THESIS

DESIGN OF A TIMBER HOWE TRUSS HIGHWAY BRIDGE

Joseph Fox

Class of Nineteen Hundred Twenty One

Department of Civil Engineering

California Institute of Technology
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TABLE OF CONTENTS

INTRODUCTION

The span of the bridge was assumed as 100 feet. The type of bridge used is the timber Howe Truss. The height of truss was taken as 20 feet between center lines of top and bottom chords. The width was taken as 18 feet center to center of trusses. The truss was divided up into five panels 20 feet long.

It was designed according to the "General Specifications for Steel Highway Bridges" by Ketchum. For the live load for the floor and its supports, a load of 80 pounds per square foot of total floor surface or a 15 ton traction engine with axles 10 feet centers and 6 feet gage, two thirds of load to be carried by rear axles.

For the truss a load of 75 pounds per square foot of floor surface.

For the wind load the bottom lateral bracing is to be designed to resist a lateral wind load of 300 pounds per foot of span; 150 pounds of this to be treated as a moving load.

The top lateral bracing is to be designed to resist a lateral wind force of 150 pounds per foot of span.

The timber to be used in the bridge is to be Douglas fir.

The unit stresses used for timber are those of the American Railway Engineering Association.

DEAD AND LIVE LOAD STRESSES

Assumed weight of bridge 60,000#.

Dead Load per truss $\frac{60,000}{2} = 30,000\#$

Dead Load per panel $\frac{30,000}{5} = 6,000\#$

Assume 1/3 taken by top panel and 2/3 by bottom panel pt.

Top panel pt. load 1/3 6,000= 2,000#.

Bottom panel pt. load 2/3 6,000= 4,000#

Live Load 75#/ft. of floor surface.

75 X 18 X 20 =13,500# Total bottom panel pt. load.

DEAD LOAD STRESSES

Member	Index Stress	Multi-plier	Dead Load Stress
$L_0 L_1$	+12	$\frac{20}{20}$	+12
$L_1 L_2$	+18	"	+18
$L_2 L_3$	+18	"	+18
$U_1 U_2$	-12	"	-12
$U_2 U_3$	-18	"	-18
$L_0 U_1$	-12	$\frac{28.3}{20}$	-17
$U_1 L_1$	+10	1	+10
$L_1 U_2$	- 6	$\frac{28.3}{20}$	-8.5
$L_2 U_2$	+ 4	1	+4

LIVE LOAD STRESS

Member	Position of Loading for Live Load.	Live Load Stress	Dead Load Stress	Total
$L_0 L_1$	Entire Bridge	$\frac{12X13500}{6000} = \pm 27.0$	+42	+39
$L_1 L_2$	" "	$\frac{18X13500}{6000} = \pm 40.5$	+18	+58.5
$L_2 L_3$	" "	$\frac{18X13500}{6000} = \pm 40.5$	+18	+58.5
$U_1 U_2$	" "	$\frac{12X13500}{6000} = 27.0$	-12	-39
$U_2 U_3$	" "	$\frac{18X13500}{6000} = 40.5$	-18	-58.5
$L_0 U_1$	" "	$\frac{17X13500}{6000} = 38.2$	-17	-55.2
$U_1 L_1$	Up to L_1	$\frac{10X13.5}{5} = \pm 27.0$	+10	$37 + \frac{37X150}{300+80} = +51.6$
$L_1 U_2$	Up to L_2	$\frac{6X13.5X28.3}{5 \quad 20} = 22.9$	-8.5	-31.4
$U_2 L_2$	Up to L_2	$\frac{6X13.5}{5} = +16.2$	+4	$20.2 + \frac{16.2X150}{300+60} = +27$
Counter $U_1 L_2$	From left up to L_1	$\frac{1X13.5X28.3}{5 \quad 20} = 3.82$	Not needed	
Counters $U_2 L_3$-$L_2 U_3$	From left up to L_2	$\frac{3X13.5X28.3}{5 \quad 20} = 11.46$		-11.46

DESIGN OF FLOOR

The flooring is to consist of 12 X $2\frac{1}{2}$ planking to be laid diagonally on stringers. An additional covering of planks 12 X $1\frac{1}{2}$ to be laid transversely on bottom layer. The lower planking shall be laid with $\frac{1}{2}$" openings. A coating of coal tar is to be applied to top surface of lower plank and bottom surface of upper plank. The lower planks to be securely spiked to each joist with 40d spikes. (Table 5 Ketchum).

The spacing of the stringer taken at 2 ft. The proportion of the concentrated load live load carried by one joist taken as equal to spacing of joist in feet divided by four feet.

Max. mom. occurs with rear wheel in center of stringer.

$$P=10000 \times 2/4 \; 5000\#.$$

Max. mom. $= \frac{5000}{4} \times 20 \times 12 = 300{,}000$ in./lbs.

Uniform dead load-

$\frac{12 \times 4}{144} \times 30 = 10\#/ft^2$. wt. of flooring.

Assume 6"X 16" stringer.

$\frac{5\frac{1}{2} \times 15\frac{1}{2}}{144} \times 30 = 16\#/ft^2$. assumed wt. of stringer.

