Thesis 1912

B.S. in C.E.

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THESIS

FOR

B. S. C. E. DEGREE

RESPECTFULLY SUBMITTED
TO THE FACULTY OF THE
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WASHINGTON AND LEE UNIVERSITY

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DESIGN

50 FT. DECK PLATE GIRDER

SINGLE TRACK

8 FT. WIDE

5 FT. DEEP

COOPERS CLASS "E" 50

DEAD LOAD

Net weight of one girder and ½ of lateral bracing and cross frames — 35000 lbs = 35 Kips.

Cross ties per linear ft —— 310 lbs.

Rails, spikes, bolts ect. <u>160</u> lbs.

470 lbs ÷ 2 x 50 = 11750 lbs = 11.75 Kips.

35 Kips + 11.75 Kips = 46.75 Kips = dead load of one girder for span.

Net Dead Load = 935 lbs per linear foot.

Maximum Dead Load Bending Moment is at section 25 = $\frac{1}{8} Wl^2$

$\frac{1}{8} Wl^2 = \frac{1}{8} \cdot 935.0 \times \overline{50}^2 = 292.19$

Bending Moment at section 25 = 292.19 K.

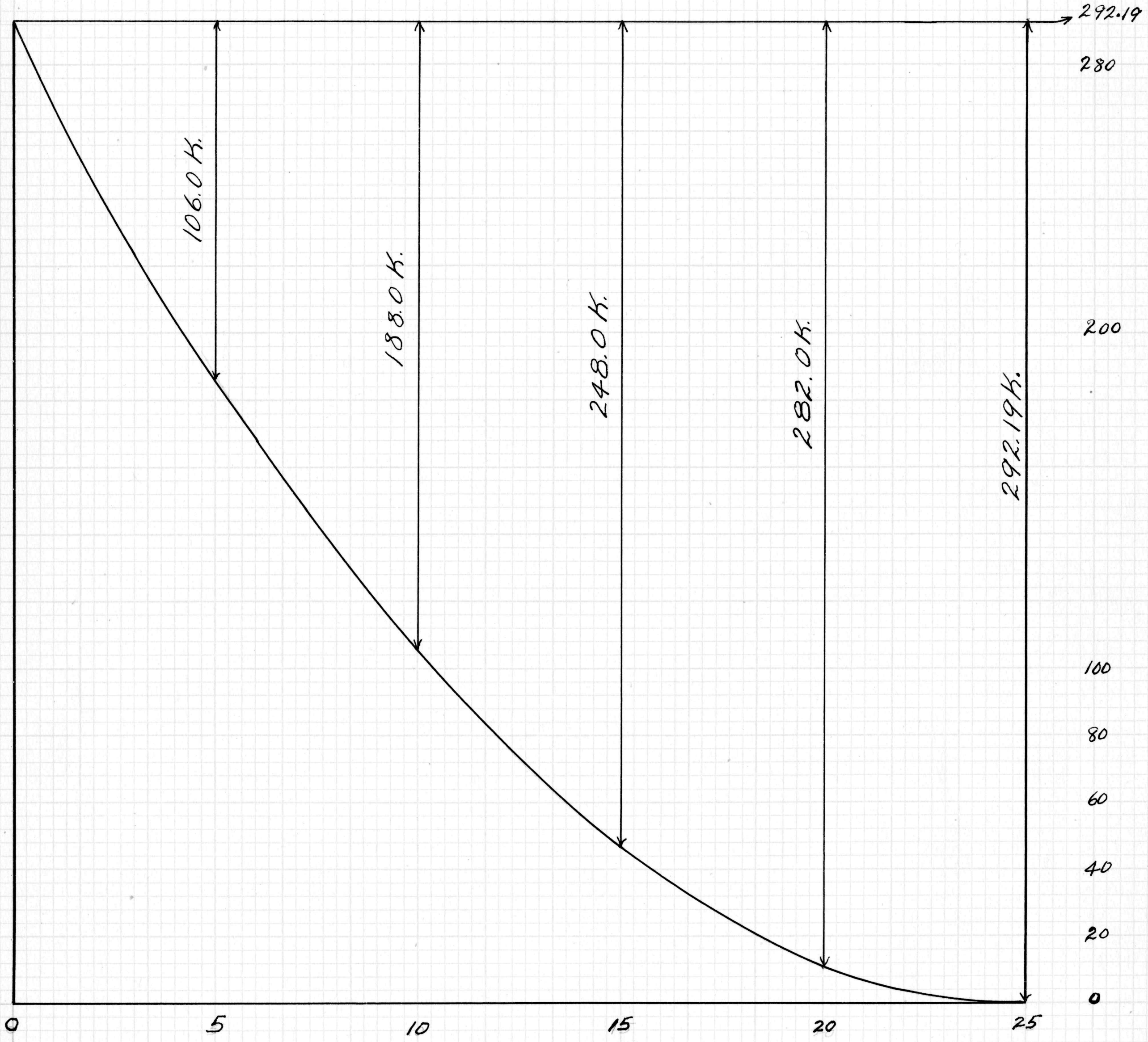
Maximum Dead Load Shear is at section 0 = $\frac{1}{2} Wl$

$\frac{1}{2} Wl = \frac{1}{2} \cdot 46.75 = 23.375$ Kips.

Maximum Dead Load Shear = 23.375 K.

Dead Load Bending Moments.

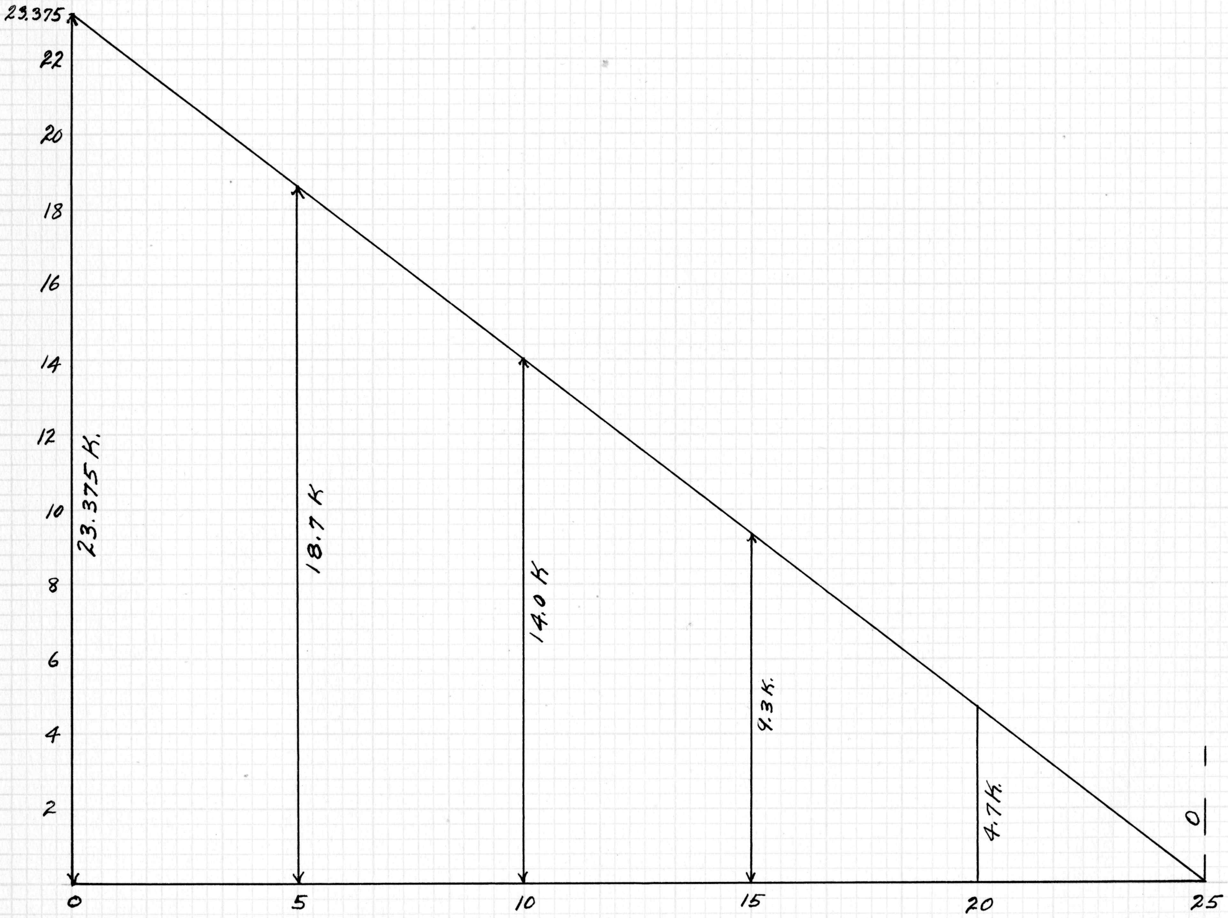
Scale
3 cm = 5 ft.
1 cm = 20 Kips.



Dead Load Shear.

Scale.

1cm = 2 Kips.
3cm = 5 ft.



Impact.

$$I = \frac{400}{L+500}$$

L = length of span.

$$I = \frac{400}{50+500} = 73\%$$

73 per cent of the live load will be added for impact.

Absolute Maximum Bending Moment.

Wheel 12 at section 24.5

Live Load	=	1160.0
Dead Load	=	846.8
Impact	=	291.0
Maximum		2297.8

Shear and Moments.

From diagram for Coopers Class E 50 similar to Part I page 122.

Shear.						
Section	0	5	10	15	20	25
Wheel at sec.	2	2	2	2	2	2
Live Load	122.0	105.0	87.0	70.0	55.0	43.0
Dead Load	23.75	18.7	14.0	9.3	4.7	0
Impact	89.1	76.7	63.5	51.1	40.2	31.4
Maximum.	234.85	200.4	164.5	130.4	99.9	74.4

Live Load Flange Stresses.						
Section	0	5	10	15	20	25
Wheel at sec.	0	2	2	3	3 or 4	4 or 5
Live Load	0	85.0	155.0	200.0	220.0	230.0

Bending Moments.

