AN INVESTIGATION OF THE STRESSES IN A PIN CONNECTED PRATT TRUSS

Thesis for the Degree of B. S.

MICHIGAN STATE COLLEGE

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An Investigation of the Stresses

in a Pin Connected Pratt Truss

A Thesis Submitted to

The Faculty of

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of

AGRICULTURE AND APPLIED SCIENCE

by

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Candidate for the Degree of

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I wish to acknowledge the valuable aid and services of Professor C. A. Miller in making the completion of this thesis possible in its present form.

Investigation

The purpose of this thesis will be to determine the stress in the various members of the bridge, such as the stringers, floor beams, chord members, web members, bracing, and lattice work.

The bridge is located about three miles west of Lansing, Michigan and crosses the Grand River at a point near Deepdale Cemetery. It was built in 1891 by R. D. Wheaton & Co. of Chicago, Ill.

The bridge is a pin connected Pratt truss composed of eight sixteen foot panels making a clear span of one hundred and twenty-eight feet. It also has two beam approaches, the one on the north end being fourteen and one half feet long and the one on the south end being twenty-nine and one quarter feet long, making the total length one hundred and seventy-one and three quarters feet. The distance between trusses is fifteen feet, center to center. The roadway is fourteen feet wide. The distance between chords is twenty feet, center to center.

This type of bridge is no longer used and at the present time the County Road Commission is planning to erect a more modern and safe structure.

SPECIFICATIONS

These specifications are taken from the Standard Specifications for Highway Bridges of the American Association of Highway Officials and the American Railway Engineering Association, and the Specifications for Steel Highway Bridges of the American Society of Civil Engineers.

Impact - Live load stresses, except those due to sidewalk loads and centrifugal, tractive, and wind forces, shall be increased by an allowance for dynamic, vibratory, and impact effects. The amount of this allowance or increment is expressed as a fraction of the live load stresses and shall be determined by the formula $I = \frac{50}{l + 125}$ where I is the impact fraction and l is the length of the loaded span in feet.

Lateral Forces - Spans of 150 feet and less shall be designed to resist a lateral force of 300 pounds per linear foot on the loaded chord and 150 pounds per linear foot on the unloaded chord. For Spans of more than 150 feet, for each additional 30 feet of span there shall be added 10 pounds per linear foot for the loaded chord and 5 pounds per linear foot for the unloaded chord.

Allowable Stresses -

Tension, net section	16,000 #/sq. in.
Compression (one diameter)	16,000
Compression, gross section $15,000 - 50 \frac{l}{r}$	
l - length of the member in inches	
r - least radius of gyration in inches	
but not to exceed	12,500
Bending in extreme fibers of rolled shapes, built sections and girders, net section	16,000
Bending in extreme fibers of pins	24,000
Shear in plate-girder and I-beam webs, net section	12,000
Shear in pins and shop-driven rivets	12,000

Strength of Lattice-bars - Latticing of compression members shall be proportioned to resist shearing stress normal to the member not less than that calculated by the formula: $R = \frac{Pl}{4000y}$

R - normal shearing stress, in pounds
P - strength of column as a compression member, in pounds
l - length of column, in inches
y - distance from neutral axis to extreme fiber, in inches

End Stiffeners - Over the end bearings of plate girders, there shall be stiffener angles, the outstanding legs of which shall extend as nearly as practicable to the outer edge of the flange angles. End stiffeners shall be proportioned for bearing on the outstanding legs of the flange angles, no allowance being made for the portions of the legs fitted to the fillets of the flange angles. End stiffeners shall be arranged, and shall be a sufficient number of rivets in their connection to the web, to transmit the entire end reaction to the bearings. They shall not be crimped.

Intermediate Stiffeners -

Webs shall be stiffened by angles riveted thereto in pairs on opposite sides, with outstanding legs not exceeding sixteen times their thickness, nor less than two inches plus one-thirtieth of the depth of the girder. Intermediate stiffeners shall be placed at points of concentrated loading and at intervals not exceeding the depth of the web, nor six feet.

NOTE !

As I was unable to obtain the plans for this bridge all measurements were made by myself.

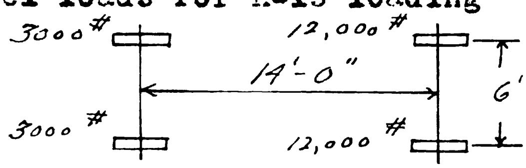
Not knowing what loading this bridge was designed for I used the H - 15 loading. The bridge evidently was not intended for such a heavy loading as it is badly stressed in some members.