Mom. $= \frac{(10+16) \times (20)^2 \times 12}{8} = 15{,}600$ in./lbs.

$315{,}600 = 1/6 \times 1500 \times 5\frac{1}{2} d^2$

$d^2 = 229$ horizontal shear $3 \times (5000 \; 26 \times 20) = 97\#/\square''$

$d = 15.2"$

Use 6"X 16" nominal size beam for stringers.

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DESIGN OF FLOOR BEAM

Weight of flooring- 10 X 20 X 18 --------------	3600
Weight of joists - 20 X 20 X 10 --------------	4000
Assumed wt. of beam 90 X 18 --------------	1620
Wt. of 6" X 4" wheelguard - 2 X 6 X 4 X 30 X 20	- 200
Total weight---	9420#
Live weight 80 X 20 X 18--------------------	28800#
Total	38220#

Mom. $= \frac{38{,}220 \times 20 \times 12}{8} = 1{,}147{,}000$ in./lbs.

$1{,}147{,}000 = 1500S$

$S = 762$

Use 2 beams S for one beam $\frac{762}{2} = 381$

Use 2 10"X 16" beams for floor beams.

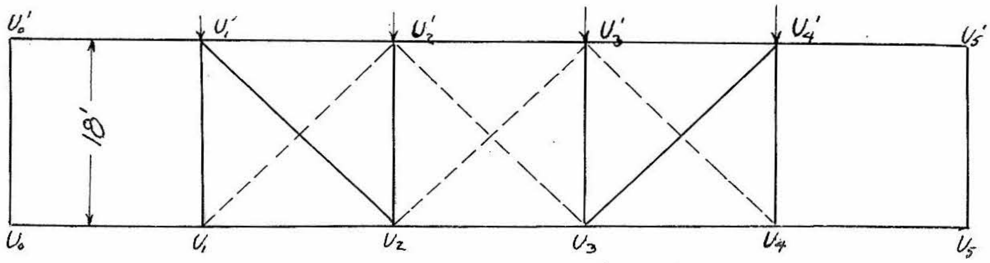
Intensity of horizontal shear $v = \frac{3 \times 38{,}220}{2 \times 2 \times 9\frac{1}{2} \times 15\frac{1}{2}} = 97.3\#/\square''$

Floor beams to be creosoted.

Total wt. of flooring and joists 7,600 X 4 = 30,400#.

Total wt. of floor beams $(\frac{9\frac{1}{2} \times 15\frac{1}{2}}{144} \times 30 \times 18)8 = 4{,}400\#$.

DESIGN OF TOP LATERAL BRACING.



The diagonal rods are to be tension rods.

To be designed for a wind load of 150#/ft. of span.

Panel pt. load 20 X 150 = 3000#.

Heavy line truss will be in action with loads as shown

Member	Index Stress	Multiplier	Stress
$U_2' U_2$	-3	1	-3
$U_1' U_2$	+3	$\frac{26.9}{18}$	+4.48
$U_1' U_1$	-6	1	-6

Member $U_1' U_2$
Stress 4,480#.
Initial tension 3,000#
Total 7,480#

$$A = \frac{7,480}{16,000} = .47 \square''$$

Use $\frac{7}{8}$"ø

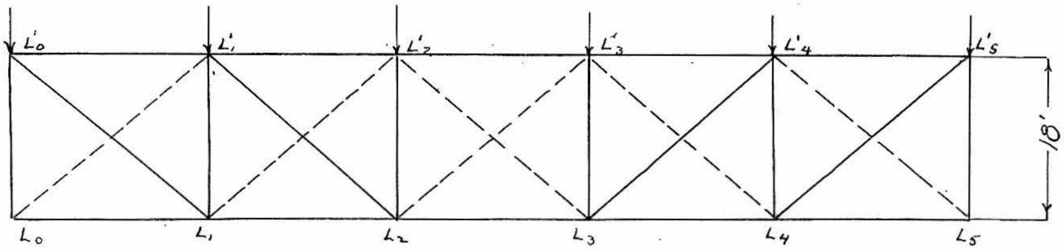
Use $\frac{3}{4}$"ø in panel $U_2 U_3$

Member $U_1' U_1$ Try $5\frac{1}{2}$" for d (cannot be less than 4")

$$A = \frac{6000}{1500\left(1 - \frac{18}{\frac{60 X 5\frac{1}{2}}{12}}\right)} = 11.5 \square''$$

Use 6" X 6" beam for $U_1' U_1$ - $U_2' U_2$

DESIGN OF BOTTOM LATERAL BRACING



5 panels @ 20' = 100'

The diagonal rods are to be tention rods.

To be designed for wind load of 300#/ft. of span, 150 pounds of this to be treated as moving load.

Heavy line truss in action under loads as shown.

Member	Index Stress	Multiplier	Dead Load Stress	Live Load Stress	Total
L L	+1.5	$\frac{26.9}{18}$	+2.24	$(\frac{6}{5} \times 1.5)\frac{26.9}{18} = 2.7$	+4.94
L L	+3	$\frac{26.9}{18}$	+4.48	$(\frac{10}{5} \times 1.5)\frac{26.9}{18} = 4.48$	+8.96
L L				$(\frac{3}{5} \times 1.5)\frac{26.9}{18} = 1.35$	+1.35

Member L_1L_2
Stress 4,940#
Initial tension 3000#
Total 7490#
Use $\frac{7}{8}$"ϕ

$A = \frac{7490}{16000} = .47$ □"

Member $L_o'L_1$
Stress 8960#
Initial tension 3000#
Total 11,960#
Use 1"ϕ

$A = \frac{11960}{16000} = .746$ □"

Use $\frac{3}{4}$"ϕ for member $L_2'L_3$.