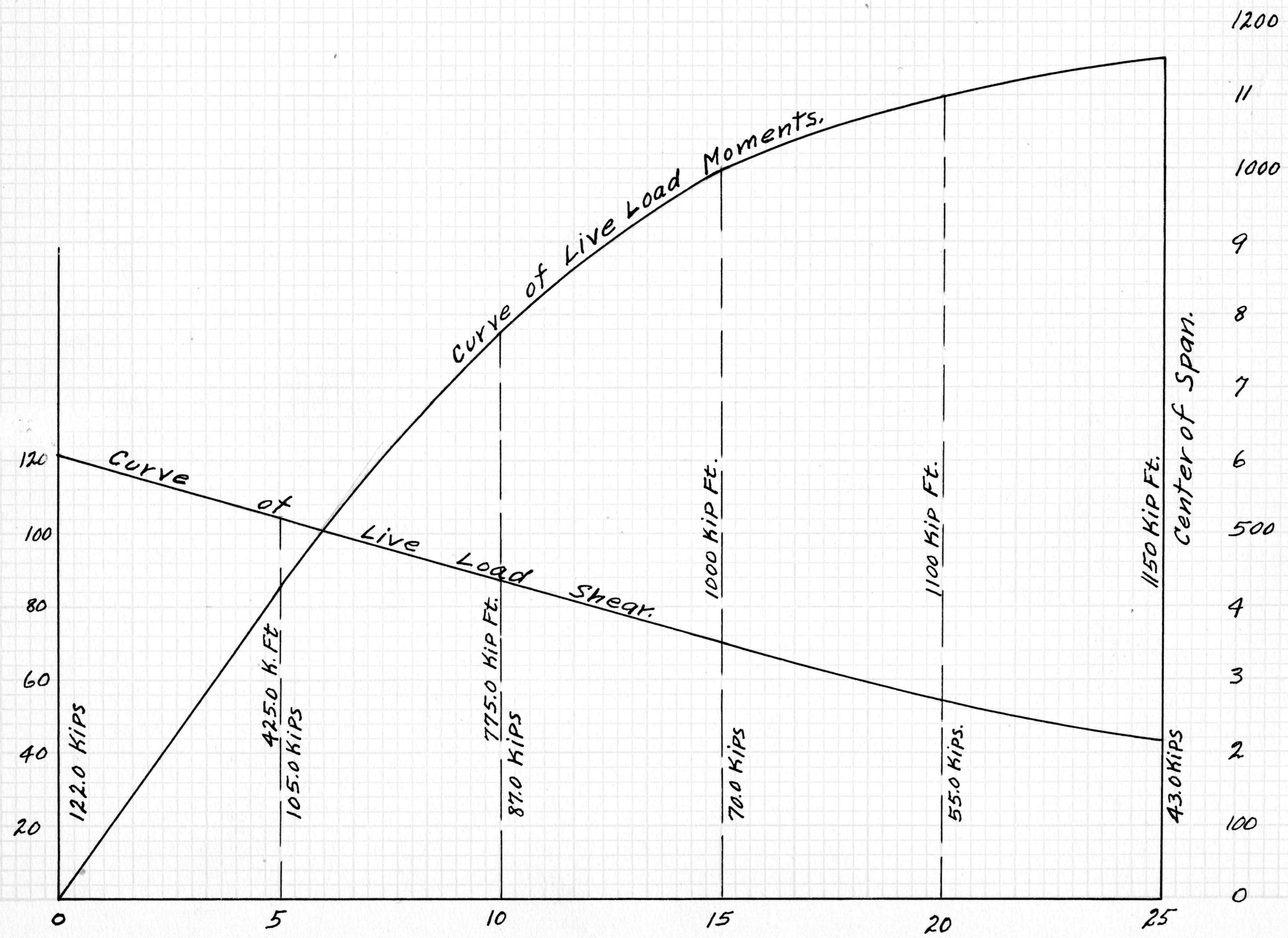
Bending Moment = Flange Stress X depth.

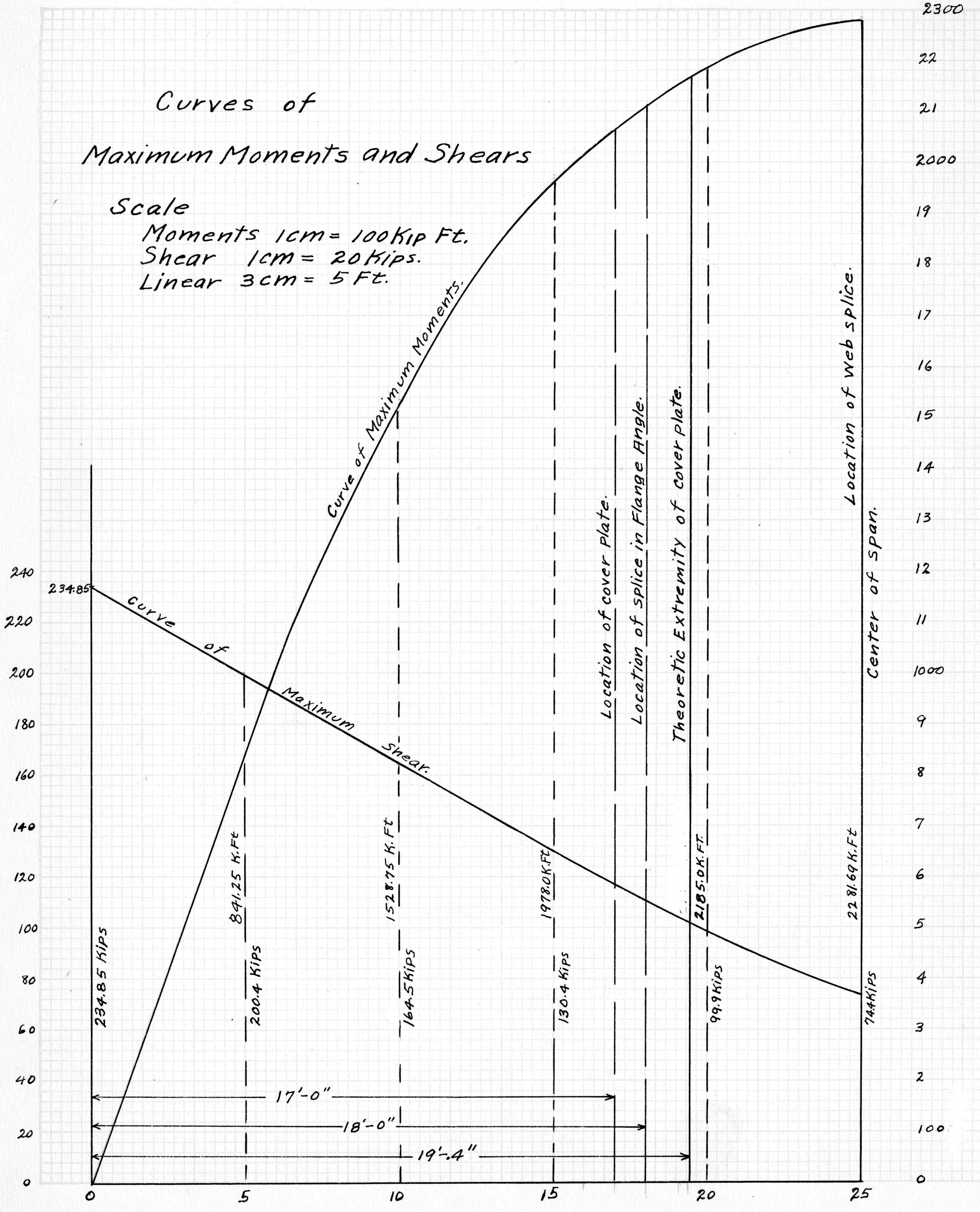
Section	0	5	10	15	20	25
Wheel at sec.	0	2	2	3	3 or 4	4 or 5
Live Load	0	425.0	775.0	1000.0	1100.0	1150.0
Dead Load	0	106.0	188.0	248.0	282.0	292.19
Impact	0	310.25	565.75	730.0	803.0	839.5
Maximum	0	841.25	1528.75	1978.0	2125.0	2281.69

Curves of

Live Load Moments and Shears.

Scale
Moments 1cm = 100 Kip Ft.
Shear 1cm = 20 Kips
Linear 3cm = 5 Ft.




Curves of
Maximum Moments and Shears
Scale
Moments 1cm = 100 Kip Ft.
Shear 1cm = 20 Kips.
Linear 3cm = 5 Ft.
Curve of Maximum Moments.
Curve of Maximum Shear.
234.85
234.85 Kips
841.25 K.Ft
200.4 Kips
1528.75 K.Ft
164.5 Kips
1978.0 K.Ft
130.4 Kips
2185.0 K.FT.
99.9 Kips
2281.69 K.Ft
74.4 Kips
Location of cover plate.
Location of splice in Flange Angle.
Theoretic Extremity of cover plate.
Location of Web splice.
Center of span.
17'-0"
18'-0"
19'-4"
0
5
10
15
20
25
0
20
40
60
80
100
120
140
160
180
200
220
240
2300
22
21
2000
19
18
17
16
15
14
13
12
11
1000
9
8
7
6
5
4
3
2
100
0

Wooden Floor.

The floor consists of cross ties of rectangular section, notched one half inch over flange, and allowance made for rivet rivet heads. The space between ties will be five inches. Every third tie will be secured to the flange by a $\frac{3}{4}''$ hook bolt. Guard timbers $6'' \times 8''$ will be laid parallel to track and notched one inch over every tie. Every third tie is bolted to the guard rail by a $\frac{3}{4}''$ bolt. The guard timbers are spliced over a tie, by a half lap joint 6" long, and a bolt passed thru the splice and tie. The guard rail will be placed about 11" from the gauge side of rail.

Old track rails will be placed inside the gauge with a clearence of 6" to 10" from each rail.

The ties will be 13" deep, 8" in breadth and 12 feet long.

Web Section.

Specifications:

The rivets used shall be seven eights of an inch indiameter. The shearing stress in web plates shall not exceed 12000 lbs per sq. in.. No web plate shall be less than $\frac{3}{8}$ inch in thickness.

The total shear at support is 234850 lbs.

Unit shearing stress = 12000 per sq. in.

Sectional area of web = $\frac{234850}{12000}$ = 19.57 sq. in.

(Assume shear uniformily distributed.)

For 60" depth, thickness = 0.326" = $\frac{3}{8}$" nearly.

$\frac{3}{8}$" thickness allows only 7 rivets in 60" depth.

A thickness of ½ inch will allow 21 rivets with an average pitch of 3 inches.

A thickness of ½ inch in web will be used.

Sectional Area of Flanges.

A Plate Girder is under the action of vertical loads as in a beam, therefore the flexure formulae applies to it.

$$M = \frac{SI}{c}$$

but transforming this equation so as to be more convienient to get the sectional area

$$A = \frac{M}{S h_1} - \frac{th}{6}$$

M = max. moment in lbs = 27576000 lbs.