Live Loading

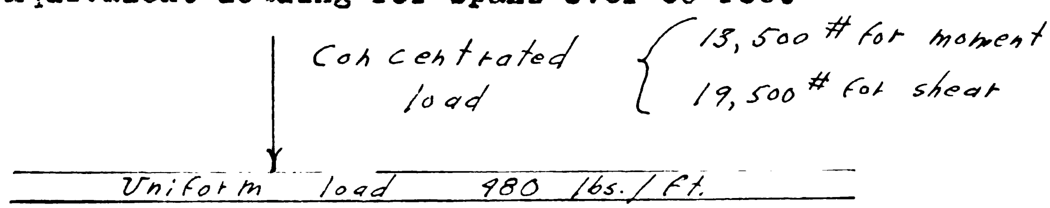
This bridge shall be considered a Class A bridge - One carrying normally heavy traffic units and the occasional passage of specially heavy loads.

For a class A bridge the H-15 loading should be used.

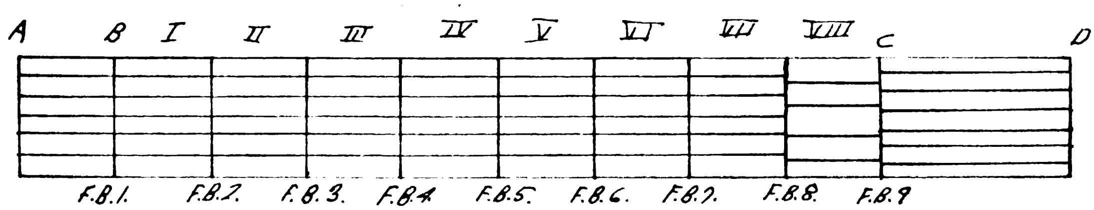
Wheel loads for H-15 loading



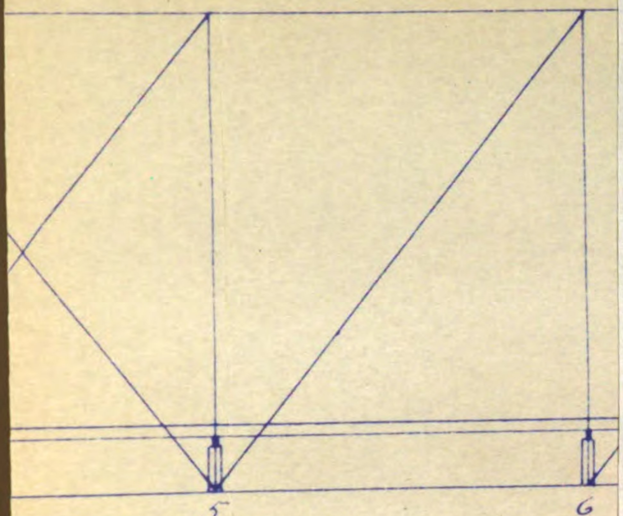
Equivalent loading for spans over 60 feet



Floor plan of the entire bridge




CHICAGO ILL


5
6

Floor System

Planks

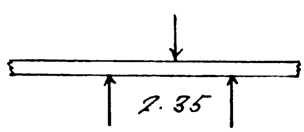
The planks are 2" x 4" and are placed side by side with the two inch side down.

The fraction of a wheel load to each stringer is found by dividing the spacing of the stringers by 4.0 - This formula found in the Specifications for Steel Highway Bridges.

The width of distribution $= 0.7(2D + W)$

D = the distance in feet from the center of the near support to the center of the wheel.

W = the width of the wheel or tire in feet.



$\frac{S}{4} \times 12{,}000 = \frac{2.35}{4} \times 12{,}000 = 7050$ #

$E = .7(2D + W)$

$E = .7(2.35 + 1.25)$

$E = 2.52$ ft.

Live load B. M.

$\frac{795}{2} \times \frac{2.35}{2} \times 12 = 5610$ in. lb.

$\frac{12{,}000}{2.52 \times 6} = 795$ lb.

Load over 6 planks

Impact B. M.

$\frac{50}{1+125} \times 5610 = 2244$ in. lb.

Dead Load B. M.

$\frac{wl^2}{8} = \frac{150}{3x6} \frac{(2.35)^2}{8} \times 12 = 69$ in. lb.

Total B. M.

7923 in. lb.

$$\frac{M}{s} = \frac{I}{c} = \frac{bd^2}{6}$$

$$\frac{7923}{s} = \frac{2 \times (4)^2}{6}$$

$s = 1490$ #/sq. in.   Allowable 2000 lb./sq. in.