DESIGN OF STEEL WEB MEMBERS

Member L_1U_1
Stress 51,600#
Allow 5000# for initial tension.
Total Stress 56,600#
Area of cross section of steel $\frac{56,600}{16,000} = 3.53\square''$
Use 2-$1\frac{1}{2}$"ϕ

Member U_2L_2
Stress 27,000#
5000# initial tension
32,000# Total Stress
$A = \frac{32000}{1000} = 2.00\square''$
Use 2-$1\frac{1}{8}$"ϕ

DESIGN OF TIMBER MEMBERS

Member L_0U_1
Stress-55,200#
Try 10"X12" Weight $= \frac{9\frac{1}{2} X 11\frac{1}{2} X 30}{144} = 22.7\#/ft.$
Component perpendicular to member 22.7Xcos 45° = 16.1#/ft.
M = 1/8 16.1(28.3) X12 = 19,400 in./lbs.
S = 209.39 $s = \frac{M}{S} = \frac{19,400}{209.39} = 93\#/\square''$

$$A = \frac{55,200}{1500\left(1 - \frac{28.3}{\frac{60X9\frac{1}{2}}{12}} - 93\right)} = 108.8\square''$$

A of 10"X12" = 109.25 □"

Member U_2U_3
Stress-58,500#
Try 12"X12" Weight $= \frac{11\frac{1}{2} X 11\frac{1}{2}}{144} X 30 = 27.6\#/ft.$
M = 1/8 27.6(20) X 12 = 16,550 in./lbs. $s = \frac{16.550}{253.48} = 66\#/\square''$

$$A = \frac{58,500}{1500\left(1 - \frac{20}{\frac{60}{12} X 11\frac{1}{2}} - 66\right)} = 59.5\square''$$

Try 10"X12" M = 1/8 22.7(20) X 12 = 13,600 in./lbs. $s = \frac{13,600}{209.39} = 65\#/\square''$

$$A = \frac{58,500}{1500\left(1 - \frac{20}{\frac{60X9\frac{1}{2}}{12}} - 65\right)} = 84\square''$$

A of 10"X12" = 109.25 □" OK

Use 10"X12" beam

Member U_1U_2
Stress 39,000#
Use same size beam as in U U - a 10"X12" beam

Member L_1U_2
Stress-31,400#
Try 8"X8" Weight $= \frac{7\frac{1}{2} X 7\frac{1}{2} X 30}{144} = 11.7\#/ft.$

Component perpendicular to member $11.7 \times \cos 45° = 8.3\#/ft.$
$M = 1/8 \times 8.3(28.3) \times 12 = 9{,}950$ in./lbs. $s = \frac{9{,}950}{10.31} = 142\#/\square''$

$$A = \frac{31400}{1500\left(1 - \frac{28.3}{\frac{60 \times 7\frac{1}{2}}{12}} - 142\right)} = 138\,\square''$$

Try a 10"X10"
Component perpendicular to member $\frac{9\frac{1}{2} \times 9\frac{1}{2}}{144} \times 30 \times \cos 45° = 11.9\#/ft.$

$M = 1/8 \times 11.9\ (28.3) \times 12 = 14{,}400$ in/lbs $s = \frac{14{,}400}{142.89} = 101''/\square''$

$$A = \frac{31400}{1500\left(1 - \frac{28.3}{\frac{60 \times 9\frac{1}{2}}{12}} - 101\right)} = 62\,\square''$$

A of 10"X10" $= 90.25\,\square''$ OK

Use 10"X10" beam.

Members $L_2U_3 - L_3U_2$
Stress - 11,460#
Try 8"X8"

Component perpendicular to member due to weight $\frac{7\frac{1}{2} \times 7\frac{1}{2}}{144} \times 30 \times \cos 45° = 83\#/ft.$

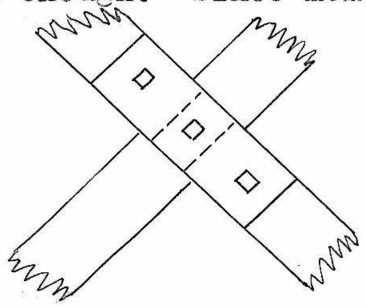
$M = \frac{1}{8} = 8.3 \times (28.3) \times 12 = 9{,}970$ in./lbs.

$s = \frac{9970}{70.31} = 142\,\#/\square''$

$$A = \frac{11460}{1500\left(1 - \frac{28.3}{\frac{60 \times 7\frac{1}{2}}{12}} - 142\right)} = 50.5\,\square''$$

Use 8"X8" for members.

Will have to butt against each other as chord members are not wide enough. Since members can act only in compression the purpose of gusset plate would be to hold members in place and would take no stress



Use a 8" X 16" X $\frac{1}{4}$" plate
Use 3 - 7/8" bolts.