S = unit stress in fiber = 17000 lbs.

t = thickness = ½ inch.

h = depth = 60 inches

h_1 = effective depth = 58.75 inches.

$$A = \frac{27{,}576{,}000}{17000 \times 58.75} - (0.12 \times \tfrac{1}{2} \times 60)$$

$$= 27.64 - 3.6$$

$$= 24.04 \text{ sq. in.}$$

Theoretic area of Flanges = 24.04 sq. in.

Composition of Flanges.

Specifications:

About one half of the flange section shall consist of angles. The number of cover plates shall be as small as practicable. The cover plates shall be of equal thickness, and shall not exceed more than four inches or eight times the thickness of the plate beyond the outer line of rivets. The net section of the tension flange shall be determined by a plane cutting it square across at any point and the greatest number of rivet holes which can be cut by any such plane or whose centers come nearer to it than 2½ inches, are to be deducted from the gross section in computing the net area. The compression flange shall have the same gross section as the tension flange.

The effective diameter of rivets shall be assumed the same as its diameter before driving; but in making deductions for rivet holes in tension members, the diameter of the holes shall be assumed to be one-eighth of an inch larger than the rivet.

One half of the net flange area as determined before = 12.02 sq. in.

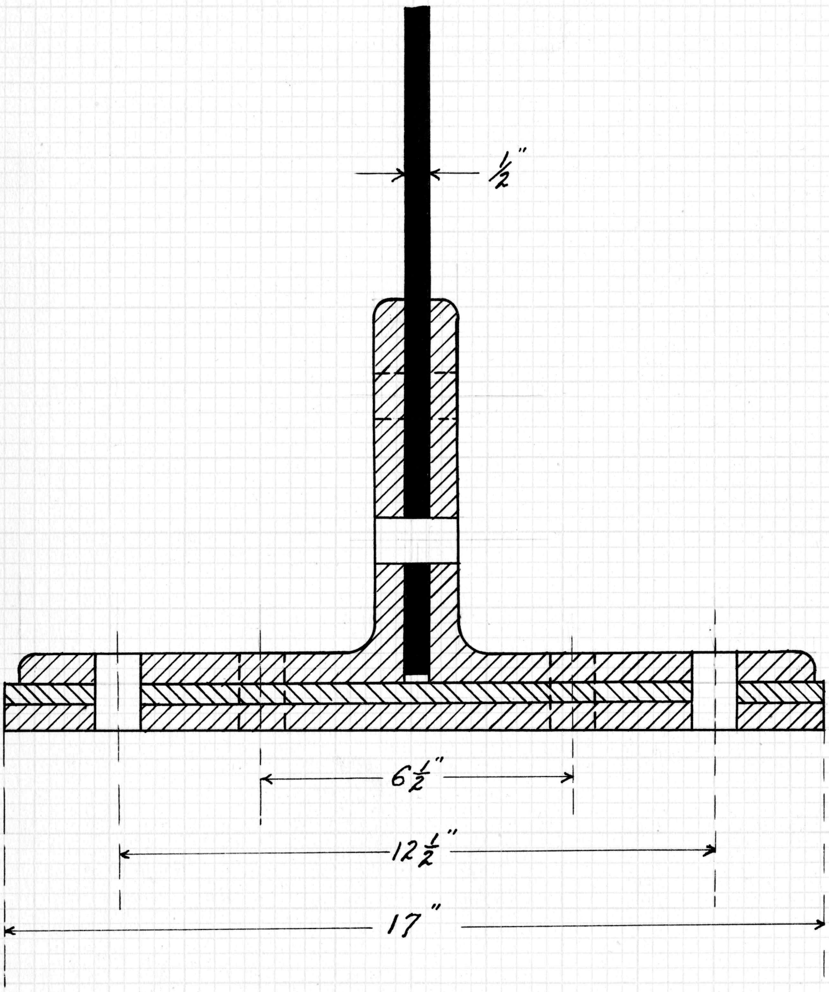
Using 8"x8" angles, the flange will consist of

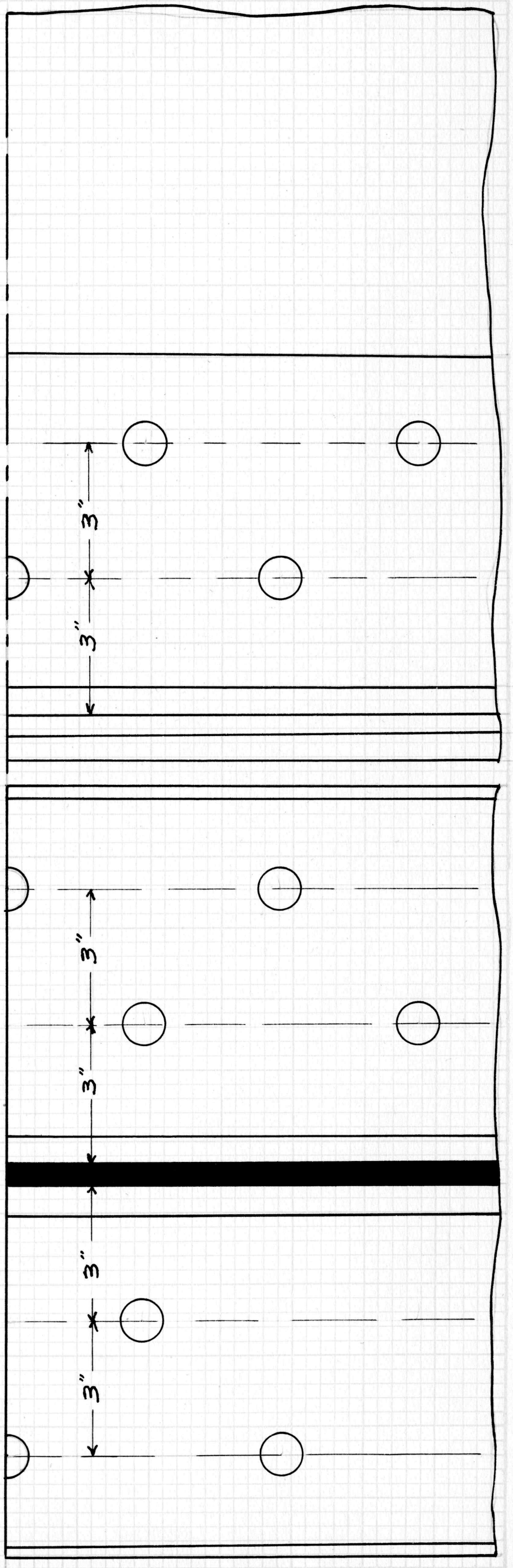
2 angles $8'' \times 8'' \times \frac{5}{8}''$ = 19.22 − 3 = 16.22 sq. in.

2 cover plates $17'' \times \frac{7}{16}$ = 14.87 − 2.34 = 12.53 sq. in.

Total net section = 28.77 sq. in.

The net section of each flange will be 28.77 sq. in. This area exceeds the theoretical area by a safe allowance.





Flange.

Angles $8'' \times 8'' \times \frac{5}{8}''$
Web $\frac{1}{2}''$ thick.
Cover Plates $\frac{7}{16}''$ thick.

Scale
1 in. = 4 inches.

Rivets $\frac{7}{8}$ inches in diameter and
3 inch pitch.

Web Splice.

Specifications:

If splices are necessary, their number shall be made as small as possible. The splice plates and rivets for the splices shall be such as to develop in every respect the full strength of the net section of the web, the main splice plates extending from flange to flange and having at least two rows of rivets on each side of the joints.

The shearing stress on rivets shall not exceed 12000 pounds per square inch of section, and the pressure upon the bearing surface of the projected semiintrados (diameter times thickness) of the rivet hole shall not exceed 24000 pounds per square inch. Plates less than $\frac{3}{8}''$ will not be used.

A web splice will be required as a plate 60 inches by 50 feet (by Cambria) is not rolled a sufficient length. The splice will be put at section twenty five.

The shear at section 25' is 74400 pounds.

The unit stress of a rivet will be 24000 lbs.

$\frac{74400}{24000} = 3.1$ sq. in.

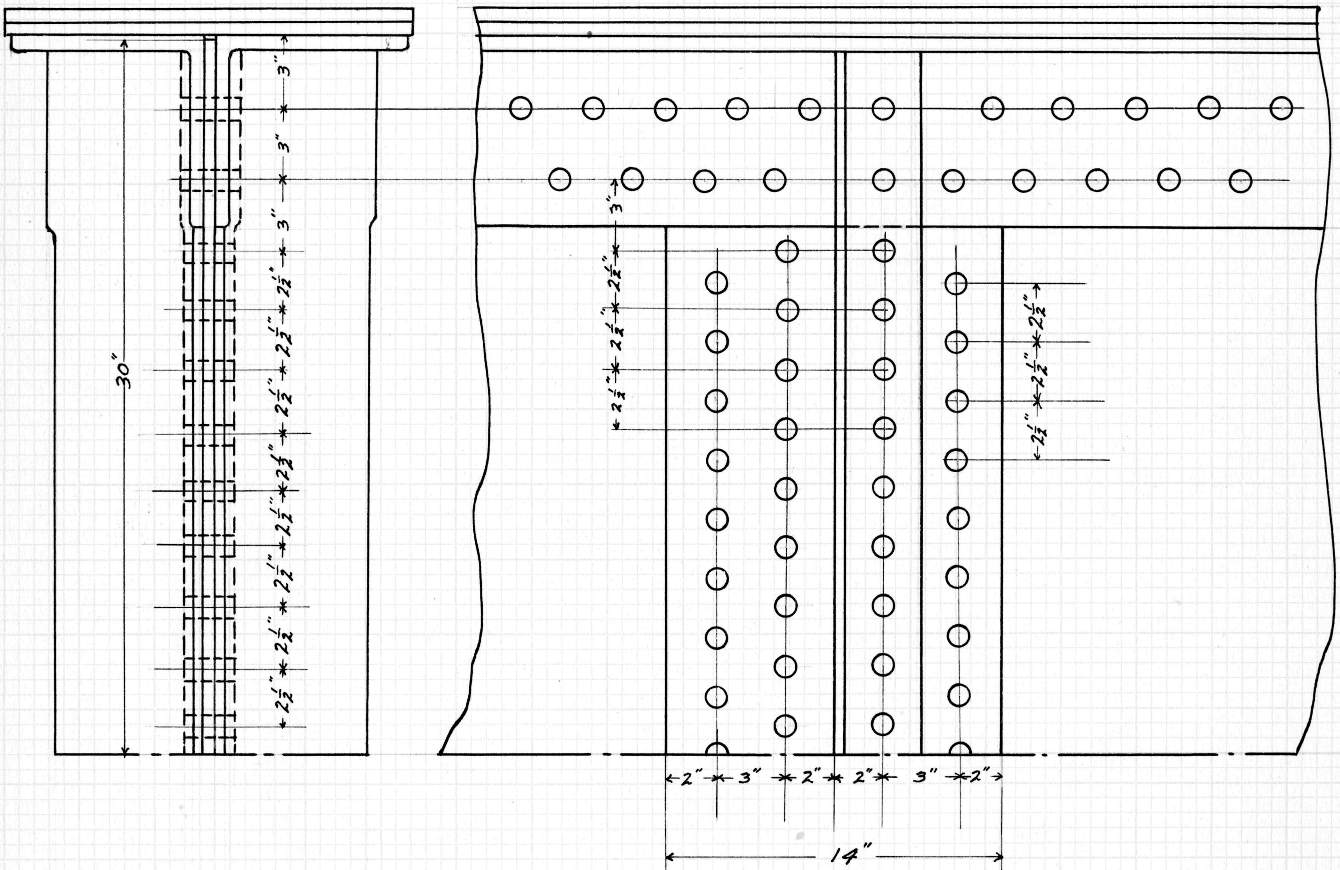
The plate will be 43.75 inches long.

