = 400$

$24^2 = 576$

$27^2 = 729$

$30^2 = 900$

$(35\frac{7}{8})^2 = \underline{1285}$

$\Sigma y = 4370$ lbs

$\therefore \frac{7440}{38} y^2 = \frac{7440}{38} \times 4370 = 856000$ lbs-in.

$3580000 - 856000 = 2,724,000$

$\therefore$ Resisting moment net section half of web, resultant:

$= 2724000$ lbs-in.

With bearing unit stress of 24000 $\frac{\text{lbs}}{\text{in}^2}$ on rivets, the allowable bearing for $\frac{7}{8}''$ rivet in $\frac{7}{16}''$ plate

$24000 \times \frac{7}{8}'' \times \frac{7}{16}'' = 9190$ lbs.

For splice plates we will use $\frac{3}{8}$" plates. Bearing of rivets greater in both plates than in web. The bearing in web will determine number of rivets in splice, since $\frac{7}{8}$" rivet in double shear at $12000 \frac{lbs}{in^2}$ gives 14430 lbs.

Moment of bearing of rivets:

Since outer row of rivets are $35\frac{7}{8}$" from neutral surface, the moment of bearing of each rivet at distance $y = 9190\, y^2/35\frac{7}{8}$. Summing up for both rows:

Inner row.		Outer row.	
$4^2 =$	16	$2^2 =$	4
$8^2 =$	64	$6^2 =$	36
$12^2 =$	144	$10^2 =$	100
$16^2 =$	256	$14^2 =$	196
$20^2 =$	400	$18^2 =$	324
$24^2 =$	576	$22^2 =$	484
$27^2 =$	729	$25^2 =$	625
$30^2 =$	900	$28^2 =$	784
	3085		2551

$$\begin{array}{r} 3085 \\ 2551 \\ \hline \end{array}$$

$\therefore\ \Sigma y = 5636$

$\frac{9190}{35.875} \times 5636 = 1441000$ lb-ins. = total bearing moment of rivets in web.

To find no. rivets required in splice plates on vertical legs of flange angles:

2724000 − 1441000 = 1283000 lb-ins, to be taken by splice plates on flange angle.

Use 2 plates, $5'' \times \frac{3}{8}''$.

For rivets in these plates, resisting moment,

$\Sigma y^2 = 1283000 \times \frac{35.875''}{9190} = 5010 \text{ in}^2.$

Since

$(33.375'')^2 = 1112.$

$(35.875'')^2 = 1285.$

We may place 2 rivets in lower row and 3 in upper row on each side of section, making

$\Sigma y^2 = 6079 \text{ in}^2.$

Also add one rivet, which, together with surplus resisting moment of above rivet, can resist flange increment under splice, making six each side of section.

– Web Stiffeners –

In order to support horizontal leg of flange angles, we will use $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles, with filler plates $\frac{3}{4}''$ thick under them at web splice and at ends of girder.

Max. shear at support = 265000 lbs, and taking s at 15000 $\frac{lbs}{in^2}$

$\frac{265000}{15000} = 17.66 \square''$ required to transmit shear to support.

Six angles, $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ give $20.58 \square''$, and this number will be used, since 4 angles do not give enough.

Bearing value of rivets governs = 9190 lbs.

$\frac{265000}{9190} = 28.8$ rivets required

The same number per angle will be used as at web splice = 19.

Spacing of stiffeners to be determined on drawing.

– Composition of Flanges –

Max. bending moment at center of girder = 4119. kip-ft.

= 49,428,000 lbs-inches.

Approximate effective depth = $76'' + .25'' - 1.5'' = 74.75''$.

Take unit tensile stress @ $17000 \frac{lbs}{in^2}$, and assume 12% gross web section as effective flange area.

Take formula, as per Part III p. 152 :

$$\text{Area of flange} = \frac{M}{sh_1} - \frac{th}{(6)}$$

$$A = \frac{49428000}{17000 \times 74.75} - \underset{(4.0\square'')}{0.12 \times \frac{7''}{16} \times 76''} = 34.86\square'', \text{ required.}$$

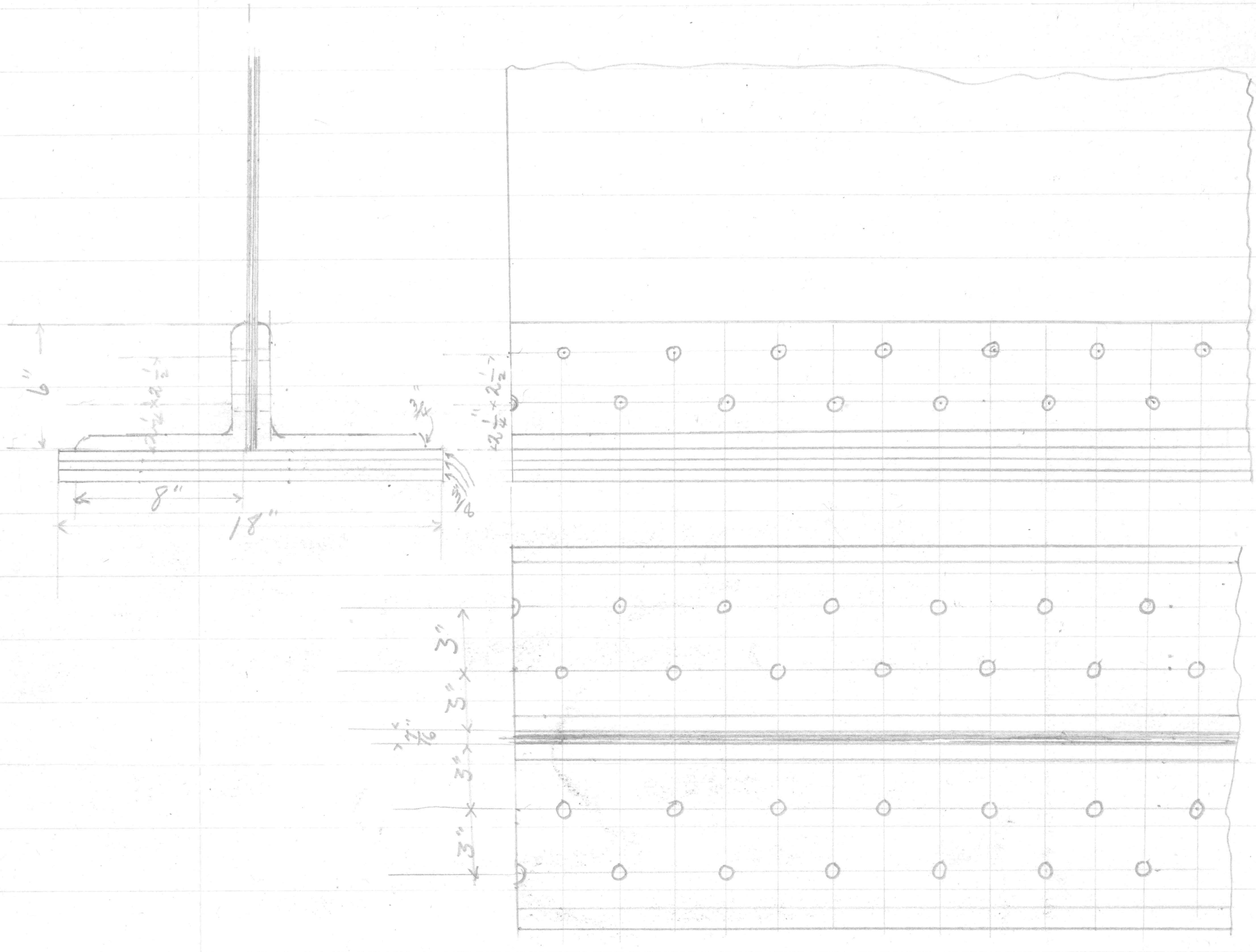
Take angles and plates as follows:

2 angles, $8'' \times 6'' \times \frac{3''}{4} = 2(9.94\square'' - 1.50\square'')$	=	$16.88\square''$
3 plates, $18'' \times \frac{3''}{8} = 3(6.75 - 0.75)$	=	18.00
Total section, net,	=	$34.88\square''$

Take 2 rows of rivets in each leg of angles, adjacent rows to stagger.

Calculated true value of h_1 at center of girder = 75.4'', and this gives $A = 34.56\square''$.

Therefore above section is sufficient.



2 angles, 8"x 6"x $\frac{3}{4}$"

3 plates 18"x $\frac{5}{8}$"

Max. Shears

Section	0'	5'	10'	15'	20'	25'	30'	35'
Live L.	139	119.6	103.8	89	75.4	62.5	50.0	38.1
Impact	98	84.0	72.8	62.5	52.9	43.9	35.1	26.8
Dead L.	28	24.0	20.0	16.0	12.0	8.0	4.0	0.0
Total	265	227.6	196.6	167.5	140.3	114.4	89.1	64.9

Pitch of rivets connecting flange and web:

Vertical component of flange stress = 1035 lbs per linear inch.

Increment flange $S' = \frac{V}{h_1} \times \frac{\text{area flange}}{\text{area flange} + \text{Equivalent flange area of web } (4.04)}$

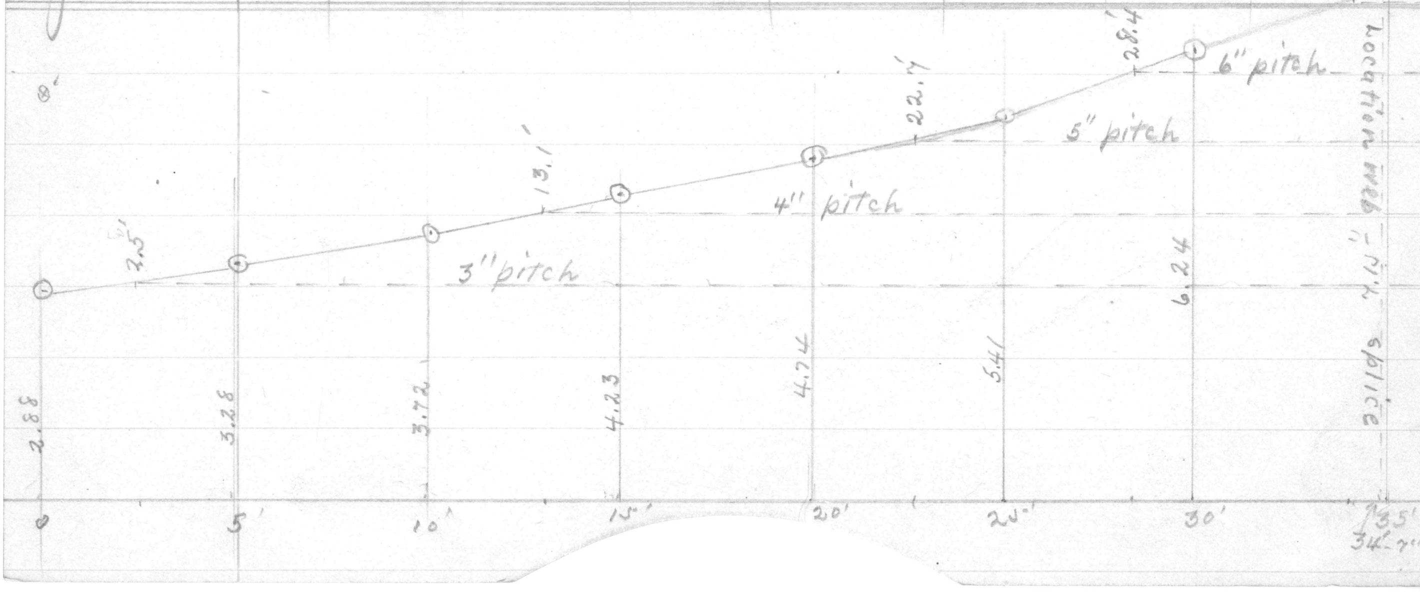
$\therefore S' = \frac{265000}{74.00} \times \frac{22.88}{27.11} = 3020$ at support (see design cover plates)

Resultant of $(3020^2 + 1035^2)^{\frac{1}{2}} = 3192$

$\therefore p = \frac{9190}{3192} = 2.88''$ at support

Determined by bearing value of rivets @ 9190

Section	0'	5'	10'	15'	20'	25'	30'	35'
pitch	2.88"	3.28"	3.72"	4.23"	4.74"	5.41"	6.24"	7.15"



Pitch adopted for angles to web:

section 0' to 13' – pitch = 3"

" 13' " 20' – " = 4"

" 20' " center – " = 5"

- Rivet pitch in cover plates: - Theoretic.