For bottom chord--area required $\frac{58500}{1500} = 39\,\square''$

Use 10"X8" beam.

DESIGN OF END JOINT

Depth of toe

$n = p \sin^2\theta + q \cos^2\theta$

Where n = normal intensity on inclined plane.
p = " " " ends of fibers.
q = " " across fibers

p=1800 q = 285
n=1025 Fig.19 Dewell-Timber framing.

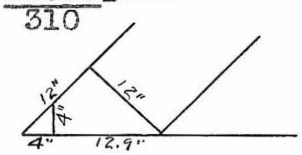
$\frac{3900}{1025} = 38.1$ □" needed for bearing

$\frac{38.1}{10} = 3.8"$

Let depth be 4"

Area required for bearing between upper and lower chords.
Allowable stress perpendicular to grain 310#/"

$\frac{39000}{310} = 126$ □"



Total area available 12.9x10 = 129 □"

Depth of tables

Assuming three to be used

$\frac{3900}{3x10x1500} = .865"$

Use 7/8"x3" tables.

Assume 3 rivets per table

Stress in each rivet $\frac{39000}{3x3} = 4330\#$

Use 3 $\frac{3}{4}$" rivets per table

Thickness of Plate

For bearing against rivets $\frac{5}{16}$"

For shear $\frac{39000}{12000x10} = 0.325"$

For tension $\frac{39000}{16000x(10-3)} = .348"$

Use $\frac{3}{8}$" plate.

Moment of rotation of Tables

$\frac{.875 \times .375}{2} \times \frac{39000}{3} = 8,100$ in./lbs.

Stress in bolts $\frac{8100}{3\frac{1}{2}} = 2320\#$

Add stress due to pin in bolster.

$\frac{1}{4} \times \frac{1}{2} \times 800 \times 10'' = 1000\#$.

Total stress in two bolts = 3320#

Use 2 ½" bolts.

Stress in bolster equals horizontal component of stress in two diagonal bolts.

$5300 \times 2 \times .707 = 5610\#$

$\frac{5610}{800 \times 10} = .70$ (Two inch pins of extra heavy steel pipe working load of 800# per linear inch.)

Use 1-2" shear pin

Distance between table for shear

$\frac{39000}{3 \times 10 \times 150} = 8.65''$ or $8\frac{3}{4}''$

Washers--

Stress in bolts 3010# $\frac{3010}{310} = 10.05\square''$

$10.05\square'' + \pi(1/2 + 1/8)^2 = 10.3\square''$

Use $3\frac{1}{4}'' \times 3\frac{1}{4}''$ Washer

Thickness $\frac{1}{8} \times 3\frac{1}{4} = 3/8''$

10 required $3/8'' \times 3\frac{1}{4}'' \times 3\frac{1}{4}''$ steel washers

Washers for diagonal bolts

Area of $\frac{3}{4}''\phi = .4418$

$.4418 \times 16000 = 7070\#$ Allowable stress

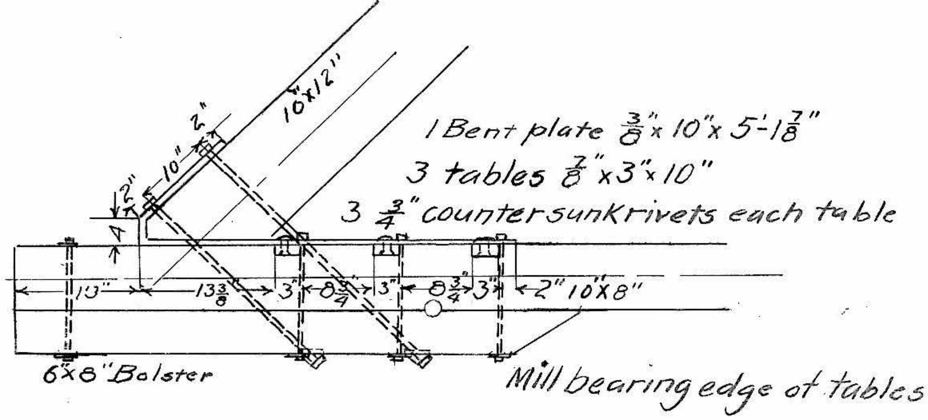
Horizontal comp. = $7070 \times \cos 45^\circ = 5000\#$

Vertical comp. = $7070 \times \sin 45^\circ = 5000\#$

Bearing area required for washer $\frac{5000}{310} = 16.1$ "

Use 4"X4" washer

Depth of dap in bolster $\frac{5000}{1500 \times 4} = .833$ or $\frac{7}{8}''$



Depth of toe **4"**

Tables 3" X $\frac{7}{8}$"

3 $\frac{3}{4}$" rivets

Distance between tables 8$\frac{3}{4}$"