$\frac{3.1}{43.75} = .0707$ but as specifications require that no plate shall be less than $\frac{3}{8}''$ thick, two plates one on each side of splice $\frac{3}{8}$ inches thick will be used. Two rows of rivets are required, the plates will therefore be $43.75'' \times 7'' \times \frac{3}{8}''$

The pitch at section twenty five is six inches, but owing to the splice the pitch will be made two and one half inches, and for uniformity the same number of rivets will be used in the web splice as in web stiffener. The rivets of the web stiffener will go thru both plates and web at this point.

Web Splice.

½ of Girder shown.



Splice Plate 43.75" X 7" X ⅜"

Scale
1½ in = 1 Ft.

Web Stiffeners.

The greatest concentrated load is 30000 lbs. and 24000 lbs in addition for impact.

The specified compressive unit stress is 17000 lbs. per sq.in. A sectional area of 3.18 sq.in. is required. As $\frac{7}{8}$ inch rivets are to be used the leg of the angle cannot be less than $3\frac{1}{2}''$, 2 angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ will furnish this area of section, but the outstanding leg will not give sufficient support to the eight inch flange. The size will therefore be increased to $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, for stiffeners otherwise than at the ends. The stiffeners will be placed at intervals of five feet, throughout the girder, one on each side of web.

The end stiffeners must take the vertical shear from the web and carry it to the bearing plate. A lower working stress is taken than for simple compression, about 15000 lbs per sq.in.

The maximum shear at section 0 is 234850 lbs. $234850 \div 15000 = 15.66$ sq.in. This area can be furnished by four angles

$6'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ but $7'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$ will be used.

The shearing value of a $\frac{7}{8}$ inch rivet in a $\frac{1}{2}$ inch plate is 10940 lbs.

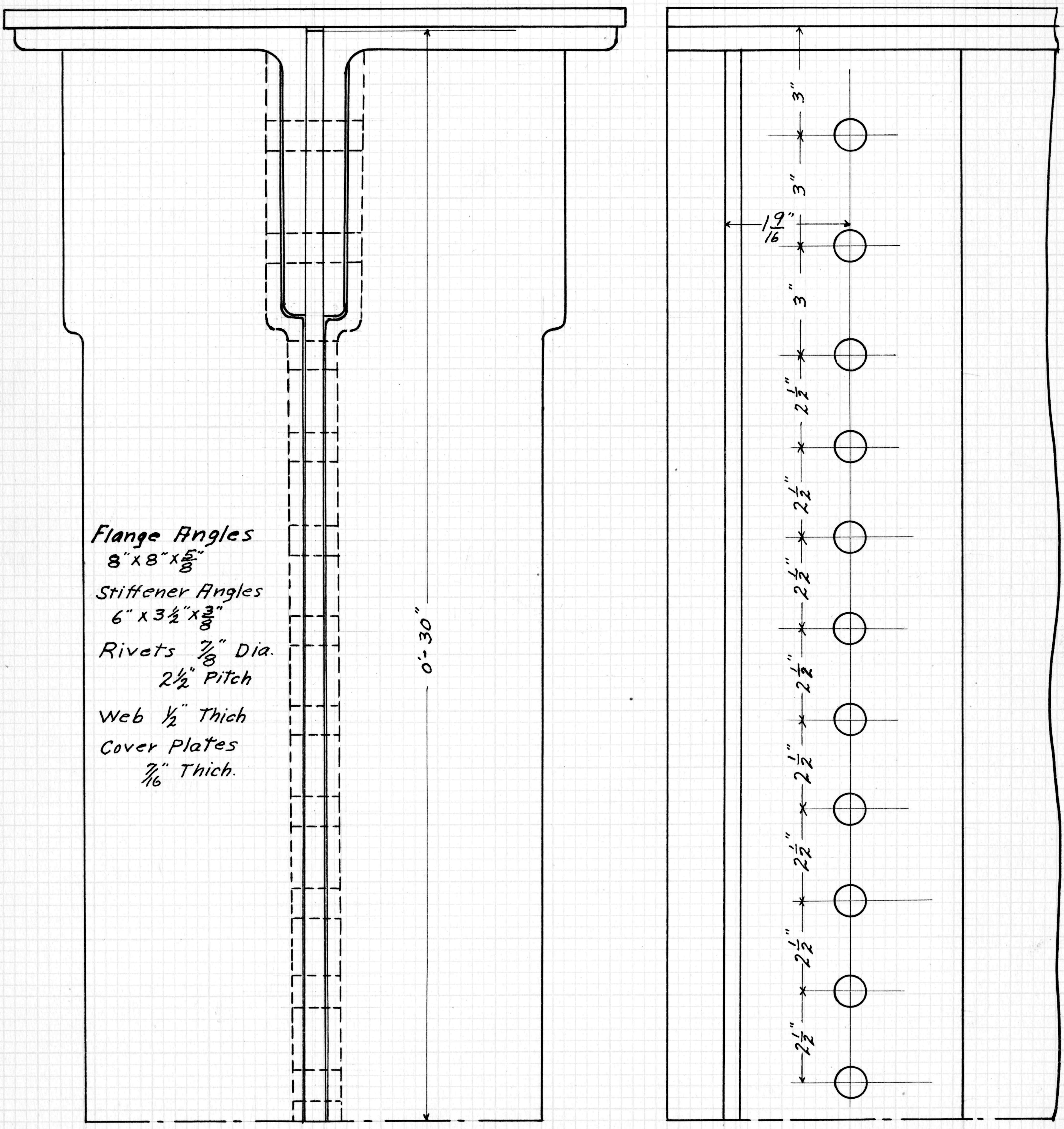
$$\frac{234850}{10940} = 22 \text{ rivets}$$

but in order to simplify construction, the same number of rivets will be used in all stiffeners, twenty four rivets at $2\frac{1}{2}$ inch pitch.

The stiffeners will be bent over flange angles, and make a tight fit under flange angles.

Web Stiffener.

½ of Girder Shown.



Cover Plates.

Specifications:

One cover plate on each flange will be extended to the end of the girder. Other cover plates will be extended at each end at least one foot beyond the point where theory requires them in order to resist the maximum bending moments in the girder.

Metal less than $\frac{3}{8}$" thick will not be used.

The rivets will be $\frac{7}{8}$" in diameter.

The combined area of two flange angles and one cover plate (using $\frac{7}{16}$" thickness) is 22.49 sq.in. The flange area of the web is 13.3% of the gross area of the web = 3.5 sq.in.

The effective depth is found to be 58.75 sq.in.

The bending moment that may be resisted by this section of the girder is

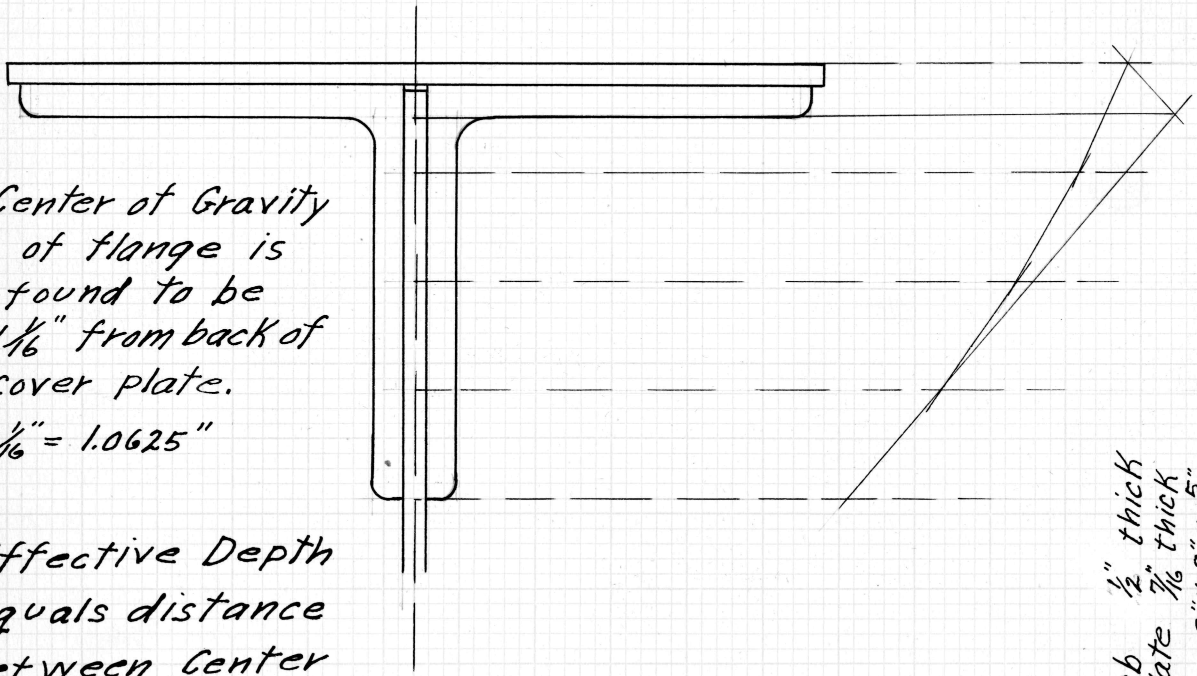
$$17000 \times 25.99 \times \frac{58.75}{12} = 2{,}165{,}800 \text{ lbs/ft.} =$$

2165.8 Kips.

The value of 2165.8 Kips is found at 19.4 feet from the support.