(1) To find percentage of stress in flange due to maximum shear taken by cover plates:

Equivalent flange area of web = 4.04 □"

Area 1 cover plate, net = 6 □"

" 2 " " " = 12 □"

" 3 " " " = 18 □"

Total flange areas:

1 plate + 2 angles + web = 6 □" + 16.88 □" + 4.04 □" = 26.92 □"

2 " " " = 12 + " + " = 32.92 □"

3 " " " = 18 + " + " = 36.92 □"

percentage of stress:

1 plate: 6/26.92 = 22.3 %

2 " , 12/32.92 = 36.4 %

3 " , 18/36.92 = 46.5 %

Increment flange stress at sections, 0', 15.4', 21.3':

Use formula $\frac{V}{h} \times$ percentage of stress

$h_0 = 74''$, $h_{15.4} = 74.68''$, $h_{21.3} = 75.4''$

$$\frac{265000}{74} \times .223 = 798$$

$$\frac{167300}{74.68} \times .364 = 815$$

$$\frac{113600}{75.4} \times .463 = 820$$

Resisting value in single shear for a $\frac{7}{8}''$ rivet = 7220 lbs.

Therefore to get rivet pitch for cover plates we divide 7220 by increments of flange stress just found.

At section 0', $p = \frac{7220}{798} = 9.05''$

" " 15.4', $p = \frac{7220}{815} = 8.86''$

" " 21.3', $p = \frac{7220}{820} = 8.8''$.

The pitch actually employed will be twice the pitch in vertical leg of angle at end of girder, and the same as for vertical leg in remainder of girder. - this in order that adjacent rivet lines shall stagger.

Moment for 2 L^s + 1 cover plate

Equivalent flange area = 12.73% gross web area.

2 L^s, $8'' \times 6'' \times \frac{3}{4}''$, net area	=	16.88 □"
1 plate, $18'' \times \frac{3}{8}''$, " "	=	6.00
12.73% gross web area	=	4.23
		27.11 total flange area.

Effective depth, above flange = 74".

Moment resisted $= \frac{17000 \times 27.11 \times 74}{12} = 2,840,000$ lbs-feet.

= moment at 15.4' from left support.

(40'-8" determined on drawing)

$2(35' - 15.4') + 2 \times 9'' = 40' - 8.4''$ Length 2nd cover plate

2 L^s + 2 cover plates

2 L^s, area (above)	=	16.88 □"
2 plates	=	12.00
12.73% web	=	4.23
		33.11 □" total flange area.

Effective depth = 74.68"

Moment $= \frac{17000 \times 33.11 \times 74.68}{12} = 3,500,000$ lbs-feet.

= moment at 21.3' from left support.

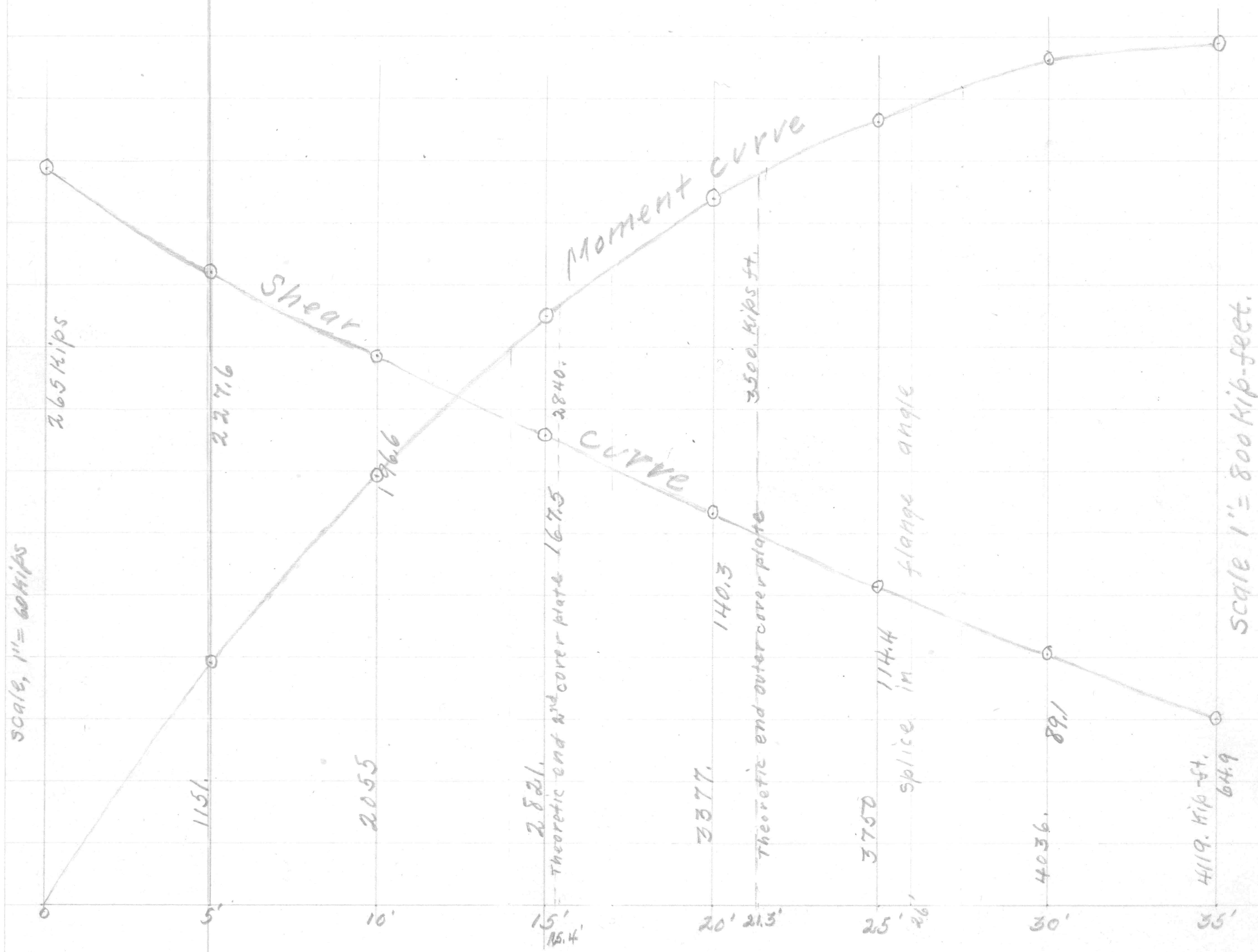
(28'-11" on drawing)