$\frac{3}{8}$" Plate

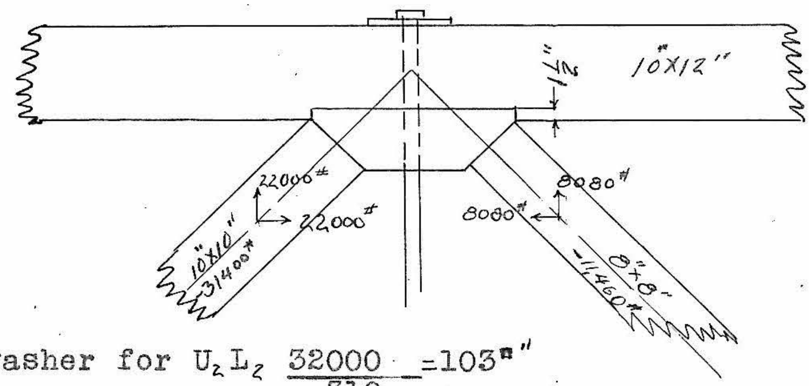
2 $\frac{1}{2}$" bolts at table to hold plate down

Use 8 bolts in all

Use 2 $\frac{3}{4}$" bolts for diagonal bolts

10 washers 8$\frac{1}{4}$"X8$\frac{1}{4}$"X$\frac{3}{8}$"

JOINT AT U_2



Area of washer for $U_2 L_2$ $\frac{32000}{310} = 103$ □"

Use 5"X10"X10½" washer

Required area for base of butt block

$\frac{22000+8080}{310} = 97$ □"

Length of butt block $\frac{97}{10} = 9.7$"or 9¾"

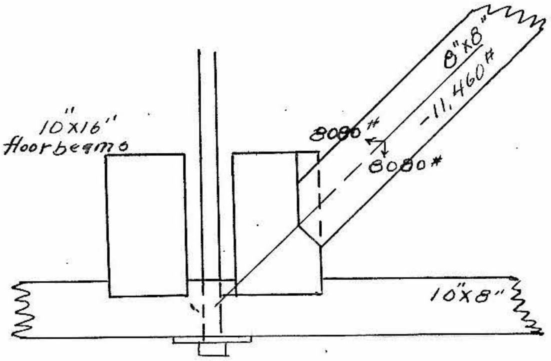
Depth of dap in chord $\frac{22000}{1500X10} = 1.48$"or 1½"

Use 2 $\frac{5}{8}$" drift bolts

350X9 = 3150# safe stress

Use $\frac{3}{8}$"X3¼"X3¼" washers

JOINT AT L_2



Area for base of butt block

$\frac{7555 + 8080}{310} = 57$ □"

Area of base of beam = 100 □"

Depth of dap in chord

$\frac{8080}{1500X10} = .54$"

JOINT AT U_1

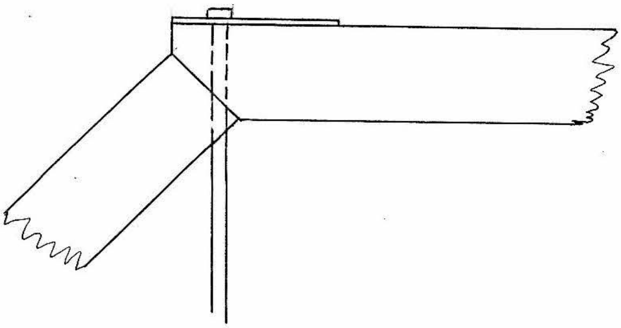
Bearing area required.

$$\frac{56,600}{310} = 183\square''$$

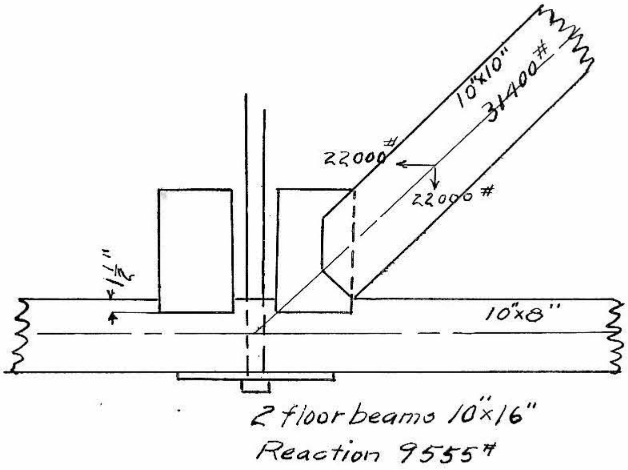
Allowing for bolt holes   gross area=187□"
12"X16" washer required
6X8" washer would be required for 1 bolt

Thickness $\frac{1}{8}$ X 8" = 1"

Use 1"X10X20" washer



JOINT AT L_1



Required area for bearing of L_1U_2 on block $\frac{31400}{1000} = 31.4\square''$

Actual area 10X10 = 100 □"

$\frac{22000 + 9555}{310} = 102\square''$ area of base of butt block

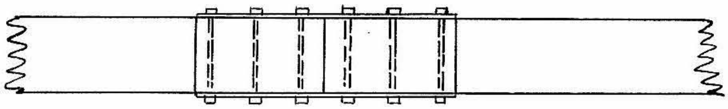
Area of base of beam 100□"