Only one cover plate is necessary and will be extended one foot beyond point where theory requires; but in order to avoid interfering with flange stiffners with a flange splice, the cover plate will be extended two feet beyond where theory requires. Cover plate will be started 17 feet from each end.

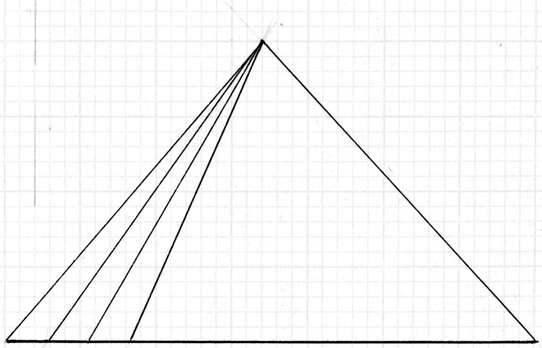
Center of Gravity of Flange.



Center of Gravity of flange is found to be $1\frac{1}{16}$" from back of cover plate.

$1\frac{1}{16}" = 1.0625"$

Effective Depth equals distance between Center of gravity axis of Flanges.



Scale $\frac{1}{4}" = 1$ in.

Web $\frac{1}{2}"$ thick
Cover Plate $\frac{7}{16}"$ thick
Angles $8" \times 8" \times \frac{5}{8}"$

To find effective depth of Girder.

Distance between cover plates = 60 in.

Two $\frac{7}{16}"$ cover plates = 0.875 in.

$60 + 0.875 - 2 \times 1.0625 = 58.75" =$ effective depth.

Theoretic Pitch of Rivets in Flanges.

The position of the Live Load which causes the maximum shear is such that the first driver of the locomotive is just on the right of the section. Its weight on one rail is distributed over approximately forty two inches and is 25000 lbs. The allowance for impact is 73%, and the weight of the track supported by one girder is about 700 lbs., making a total load of 43950 lbs. or 1047 lbs per linear inch.

At section 0' which is at the support each flange is composed of two angles and one cover plate and the increment of flange stress per linear inch resisted by the flange alone is,

$$\frac{22.49}{25.99} \cdot \frac{234,850}{58.75} = 3478 \text{ pounds.}$$

The ratio $\frac{22.49}{25.99}$ is the ratio of the two flange angles and one cover plate to the total flange area. 234,850 pounds is the total vertical shear at section 0', and 58.75 inches is the effective depth.

The bearing of a ⅞ inch rivet in a ½ inch plate, with allowable bearing stress of 25000 pounds per sq. in. is 10,940 lbs.

The resultant of 1047 and 3478 is 3632, and the required pitch is $\frac{10940}{3632} = 3.01$ inches.

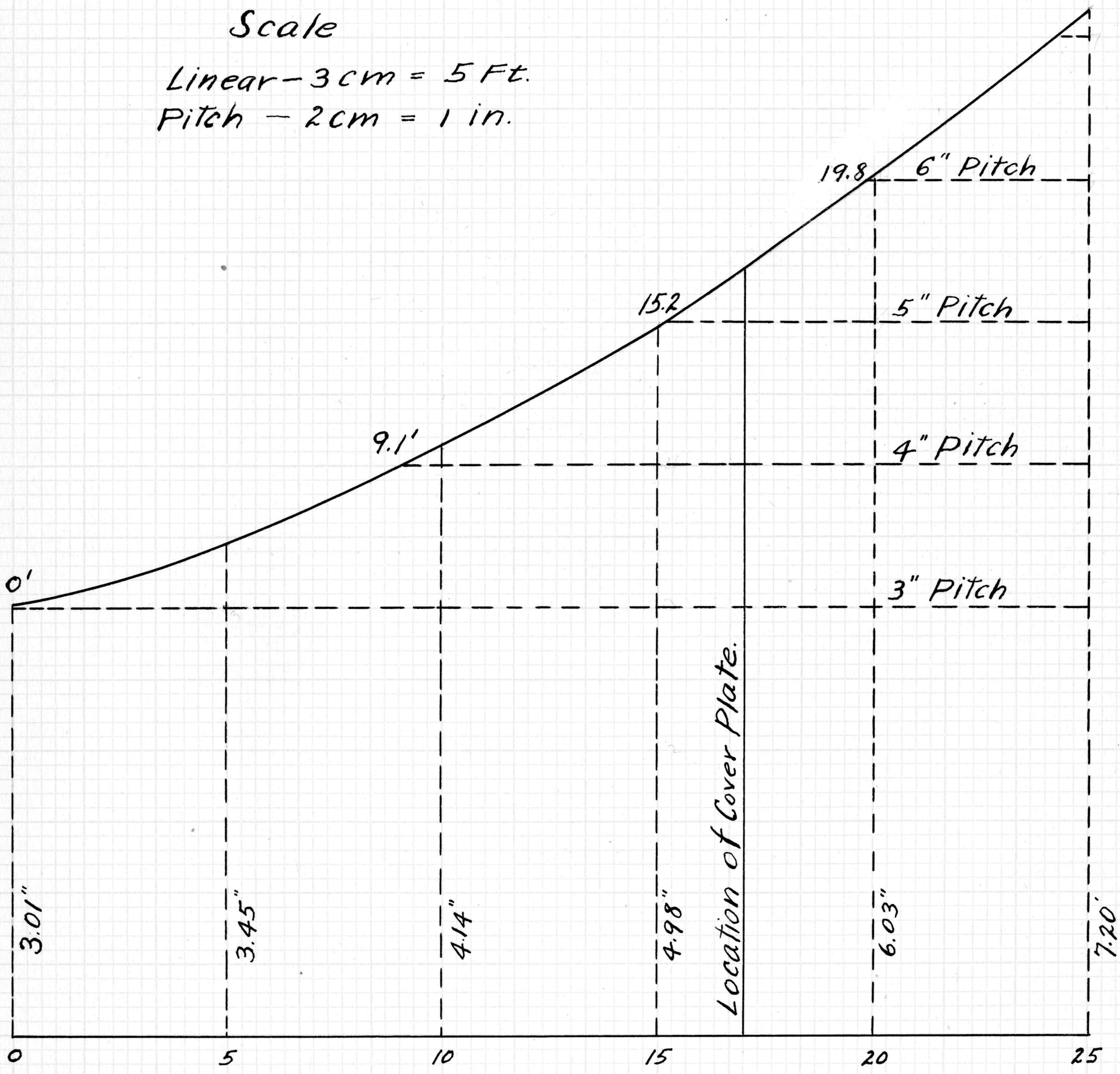
In a similar manner the pitch is determined at each section and the resutls laid off as ordinates, and a curve drawn thru the points.

Tabulation of Pitches at Sections

Section	0	5	10	15	20	25
Pitch	3.01	3.45	4.14	4.98	6.03	7.20

Curve of Theoretic Rivet Pitch in Flanges.

Scale
Linear – 3 cm = 5 Ft.
Pitch – 2 cm = 1 in.



Location of Flange Rivets.

Specifications:

The pitch of rivets shall not be less than three diameters when on the same line, nor less than two and one half times the diameter when staggered. The pitch in the direction of the stress shall never exceed six inches, nor sixteen times the thickness of the thinnest outside plate. When two or more thicknesses of plate are riveted together in compression members, the outer row of rivets shall not be more than four diameters from the side edge of the plate. No rivet hole center shall be less than one and one half diameters from the edge of a plate, and, whenever practicable, this distance is to be increased to two diameters.

According to specifications a $\frac{7}{8}$ inch rivet will be used, and a three inch pitch will be used at end of girder for ten feet. Extra rivets being placed at end on stiffeners. A four inch pitch will be used between sections ten and fifteen feet from each end, and a five inch pitch will be used between sections fifteen and twenty feet from each end. A six inch pitch will be used for the middle sections, twenty feet from each end inward.

There will be two rows of staggered rivets on each angle.

Rivets in the web stiffeners will be of two and one half inch pitch.

Rivets on web splice and near splice will be three inch pitch. A three inch pitch will be used for two feet on each side of splice.

A $2\frac{1}{2}$ inch pitch will be used in the flange angle splice.

Flange Splices.

Specifications:

Splices in flange angles must be avoided when possible. Where the span is long and flange splices are unavoidable, they must be so located that no two pieces of either the flange or the web shall be spliced within two feet of each other, and so that no flange and web splice shall occur at any point where there is not an excess of sectional area above the theoretical requirments.

Flange splices will be put in at the ends of the second cover plate, in order to take advantage of the extra strength at this point. The splice will therefore occur at points $17\frac{1}{2}$ feet from each end.

Each of the $8'' \times 8'' \times \frac{5}{8}''$ angles has a net area of $9.61 - 1.5 = 8.11$ square inches, and is to be spliced on by an $8'' \times 8''$ angle cut down to fit the face of the flange angle and to have at least the same area, this area can be furnished by a $8'' \times 8'' \times \frac{3}{4}''$ angle each leg being cut down to $7\frac{1}{4}$ inches.