The planks are O. K.

Stringers Panel VIII

The stringers in panel eight are composed of four R. I. B. 12" x 5" - 35 #/ft. Section modulus - 37.8 in.^{3} and two channels 12" x 3" - 25 #/ft. Section modulus - 23.9

The spacing of the stringer is variable but the maximum is three feet.

Interior stringers

Fraction of a wheel load to each stringer is

S/4 = 3/4 = .75

Maximum live shear

12,000 x .75 = 9,000 #

Maximum dead shear

3.00 x 1/3 x 4.5 x 8 + 35 x 8 = 316 #

Impact shear

$\frac{50}{16 + 125}$ x 9,000 = 3,190 #

Total shear 12,506 #

Stress

$\frac{12,506}{12 \times .423}$ = 2,440 #/sq. in.

Allowable 12,000 #/sq. in.

Maximum live B. M.

$\frac{1}{4}$ x 9,000 x 16 x 12 = 432,000 in. lb.

Maximum dead B. M.

1/8 x 39.5 x $(16)^2$ x 12 = 15,200 in. lb.

35 channel
4.5 Floor
39.5

Impact B. M.

432,000 x $\frac{50}{141}$ = 122,000 in. lb.

Total Bending Moment

569,200 in. lb.

Section modulus required

$$\frac{569,200}{16,000} = 35.6 \text{ in.}^3$$

The beam used has a section modulus of 37.8 in.^{3}

so the stringer is O. K.

Outside stringers panel eight

Dead load on outside stringers

1.5 x 1/3 x 4.5 = 2.25 #/ft.

Maximum dead shear

2.25 x 8 + 25 x 8 = 218 #

Maximum live shear - outside foot considered not loaded

$$\frac{3.0 - 1.0}{3.0} \times 12,000 = 8,000 \#$$

Impact shear

$$8,000 \times \frac{50}{141} = 2840 \#$$

Total shear 11,058 #

Stress

$$\frac{11,058}{12 \times .337} = 2,730 \#/\text{sq. in.}$$

Allowable 12,000 #/sq. in.

Maximum live B. M.

$\frac{1}{4}$ x 8,000 x 16 x 12 = 384,000 lb. in.

Maximum dead B. M.

$1/8 \times 27.25 \times (16)^2 \times 12 = 10,500$ lb. In.

Impact B. M.

$$\frac{50}{141} \times 384,000 = 136,000 \text{ lb. in.}$$

Total Bending Moment

530,500 lb. in.

Section Modulus required

$$\frac{530,500}{16,000} = 33.2 \text{ in.}^3$$

The channels used have a section modulus of 23.9 but it isn't likely they are loaded as calculated.

Stringers - Beam approach at the South end

This approach is 29.25 feet long and is composed of six R. I. B. 12" x 5" - 35 #/ft. Section modulus - 37.8 in^3 and two channels 12" x 3" - 25 #/ft. Section modulus - 23.9

Spacing

They are spaced two feet, center to center.

Interior stringers

Fraction of a wheel load to each stringer

$\frac{S}{4} = \frac{2.0}{4.0} = .5$

Maximum live shear

$12{,}000 \times .5 = 6{,}000$ #

Maximum dead shear

$2 \times 1/3 \times 4.5 \times \frac{29.25}{2} + 35 \times \frac{29.25}{2} = 555.9$

Impact shear

$\frac{50}{29.25 + 125} \times 6{,}000 = 1950$ #

Total shear 8,505.9 #

Stress

$\frac{8{,}505.9}{12 \times .428} = 1660$ #/sq. in.

Allowable 12,000 #/sq. in.

Maximum live B. M.

$\frac{1}{4} \times 6{,}000 \times 29.25 \times 12 = 527{,}000$ lb. in.

Maximum dead B. M.

$1/8 \times 38.00 \times (29.25)^2 \times 12 = 43{,}700$ lb. in.

Impact B. M.

$\frac{50}{154.25} \times 527{,}000 = 171{,}000$ lb. in.

Total Bending Moment

746,700 lb. in.

Section Modulus required

$$\frac{746,700}{16,000} = 46.7 \text{ in.}^3$$

R. I. B. sections have a section modulus of 37.8 in.^{3}

Outside stringers - beam approach at the south end.

Dead load

1.0 x 1/3 x 4.5 = 1.5 #/ft.