$2(35' - 21.3') + 2 \times 9'' = 28' - 10.8''$ Least length outer cover plate.

A lap of 9" over necessary length was allowed.

17

Lengths of cover plates:



— Flange Splice —

Let splice be at section 26' from end.

Moment at splice = 3925 kip ft. = 47100 kip. inches

Effective depth = 75.4"

Flange stress = $\frac{47100}{75.4}$ = 625. kips.

Area 1 L = 8.44 □" net, ~~&~~ total flange section = 34.88 □".

∴ $\frac{8.44}{34.88}$ x 625000 lbs = 151,200 lbs, stress for spliced angle.

Shearing stress in $\frac{7}{8}$" rivets, (single) = 7220 $\frac{lbs}{in^2}$

∴ $\frac{151200}{7220}$ = 21 rivets for angle.

This would make splice L too long to go between web stiffeners.

∴ use one angle 8" x 6" x $\frac{9}{16}$" and one plate 5" x $\frac{9}{16}$", with legs of angle cut down to $7\frac{1}{8}$" x $5\frac{1}{8}$" x $\frac{9}{16}$".

Net area angle = 7.56 □" − 0.985 □" − 1.13 □" = 5.45 □".

" " plate = 2.81 □" − 0.562 □" = 2.25 □"

7.70

$\frac{5.45}{7.70}$ x 151200 = 107200 lbs, proportion stress for angle.

$\frac{2.25}{7.70}$ x 151200 = 44,200 " " " " plate.

The bearing value in $\frac{3}{4}$" L is greater than stress of rivets in double shear, which

$S_s = 14430 \frac{lbs}{in^2}$

With $S_s = 14430 \frac{lbs}{in^2}$:

No. rivets upper leg of angle = 5 rivets + 3 rivets to transmit local increment flange stress. = 8

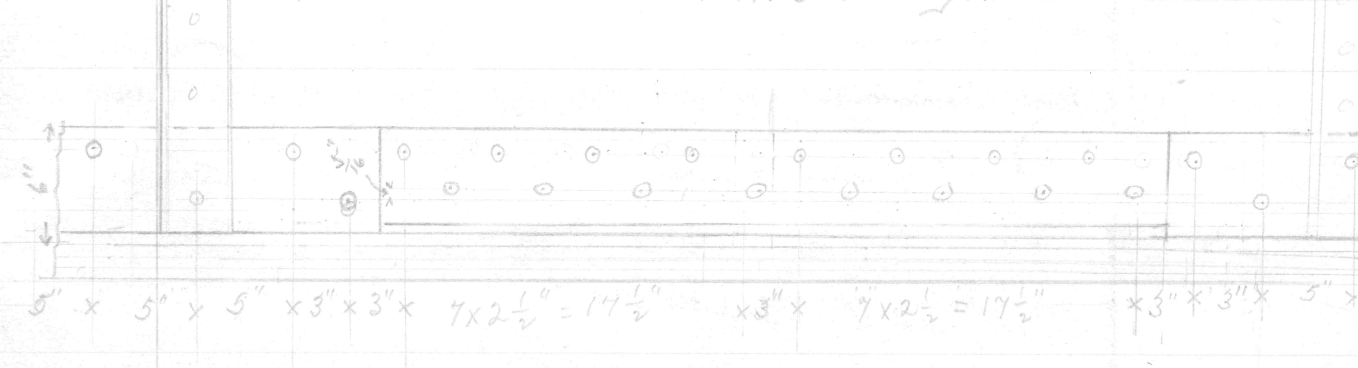
Lower leg = 4 + 4 = 8 rivets, each side of splice.

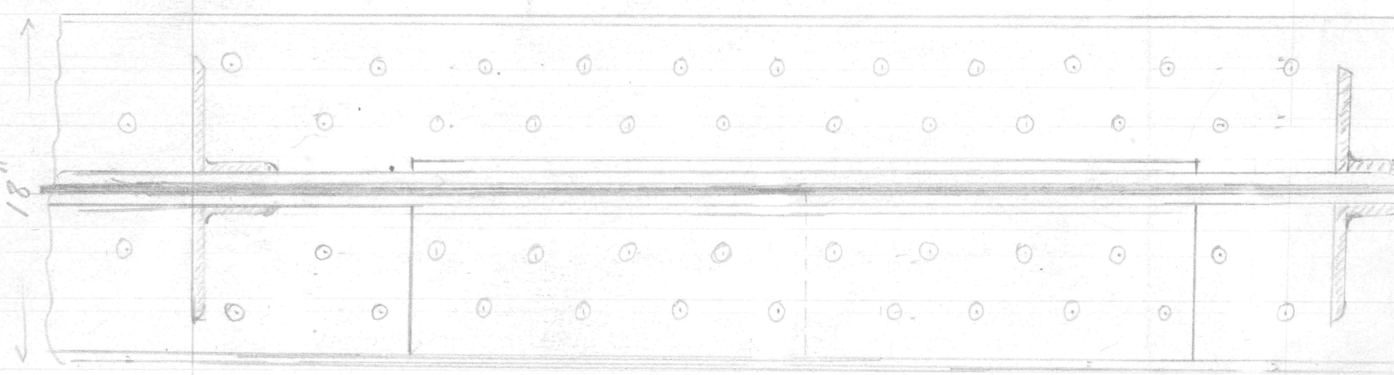
plate = 4 + 4 = 8 " " " "

∴ length splice ∠ and plate = 3' 5"

Since $\frac{3}{8}$" clearance is required, offset rivet lines $\frac{5}{16}$", vertical leg.