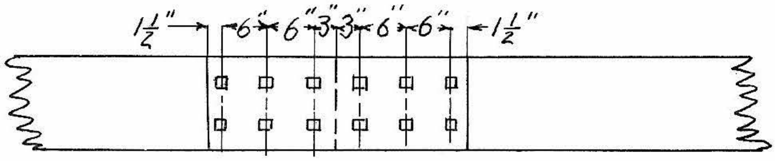
Depth of dap in chord

$$\frac{22000}{1500 X 10} = 1.47'' \text{ or } 1\tfrac{1}{2}''$$

DESIGN OF COMPRESSION SPLICE

Center of U_2U_3





2 ¼" X 8" X 2' 9" plate

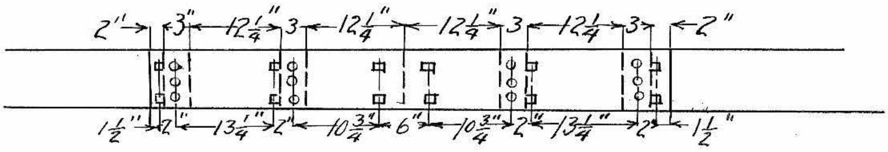
12 $\frac{5"}{8}$ bolts

DESIGN OF TENSION SPLICE

Bottom Chord

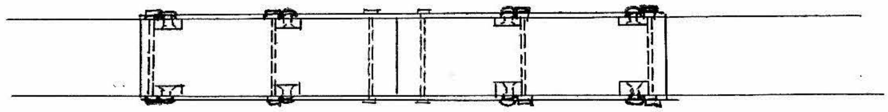
Stress L_2L_3 58,500# 8"X10" beam
Splices 35' from each end.

Side Elevation



2 $\frac{1}{2}$"X8" plates
8 $1\frac{1}{4}$"X3"X8" tables bearing edges milled

All rivets $\frac{3}{4}$"
All bolts $\frac{5}{8}$"



STEEL TABLED FISH PLATE SPLICE

Bearing area required for tables $\frac{58500}{1500} = 39"$

Total combined depth of tables $\frac{39}{2X8} = 2.44"$ or $2\frac{1}{2}"$

Use tables $1\frac{1}{4}$"X8" requires 8 tables in all.
Each table transmits $\frac{58500}{4} = 14625\#$ and requires 3 $\frac{3}{4}$" rivets as determined by shear for plate $\frac{1}{2}$" thick.
Net section of one plate $\frac{1}{2}(8-(\frac{3X1}{8}) = 2.69"$

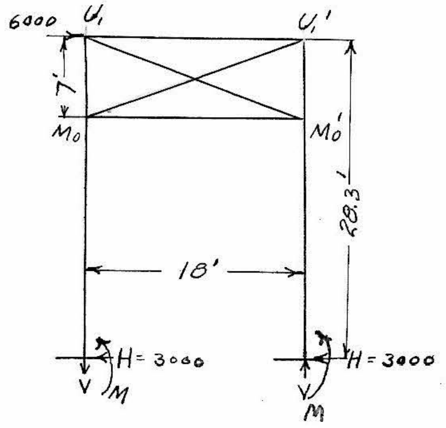
Net section of one plate required $\frac{58500}{2X16000} = 1.83$

Size of bolts required to resist moment on tables
$M = \frac{14625 \ X \ 5/6 \ \frac{1}{4}}{2} = 11450$in./lbs.

Tension in bolts $\frac{11450}{3\frac{1}{2}} = 3270\#$ Use 2 $\frac{5}{8}$" bolts

Distance required between tables for shear $\frac{58500}{4X8X150} = 12.2"$ or $12\frac{1}{4}"$

PORTAL BRACING



Assume zero stress in U M for conditions shown

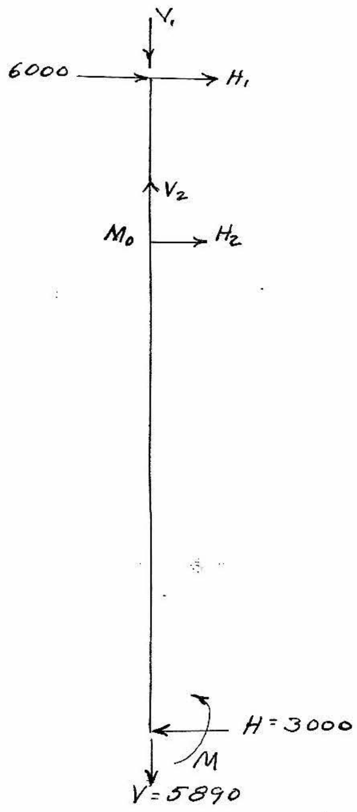
$$M - 3000 \times 10.65 = 0$$
$$M = 31{,}950 \text{ ft./lbs.}$$