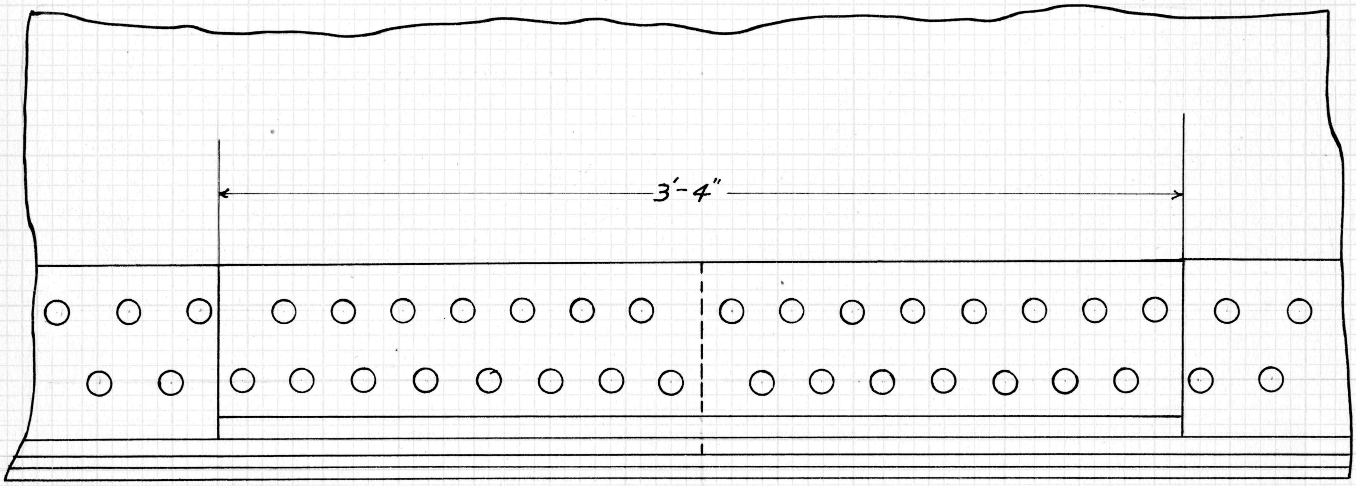
The single shear of a $\frac{7}{8}$ inch rivet is 7220 pounds.

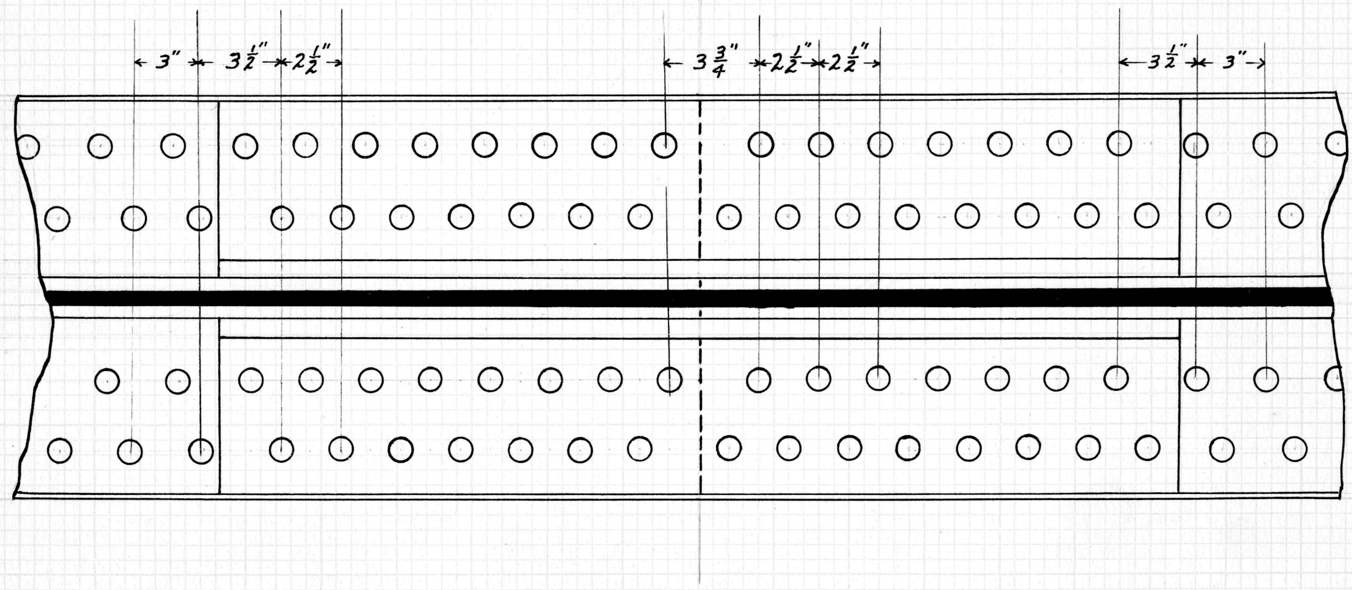
The shear at section 18 feet is 112000 lbs. this will require fifteen rivets, but sixteen will be used on each side of the splice, on each leg, making splice 3'4" long. A $2\frac{1}{2}$ inch pitch will be used.

Flange Splice.

Angles $8'' \times 8'' \times \frac{5}{8}''$
Splice Angles $7\frac{1}{4}'' \times 7\frac{1}{4}'' \times \frac{3}{4}'' \times 3'\text{-}4''$
Pitch of rivets – $2\frac{1}{2}''$

Scale
$1\frac{1}{2}'' = 1$ Ft.





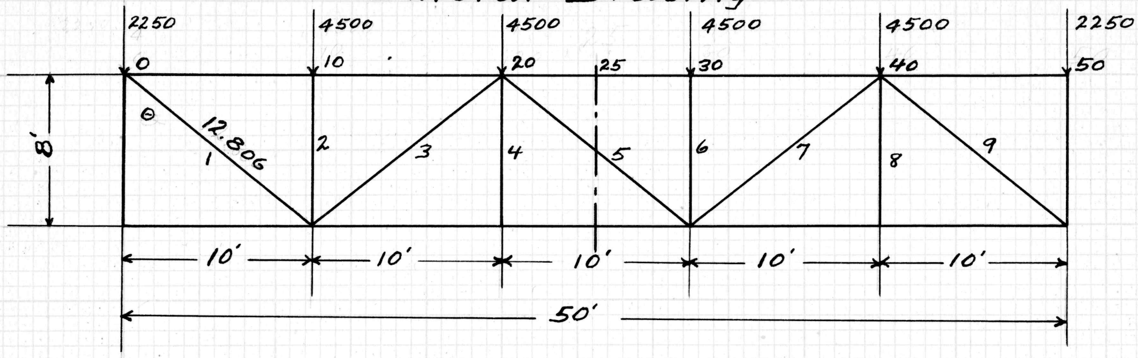
Lateral Bracing.

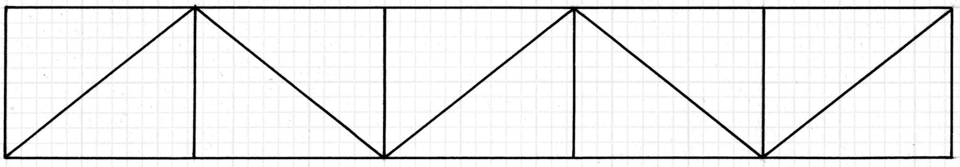
Specifications:

The lateral bracing shall be proportioned for a static wind load of 150 pounds per linear foot on each system. The system connected to the loaded flanges shall be proportioned also for a moving wind load of 300 pounds per linear foot. The compression flanges of the girder shall be so stiffened laterally that the unsupported length shall not exceed 12 times the width of the flange. All members shall be so proportioned that the tensile unit stress shall not exceed 17000 pounds per square inch, nor the compressive unit stress to exceed 17000 pounds per square inch reduced in proportion to the ratio of the length to the least radius of gyration of the section, by the following formulae;

$P = 17000/\left(1 + \frac{1}{11000} \cdot \frac{l^2}{r^2}\right)$, in which "p" is the permissible working stress per square inch in compression, l the length of piece in inches between centers of connections, and r the least radius of gyration of the section in inches. No compression member in the wind bracing shall have a length exceed 120 times its least radius of gyration. For members of any importance, more than two rivets are to be used for each connection. In field riveting the number of rivets found by the specified unit stresses shall be increased 25 percent if driven by hand, or 10 percent if satisfactory power riveters are used.

Lateral Bracing.





Sec. $\theta = 1.61$

Struts.

Dead Load 150 lbs. per liner ft.

Panel length 10 ft.

$150 \times 10 = 1500$ lbs = Dead Panel Load.

Live Load 300 lbs per linear ft.

Panel Length 10 ft.

$300 \times 10 = 3000$ lbs = Live Panel Load.

Diagonals.

Panel Load.

150 lbs. per linear ft = static wind load.
300 lbs per linear ft = moving wind load.
Panel length = 10 ft.

$(150+300)\,10 = 4500 = 4.5$ Kips.

$S = V \sec\theta \qquad \sec\theta = 1.61$

$$V = \frac{\text{Panel Load}}{\text{No. of Panels}}\left(1+2+\cdots \text{No. of Panels to right of section}\right)$$

$$S = V\sec\theta$$

$V_1 = \frac{4.5}{5}(1+2+3+4) = 9$

$S_1 = V_1 \sec\theta = 9 \times 1.61 = 14.49K = \pm 14500$ lbs.

$V_3 = \frac{4.5}{5}(1+2+3) = 5.4$

$S_3 = V_3 \sec\theta = 5.4 \times 1.61 = 8.694 K = \pm 8700$ lbs.

$V_5 = \frac{4.5}{5}(1+2) = 2.7$

$S_5 = V_5 \sec\theta = 2.7 \times 1.61 = 4.347 K = \pm 4400$ lbs.

S_7 & S_9 are respectfully equal to S_3 & S_1.

Member	1	3	5	Strut
Dead Load	4830	2900	1500	1500
Live Load	9670	5800	2900	3000
Maximum	± 14500	± 8700	± 4400	± 4500

Lateral Bracing

Larger angles answers the purpose of a larger number of smaller angles and uses fewer rivets and plates.

Using a $6'' \times 6'' \times \frac{3}{8}''$ angle, l the length of the diagonal equals 119 inches. The radius of gyration of a $6'' \times 6'' \times \frac{3}{8}''$ angle is 1.19 & $\frac{l}{r} = \frac{119'}{1.19}$ equals 100 which is less than 120, therefore can be used.

The sectional area of a $6'' \times 6'' \times \frac{3}{8}''$ angle is 4.36 square inches.

The specified column formulae

$p = 17000/(1 + \frac{1}{11000} \cdot \frac{l^2}{r^2})$ gives an average compression per square inch of 8905 pounds. Hence required area of end diagonal is

$\frac{14500}{8905} = 1.63$ square inches.

A $6'' \times 6'' \times \frac{3}{8}''$ angle gives an area of 4.36 sq. in. but on account of eccentricity and connections, this angle will be used as its thickness is least allowable.

Distance of the C. of G. of the angle to back of flange is 1.64 inches, (by Cambria) and I about neutral axis parallel to flange is 15.39 and corresponding radius of gyration = 1.88, then $\frac{l}{r} = \frac{119}{1.88} = 63$

The maximum compression on flange is

$$S' = \frac{14500}{4.36}\left(1 + \frac{63 \times 63}{11000}\right) = 4523 \text{ lbs per sq. in.}$$

compression. Where 4.36 equals the sectional area of a $6'' \times 6'' \times \frac{3}{8}''$ angle, and $63 = \frac{l}{r}$.