No allowance needed on other leg.

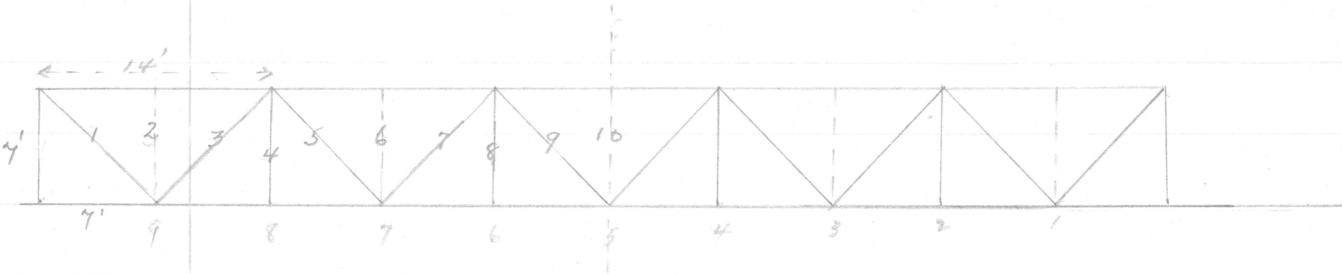




1 splice angle $7\frac{1}{8}$" × $5\frac{1}{8}$" × $\frac{9}{16}$" – 3'5"

1 " plate 5" × $\frac{9}{16}$" — 3'5"

Lateral Bracing.



Static wind load per system 150 lbs per ft.

Moving wind load upper system = 300 lbs per ft.

Static panel load = 150 x 7' = 1050 lbs

Moving " " = 300 x 7 = 2100 lbs

Stresses in diagonals:

	1	3	5	7	9	any strut
I, Load	6680	5200	3716.	2230	743.	-1050
L "	13380	10700.	8320.	6250	4460.	-2100
Max. stress	±20060.	±15900.	±12036.	±8480.	±5203.	-3150

Diagonals are designed to take either tension or compression.

Lower lateral bracing same as above, but without sub-struts.

Tensile and compressive stress not over 17000 $\frac{lbs}{in^2}$ and compressive members to be designed by column formula; $p = 17000/\left(1+\frac{1}{11000}\frac{l^2}{r^2}\right)$.
l/r shall not exceed 120.

As per diagram next page, length of diagonal = 89"

$\therefore \frac{89"}{120} = r = 0.74$ = radius of gyration

void

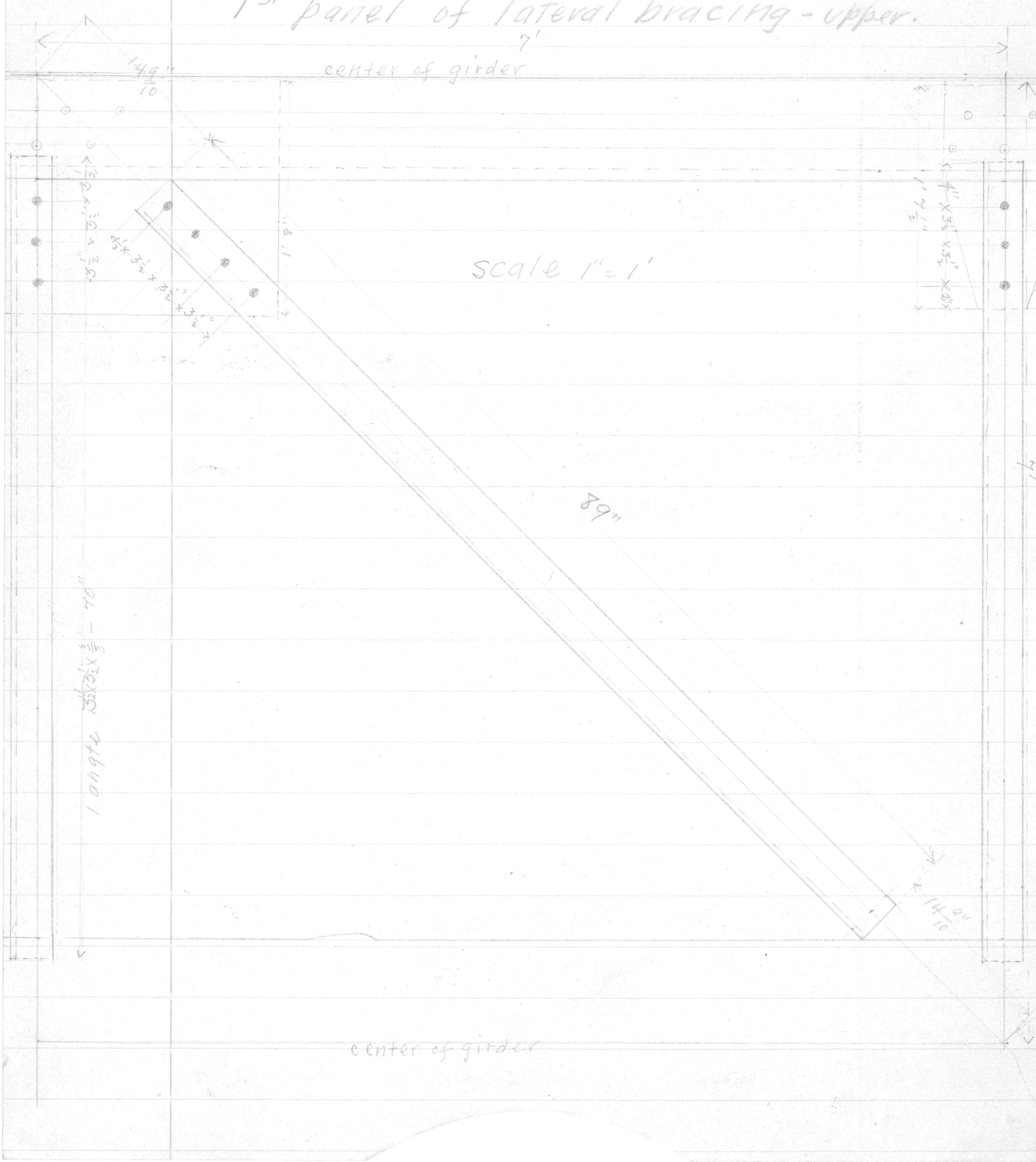
An angle 5" x 4" x $\frac{3}{8}$" has r = 0.85, area = 3.24□".
$l/r = \frac{89"}{0.85} = 102"$, and substituting in formula:

$p = 17000/\left(1+\frac{1}{11000}\cdot(102)^2\right) = 8650$ lbs compressive stress.

$\therefore$ for end diagonal:
$\frac{20060}{8650} = 2.32$□" area required.
Above allows excessive sectional area.

Use angle 4" x 4" x $\frac{3}{8}$"
$r = 0.79$; area = 2.86□"; $\frac{l}{r} = \frac{89}{.79} = 113$.
$p = 17000/\left(1+\frac{1}{11000}\cdot 113^2\right) = 7550$ lbs
$\therefore$ area required = $\frac{20060}{7550} = 2.66$□"

1st panel of lateral bracing - upper.



To investigate $4'' \times 4'' \times \frac{3}{8}''$ angle for bending due to column action and eccentric connection:

Distance back of ∠ to c.g = 1.14", area = 2.86□"

$I = 4.36\,(in^4)$, radius gyration = 1.23"

$l/r = 89/1.23 = 72.4$

Maximum compressive stress on concave side =

$$S' = \frac{20060}{2.86}\left(1 + \frac{72.4^2}{11000}\right) = 10360 \text{ lbs-in}$$

Moment due to eccentric connection

$$= \frac{20060}{1.14} = 17600 \text{ lbs-in.}$$

and compressive stress outer fiber of angle

$$S'' = \frac{17600 \times 1.14}{4.36 - \frac{20060 \times 89^2}{9.6 \times 29000000}} = 5300 \text{ lbs-in}$$

∴ Total stress = 10360 + 5300 = 15660. lbs-in

∴ Use ∠s $4'' \times 4'' \times \frac{3}{8}''$ for diagonals.

Lateral struts:

Length struts = 70".

∴ r must be not less than $\frac{70''}{120} = .584''$

$l/r = 70/.68 = 103$, for a $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}''$ angle.

$p = 17000\left(1 + \frac{103^2}{11000}\right) = 8650$

Compressive stress in struts, not over 2100 lbs.

$2100/8650 = 0.243\square''$ area required for angle.

We cannot use angles less than $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{3}{8}$ under specifications, and area this angle $= 2.49\square''$.

Same size angles will be used for lower laterals.

Rivets for connecting lateral angles to plates:—
Since rivets are field driven, number must be increased 25%.

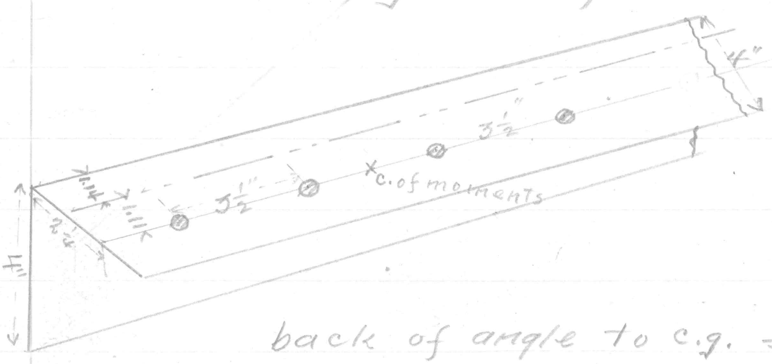
For end diagonal, $P = 20060$ lbs.

Value of $\frac{7}{8}$" rivet in single shear = 7220 lbs.

20060/7220 = 3 rivets required.

Add one rivet for eccentricity = 4 rivets.

Investigation for eccentricity:



back of angle to c.g. = ~~1.14~~ 2.25"

rivet line to c.g. of ∠ = 1.11"

Moment of rotation in plane of shearing surfaces for pressure in diagonal = 20060 × 1.11 = 22,260. lbs.

Longitudinal shear each rivet = $\frac{20060}{4}$ = 5020 lbs ∥ to axis.

Distances rivets to c. of moments = $\begin{cases} 2 \text{ rivets @ } 5.25" \\ 2 \quad " \quad @ \ 1.75" \end{cases}$

Let shear in most distant rivet = P

$$\left[(5.25)^2 + (5.25)^2 + (1.75)^2 + (1.75)^2\right] \frac{P}{5.25} = 22260 \text{ (from above)}$$

∴ Shear ⊥ to rivet line, $P = 1910.$ lbs

$\sqrt{(5020)^2+(1910)^2} = 5375$ lbs, resultant shear.

5375 × 4 rivets = 21500 lbs to be resisted.

20% of 7220 lbs = 5770 lbs, allowable shear on field rivets.

5770 × 4 = 23080 lbs, rivets can resist.