$$6000 \times 28.3 - 2 \times 31{,}950 - 18V = 0$$
$$V = 5890\#$$

$$-31{,}950 + 3000 \times 28.3 - 7H_2 = 0$$
$$H_2 = +7564\#$$

$$6000 + H_1 + 7564 - 3000 = 0$$
$$H_1 = -10564$$

$$V_1 = 0$$
$$V_2 = V \qquad V_2 = 5890\#$$



Stress in Diagonal

$$5890 \times \frac{19.3}{7} = 16{,}250\#$$

Stress in $U_1 U_1' = -10{,}564\#$

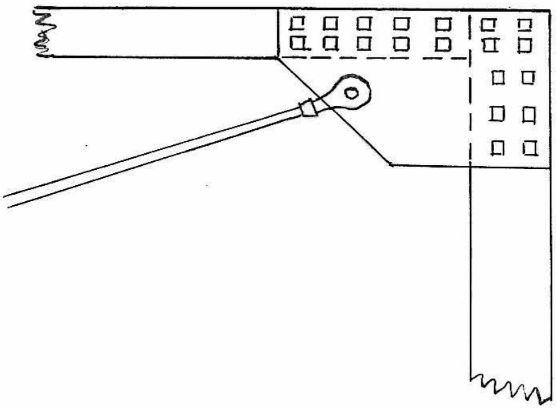
Stress in $M_0 M_0' = 7564 - \frac{5890 \times 18}{7} = -7586\#$

For diagonals $\frac{16250}{16000} = 1.02^{\square}$" use $1\frac{1}{4}$"ϕ upset

For $U_1 U_1'$ $\dfrac{10564}{1500\left(1 - \dfrac{19.3}{\frac{60}{12} \times 5\frac{1}{2}}\right)} = 23.5^{\square}$"

Use 6"X6" beam for $U_1 U_1'$ and $M_0 M_0'$

PORTAL BRACING CONNECTION



Use #4 clevis
2" pin

$$\frac{16,250}{2 \times 12000} = .677'' - \frac{11}{16}'' \text{ plate}$$

$$\frac{16250}{(2 \times 2 \;\; 2 \times \frac{11}{16} \;\; 12000} = .25'' \quad \frac{3}{4}'' \text{ distance}$$

of hole from edge of plate.

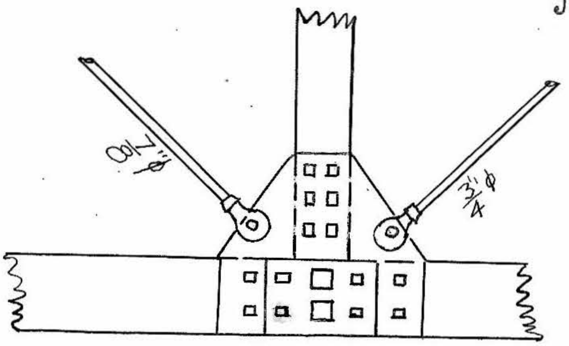
$$\frac{16,250 \times \frac{7}{19.3}}{1200} = 4.9 \text{ or } 5 \; \frac{7}{8}'' \text{lag screws}$$

to transmit vertical component.

$$\frac{16,250 \times \frac{18}{19.3}}{1200} = 12.6 \text{ or } 13 \; \frac{7}{8}'' \text{lag screws}$$

to transmit horizontal component.

TOP LATERAL BRACING CONNECTIONS



Joint at U_2

Lateral resistance of lag screws is 1200# for $\frac{7}{8}$" screws driven 5"

$\frac{7480}{1\frac{1}{2} X 12000} = .416"$ or ½" plate

$\frac{7480}{(2 X 1\frac{1}{2} + 2 X \frac{1}{2})} = .156$ or ¾" distance from hole to edge of plate.

Use a 1½" cotter pin

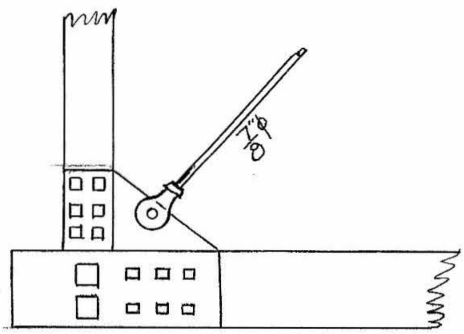
#3 clevis for ¾"φ
#4 clevis for $\frac{7}{8}$"φ

$\frac{7480 X \frac{18}{26.9}}{1200} = 5$ or 6 $\frac{7}{8}$"lag screws for strut connection.

$\frac{7480 X \frac{18}{26.9}}{6} = 835\#$ stress in each screw

$\frac{835}{(2X\frac{7}{8}+2X5)170} = .435"$ or 1" spacing of screws for shear on wood

$\frac{7480 X \frac{20}{26.9}}{1200} = 5$ or 8 $\frac{7}{8}$"lag screws to transmit horizontal component.



Joint at U_1

Use ½" plate

Use #4 clevis 1½" cotter pin

¾" from hole to edge of plate

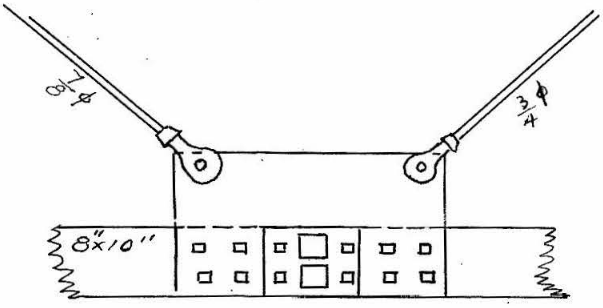
6 $\frac{7}{8}$"lag screws to transmit vertical component.