The bending moment due to eccentricity is $14500 \times 1.64 = 23780$.

By Article 117 Mechanics of Materials the compressive stress in the outer fiber is

$$S'' = \frac{23780 \times 1.64}{15.39 - \frac{14500 \times 119 \times 119}{9.6 \times 29000000}} = 2680 \text{ pounds per}$$

square inch. Total stress $= 4523 + 2680 = 7203$ pounds per square inch. (Since maximum stress is 17000 lbs. per sq. in. this will do.) This size will be used throughout since specifications require $\frac{l}{r}$ greater than 120.

The length of the lateral braces perpendicular to girder is 82", hence radius of gyration must not be less than $\frac{82}{120} = 0.68$ in.; as the stress is only 4500 pounds, a $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle will be large enough, this being the smallest allowed under specifications with $\frac{7}{8}$ inch rivets. The same size laterals will be used in the lower system.

Rivets.

The connecting rivets are in single shear. Aa they are field rivets their number will be increased 25 percent for hand riveting.

The shearing stress gives two rivets, eccentricity one rivet, and increasing 25 % equals four rivets for end diagonals.

Three rivets will be used in other diagonals.

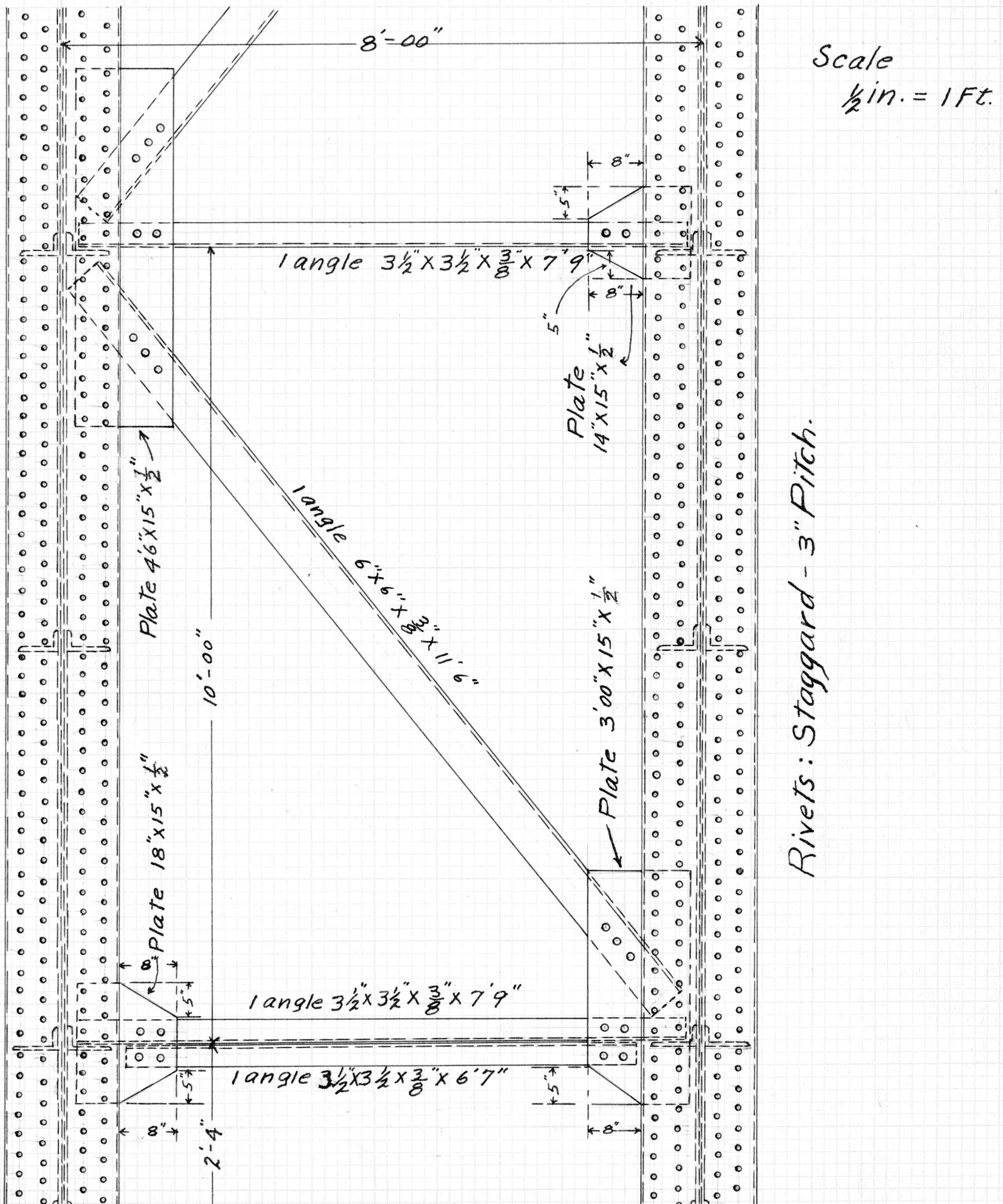
Longitudinal shear in each rivet is $\frac{14500}{4}$ equals 3825 which being, under the allowable shearing stress will be used. The allowable stress for field rivets being 5770.

The pitch will be three inches and the pitch line two and one half from back of flange.

Not less than three rivets will be used in any diagonal.

The end diagonals of upper system will be increased to ½ inch in thickness on account of adding 75 percent of tension due to compression area as theory requires.

Lateral Bracing.



Transverse Bracing.

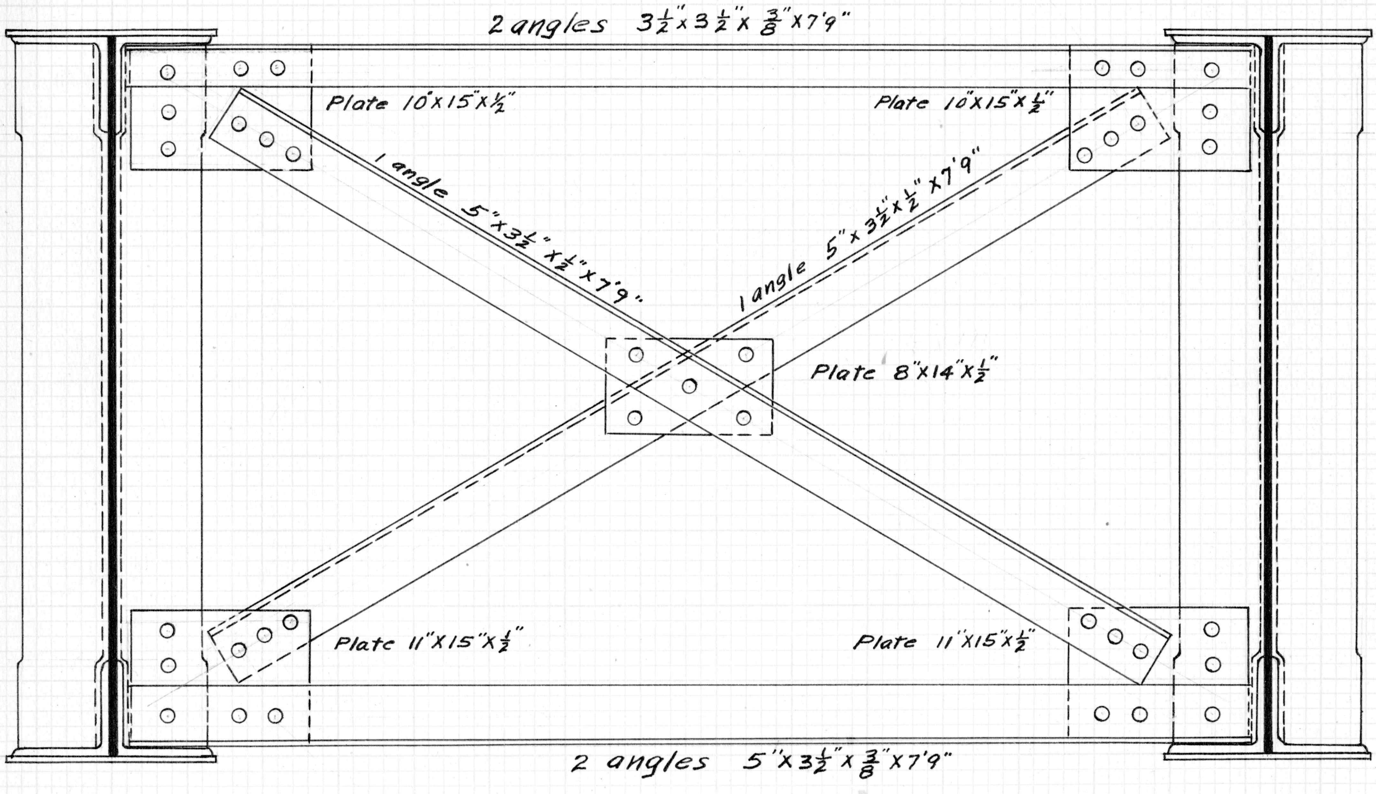
The object of the intermediate cross frames is to increase the general stiffness of the bridge. Each connection shall have three rivets, and where the diagonals cross they shall be riveted to a small connecting plate.

The end cross-frame must transfer the reaction of the upper lateral system to the support. This reaction is 18000 pounds. Each diagonal takes one half = 9000 pounds. This is easily carried by a $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle. But for stiffness in end frames which is very important, an angle with larger legs will be used; as an eight inch web flange is used a $3\frac{1}{2}$ inch leg would not give enough stiffness, so each end diagonal will be composed of one angle $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$.

The upper horizontal will be composed of two angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$. The lower horizontal of two angles $5'' \times 3\frac{1}{2} \times \frac{3}{8}''$.