∴ 4 rivets will be needed

For 2nd diagonal:

Resultant shear for 3 rivets was found = 5850 lbs

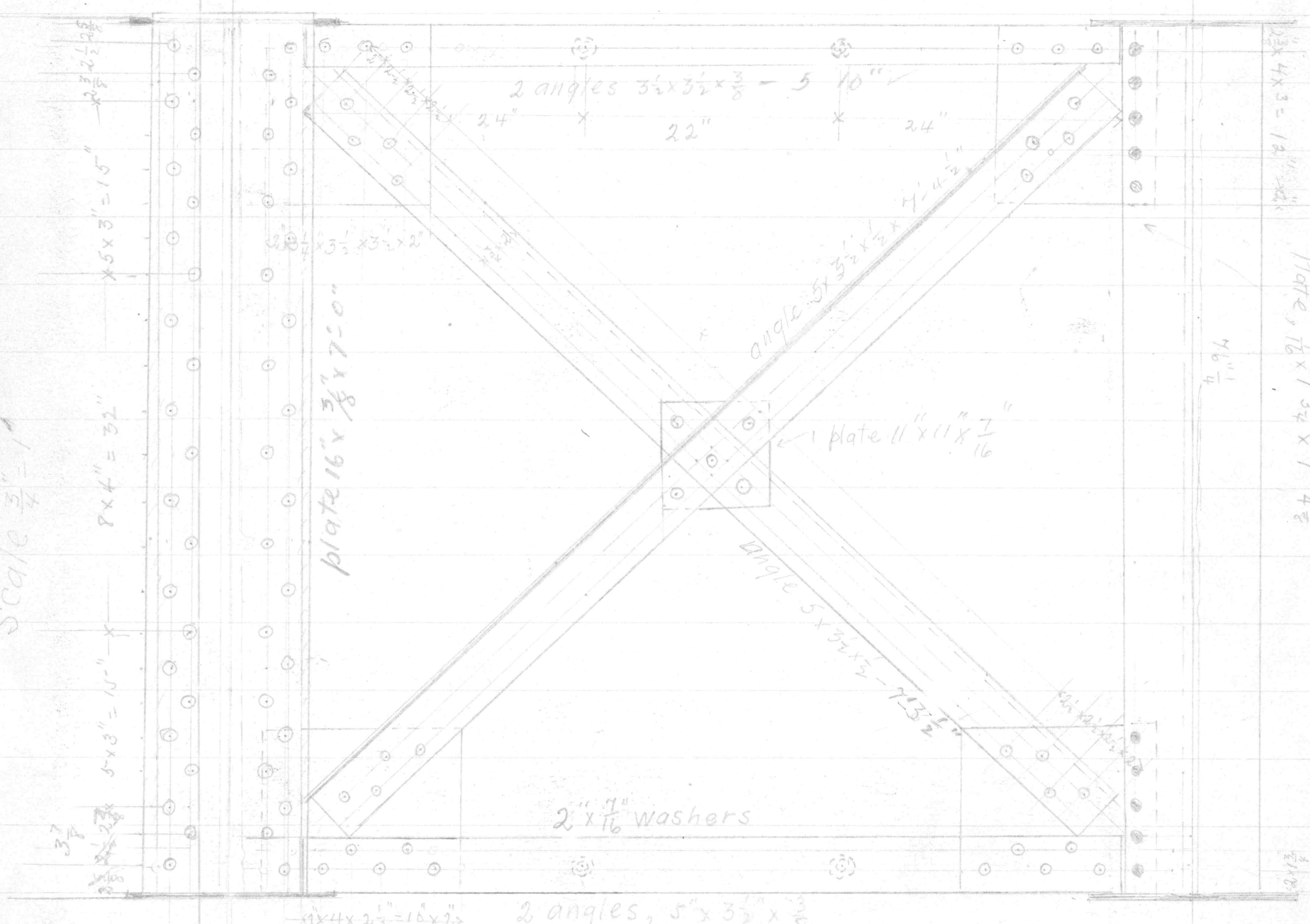
3 × 5850 = 17550 lbs, to be resisted

3 × 5770 = 17310 " " , allowable

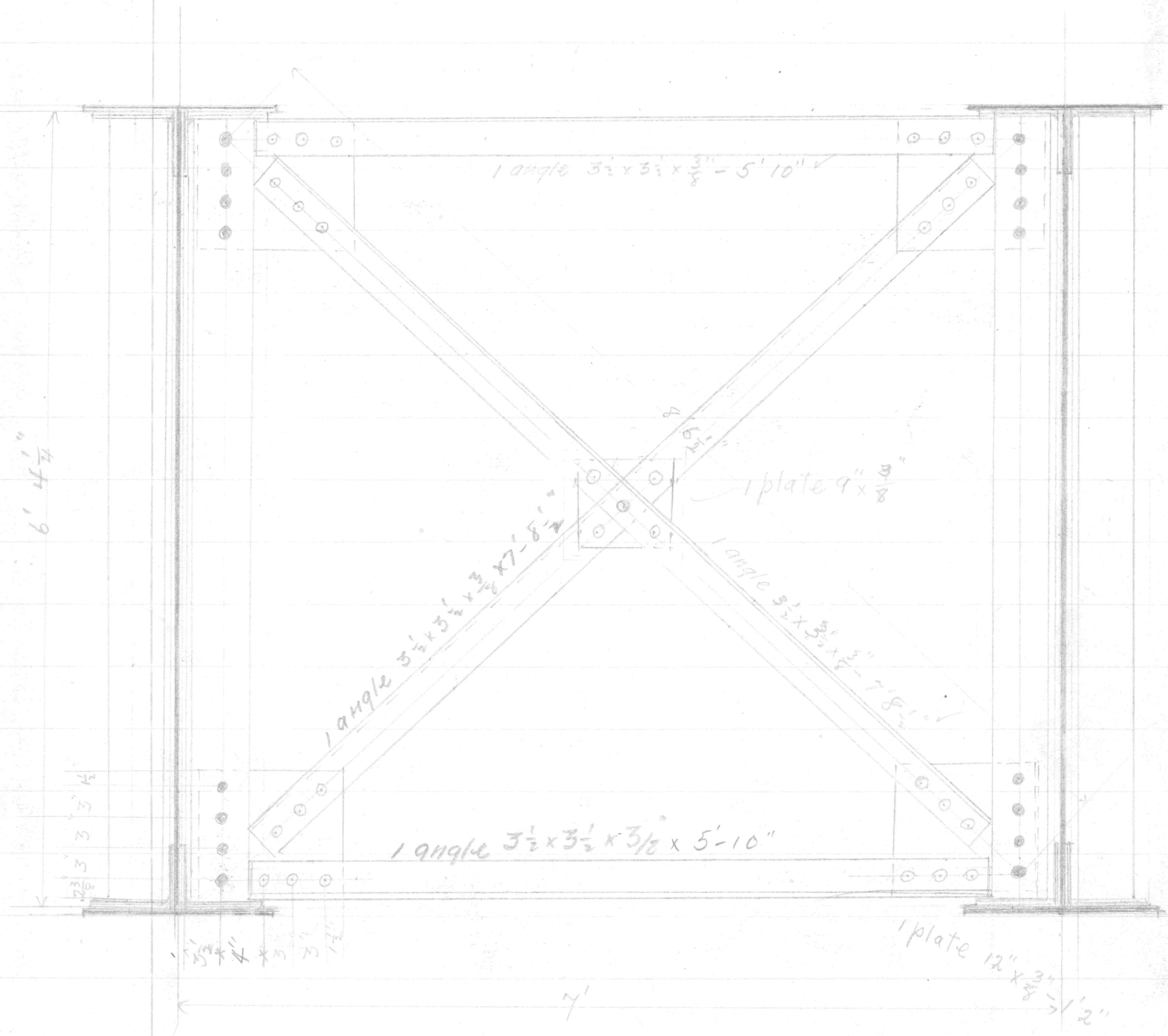
∴ 4 rivets also needed here.

For other members we will use 3 rivets, since specifications do not allow less.

End Cross Frame.



Intermediate cross frame.



– Estimate of weight. –

Material for $\frac{1}{2}$ of the girder:

Flanges:

4 angles, $8'' \times 6'' \times \frac{3}{4}'' \times 35'\text{-}10''$ @ 33.9 $\frac{\text{lbs}}{\text{ft}}$ = 4850 lbs.

2 cover plates, $18 \times \frac{3}{8} \times 35'\text{-}10''$ @ 22.95 $\frac{\text{lbs}}{\text{ft}}$ = 1645.

" " " $18 \times \frac{3}{8} \times 20'\text{-}4''$ = 932

" " " $18 \times \frac{3}{8} \times 14'\text{-}6''$ = 665

8092

Flange Splices:

2 cover angles, $7\frac{1}{8} \times 5\frac{1}{8} \times \frac{9}{16} \times 3'5''$, @ 25.8 $\frac{\text{lbs}}{\text{ft}}$ = 176.

2 splice plates, $5 \times \frac{9}{16} \times 3'5''$, @ 9.56 " = 65.

241

Web:

1 plate, $76'' \times \frac{7}{16}'' \times 35'10''$, @ 113.1 $\frac{\text{lbs}}{\text{ft}}$ = 4055.

Web splice:

2 plates, $13'' \times \frac{3}{8} \times 5'4''$ @ 16.58 $\frac{\text{lbs}}{\text{ft}}$ = 176.

4 " , $5'' \times \frac{3}{8}'' \times 3'$ @ 6.38 " = 77.

253

Stiffeners:

21 angles, $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}'' \times 6'\text{-}4\frac{1}{4}''$ @ 11.7 $\frac{\text{lbs}}{\text{ft}}$ = 1560