Use 6 $\frac{7}{8}$"lag screws to transmit horizontal component

Lag screws spaced 1"

BOTTOM LATERAL BRACING

Joint at L_2



Use #3 clevis for $\frac{3}{4}$" ϕ

Use #4 clevis for $\frac{7}{8}$" ϕ

Use $1\frac{1}{2}$" cotter pin

$$\frac{7940}{1\frac{1}{2} \times 12000} = .44 \text{ or } \tfrac{1}{2}\text{" plate}$$

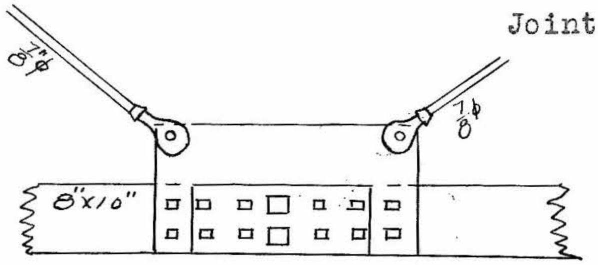
$$\frac{7940}{(2 \times 1\frac{1}{2} + 2 \times \frac{1}{2})\ 12000} = .165\text{"} - \tfrac{3}{4}\text{"}$$

from hole to edge of plate

$\frac{7940}{1200} = 6.6$ 7 $\frac{7}{8}$" lag screws 5" long

Use 12 $\frac{7}{8}$" lag screws

Joint at L_1



Use #4 clevis for $\frac{7}{8}$" ϕ

Use #4 clevis for 1" ϕ

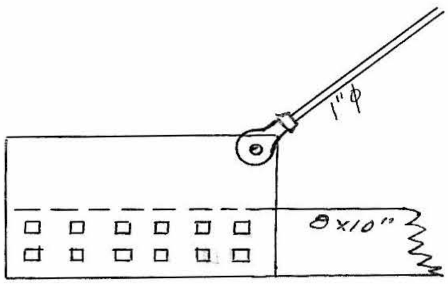
Use $1\frac{1}{2}$" cotter pin

$$\frac{11960}{1\frac{1}{2} \times 12000} = .665\text{" or } \tfrac{11}{16}\text{" plate}$$

$$\frac{11960}{(2 \times 1\frac{1}{2} + 2 \times \frac{11}{16})\ 12000} = .228 \text{ or } \tfrac{3}{4}\text{" from hole to edge of plate}$$

$\frac{11960}{1200} = 10$ Use 12 $\frac{7}{8}$" lag screws

Joint at L_0



Use #4 clevis

Use $1\frac{1}{2}$" cotter pin

$$\frac{11960}{1\frac{1}{2} \times 12000} = .665\text{"} - \tfrac{11}{16}\text{" plate}$$

$$\frac{11960}{12000(2 \times 1\frac{1}{2} + 2 \times \frac{11}{16})} = .220 \text{ or } \tfrac{3}{4}\text{" distance}$$

from hole to edge of plate

$\frac{11960}{1200} = 10$ Use 12 $\frac{7}{8}$" lag screws

ECCENTRIC MOMENT

The outside vertical rod is to be at a greater distance from the center of the top chord than the other vertical rod. This is to compensate for the eccentric moment due to the top lateral connections being off center.

Eccentric moment developed at joint U_2 3000X3 = 9000 in./lbs.

Stress in U_2L_2 = 27,000#. Stress in one rod=13,500#.

$\frac{9000}{13,500}$ = .67" distance that rod is to be moved off center.

Eccentric moment developed at joint U_1 6000X3 18000 in./lbs.

Stress in U_1L_1 = 51,600#. Stress in one rod=25,800#.

$\frac{18000}{25800}$ = .7" distance that rod must be moved off center.

CAMBER

The top chord is to be cambered by increasing the length $\frac{3}{16}$" in 10 feet.

ESTIMATE OF WEIGHT

Total wt. of flooring and joints	30,400
Total wt. of floor beams	4,400
Total wt. of chords-	
Upper chord $2\left(\frac{10X12}{144} X 60X30\right) = 3000$	
Lower chord $2\left(\frac{8X10}{144} X 100X30\right) = 3333$	
Total 6333	6,333
Total wt. of diagonal web members-	
$2\left(\frac{10X10}{144} X 28.3X30\right) = 1178$	
$2\left(\frac{8X8}{144} X 28.3X30\right) = 756$	
$2\left(\frac{10X12}{144} X 20.3X30\right) = 1412$	
Total 2 X 3346 =	6,692
Total wt. of top lateral bracing-	
$\frac{4X6X6}{144} X 18X30 = 480$	
4X2.04X27 = 221	
2X1.50X27 = 81	
Total 782	782
Total wt. of vertical web members-	
2 (2X6.008X21) = 484	
2 (2X3.38X21) = 287	
Total 2 X 771 =	1,542
Total wt. of bottom lateral diagonals-	
4X2.04X28 = 221	
4X2.67X27 = 288	
2X1.50X27 = 81	
Total 590	590
Total wt. of portal bracing-	
$\frac{4X6X6}{144} X 18X30 = 540$	
4X4.173X19.3 = 323	
Total 863	863
Add 15% of 60,000 for details	
.15X60,000 = 9000	9,000
TOTAL =	60,602#