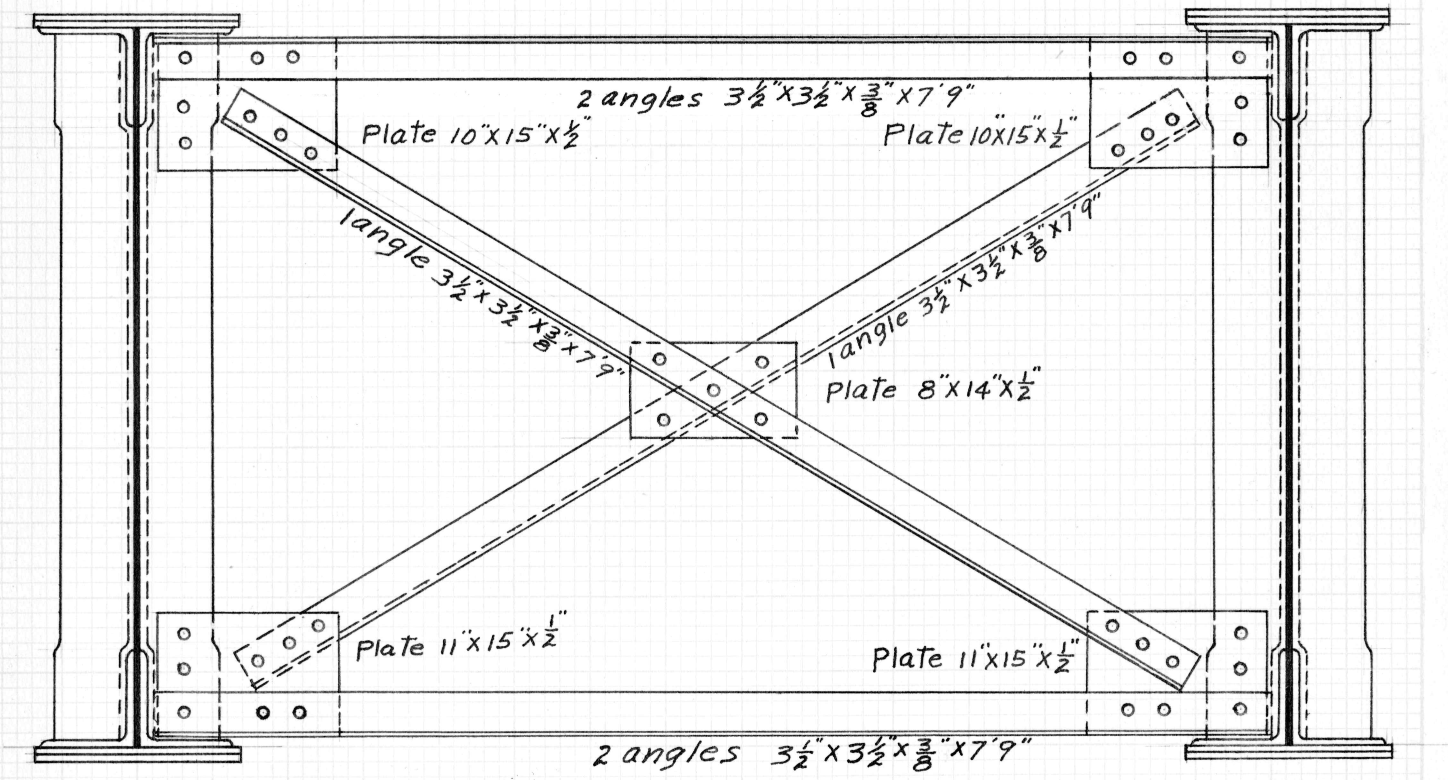
End Cross Frame.

Scale: $\frac{3}{4}$ In. = 1 Ft.



Intermediate Cross Frame.

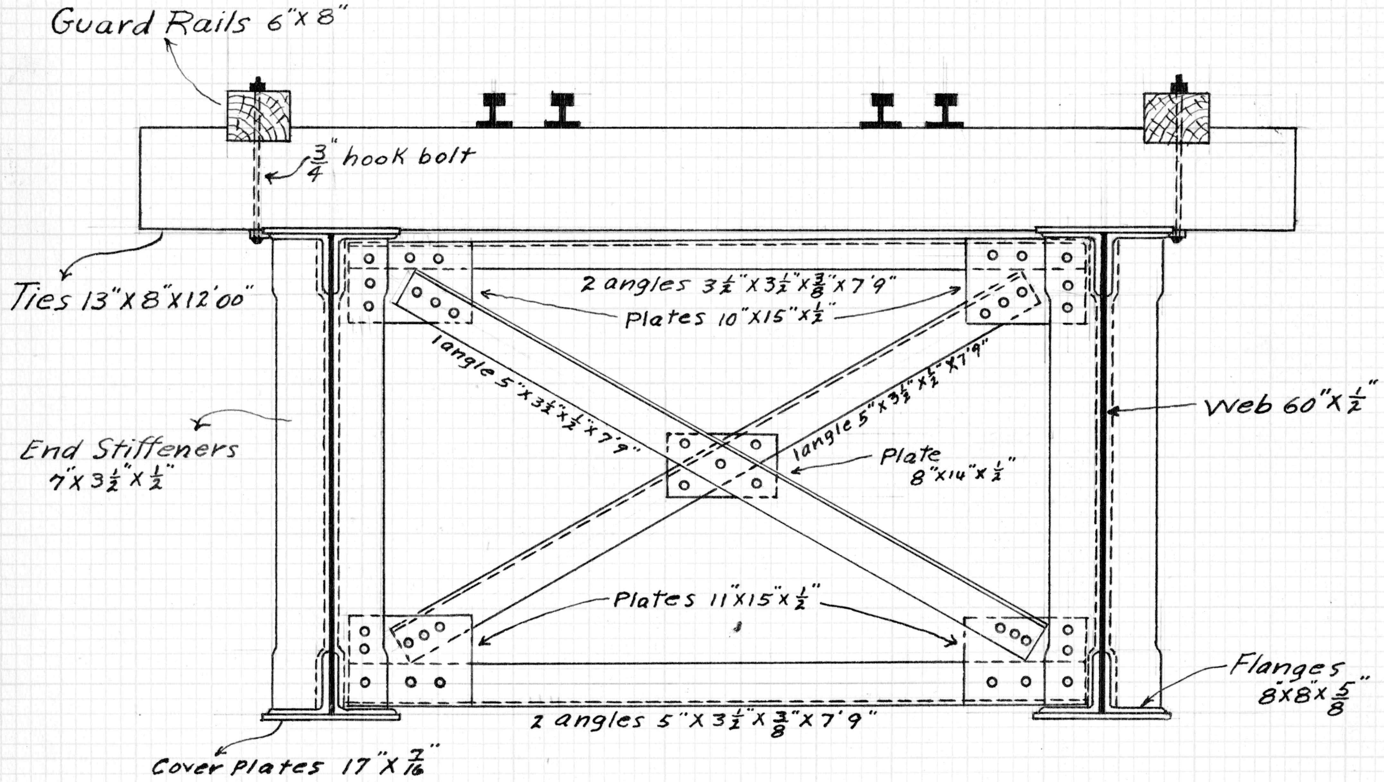
Scale: $\frac{3}{4}$" = 1 Ft.



End View

Showing Ties, Track, and Stringers.

Scale
1/2" = 1 Ft.



All rivets 7/8"

Bearings at Supports.

The Girder will rest on bearing plates, two at each end.

To allow for expansion one end will be fixed and the other free to move in the direction of expansion, longitudinally for a variation in temperature of 150 degrees Fahrenheit, but will be anchored against lifting or moving side ways.

The masonry pressure will not exceed over 400 pounds per sq. in.

$$\frac{234\,850}{400} = 588 \text{ sq. in.}$$

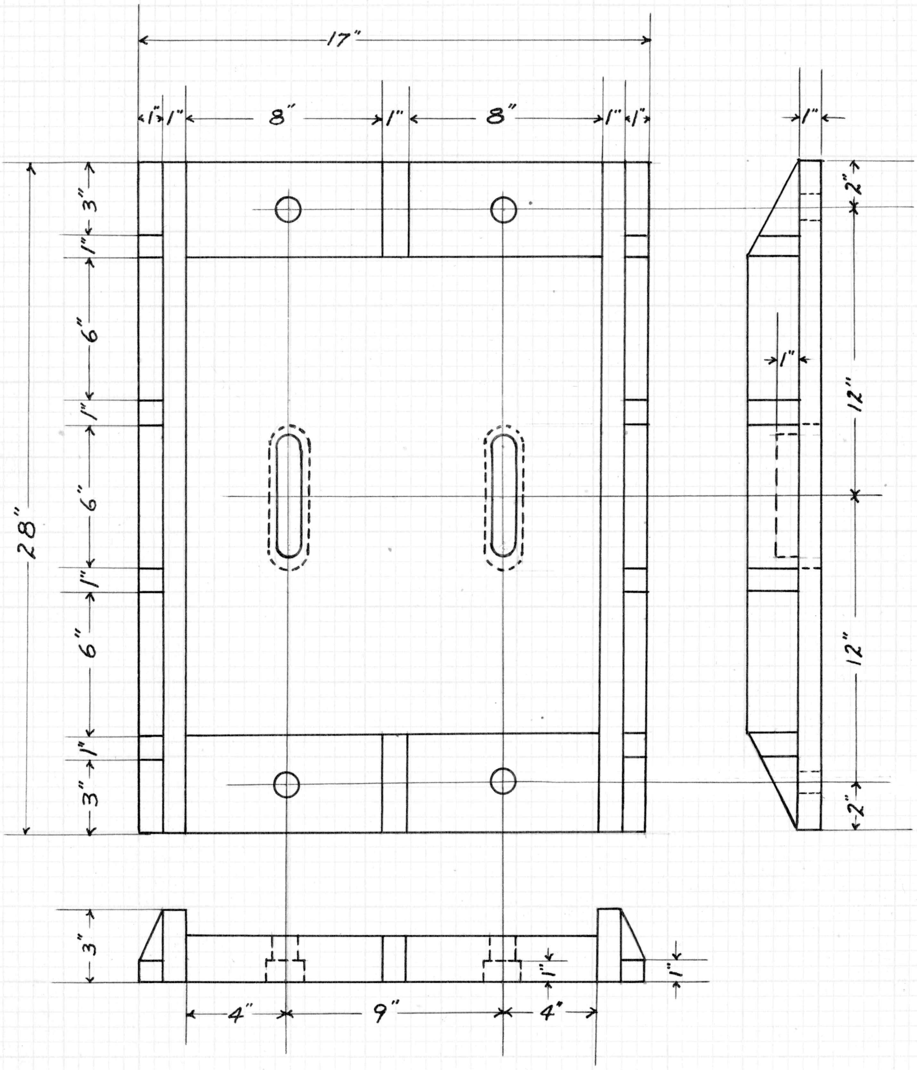
Each plate will extend from under the edge of the flange about two inches.

Plates will therefore be 21 inches wide.

The area of 588 sq. in. can be furnished by a 21" X 28" plate.

Bearing Plate

Free End



Scale
$1\frac{1}{2}" = 1$ Ft.

Casting