5 fillers, $3\frac{1}{2} \times \frac{3}{4} \times 5'4\frac{1}{4}''$ @ 8.93 " = 239

2 plates, $20'' \times \frac{3}{4} \times 5'4\frac{1}{4}''$ @ 51.0 " = 545.

1 end cover plate, $16'' \times \frac{3}{8}'' \times 7'\text{-}0''$ @ 22.95 " = 161

2505

Total 15146 lbs

One-half of upper Lateral System.

Braces: 5 angles, 4"x4"x 3/8 x 7'-5" @ 9.8 lbs =	364
2½ angles, 3½"x 3½"x 3/8"x 5'-10" @ 8.5 lbs	123
11 connecting 7/16" plates, aggregating 33.4 sq. ft., @ 17.85 lbs =	596
	1083

One-half of lower lateral system.

Braces: 5 angles, 4"x 4"x 3/8"x 7'-5" @ 9.8 lbs =	364
8½ connecting 7/16" plates, aggregating 29. sq. ft, @ 17.85 lbs =	517
	881

End cross frame:

2 angles, 3½"x 3½"x 3/8"x 5'-10" @ 8.5 lbs =	99
2 angles, 5"x 3½"x ½"x 7'-4" @ 13.6 lbs =	199
2 angles, 5"x 3½"x 3/8"x 5'-10" @ 10.4 lbs =	122
5 connecting 7/16" plates, aggregating 6.7 sq. ft, @ 17.85 lbs =	120
4 washers, 7/16" =	4
	544

Intermediate cross frame:

2 angles, 3½ x 3½ x 3/8 x 5'-10" @ 8.5 lbs =	99
2 angles, 3½ x 3½ x 3/8 x 7'-7" @ 8.5 lbs =	128
5 connecting 7/16" plates, aggregating 4.5 sq. ft, @ 17.85 lbs =	80
	307

Dead Load For One Girder, Excluding Track.

1 girder, 2 x 15146 =	
½ upper lateral system =	1083
½ lower lateral system =	881
1 end cross frame =	544
2 intermediate cross frames =	614
2463 pairs of rivet heads @ 0.452 =	1115

Gross weight for length of 71'-8" = 4237 lbs

Net weight for length of 70' (the span) = 4140 lbs.