Maximum dead shear

$$1.5 \times \frac{29.25}{2} + 25 \times \frac{29.25}{2} = 383 \#$$

Maximum live shear

$$\frac{2.0 - 1.0}{2.0} \times 12,000 = 6,000 \#$$

Impact shear

$$6,000 \times \frac{50}{154.25} = 1950 \#$$

Total shear 8,338 #

Stress

$$\frac{8,333}{12 \times .387} = 1790 \#/\text{sq. in.}$$

Maximum live B. M.

$$\tfrac{1}{4} \times 6,000 \times 29.25 \times 12 = 527,000 \text{ lb. in.}$$

Maximum dead B. M.

$$1/8 \times 26.5 \times (29.25)^2 \times 12 = 34,100 \text{ lb. in.}$$

Impact B. M.

$$\frac{50}{154.25} \times 527{,}000 = 171{,}000$$

Total Bending Moment   732,100 lb. in.

Section modulus required

$$\frac{732{,}100}{16{,}000} = 45.3 \text{ in.}^3$$

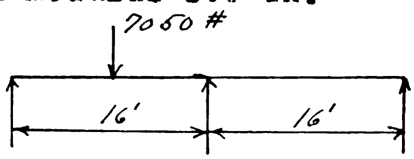
The channels used have a modulus of 23.9 . Again the channels are seemingly over-stressed but it isn't likely that a wheel will come as close to the edge as I have figured.

Stringers AB, panel I, panel II, panel III, panel IV, Panel V, panel VI, panel VII.

These stringers are continuous over two panels, AB & panel I, etc.

The five interior stringers are R. I. B.3s. They are 6" x 3-3/8" - 12.5 #/ft. Section modulus 7.3 in.^{3}

The outside stringers are channels, 6" x 2" - 10.5 #/ft. Section modulus 5.0 in.^{3}



Fraction of a wheel load to each stringer

$$\frac{S}{4} = \frac{2.35}{4} = .587$$

$.587 \times 12{,}000 = 7050$ #

The maximum shear and bending moment occur when a wheel load is in the middle of one span.

From Pocket Companion - page 179

Maximum live shear = .594 W = .594 x 7050 = 4190 #

Maximum live B. M. = .203 Wl = .203 x 7050 x 192

= 274,800 in. lb.

Maximum dead shear

2.35 x 1/3 x 1/6 x 4.5 x 6 x 8 + 12.5 x 8 = 128.2 #

Impact shear

$$\frac{50}{141} \times 4190 = 1490 \text{ \#}$$

Total shear = 5808.2

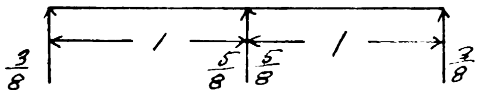
Stress

$$\frac{5303.2}{6 \times .23} = 4210 \text{ \#/sq. in.}$$

Allowable 12,000 #/sq. in.

Dead Bending Moment $w = 12.5 + 2.35 \times 1.5 = 16.03$

The reactions for a uniform load are as shown.



$3/8 \times 16.03 \times 16 \times 8 - 16.03 \times 8 \times 4 = 256$ ft. lb.

$256 \times 12 = 3080$ in. lb.

Impact B. M.

$$274{,}800 \times \frac{50}{141} = 97{,}500 \text{ in. lb.}$$

Total Bending Moment 378,440 in. lb.

Section modulus required

$$\frac{375{,}330}{16{,}000} = 23.5 \text{ in.}$$

The beam used has a section modulus of 7.3 in.^{3}

This is very severely over-stressed.

Stringers - Outside

Dead load on outside stringers

1.17 x 1/3 x 4.5 = 1.76 #/ft.

Maximum dead shear

1.76 x 8 + 10.5 x 8 = 93 #

Maximum live shear

Wheel load $\frac{2.3 - 1.0}{2.3}$ x 12,000 = 6,780 #

From Pocket Companion

Maximum live shear = .594 W = .594 x 6,780 = 4025 #

Maximum live B. M. = .203 Wl = .203 x 6,780 x 192

= 264,000 in. lb.

Impact shear

$\frac{50}{141}$ x 4025 = 1428 #

Total shear 5551 #

Stress

$\frac{5551}{6 \times .314}$ = 2950 #/sq. in.

Allowable 12,000 #/sq. in.

Maximum dead B. M.

3/8 x 12.26 x 16 x 8 - 12.26 x 3 x4 = 2350 in. lb.

Impact B. M.

264,000 x $\frac{50}{141}$ = 93,600 in. lb.

Total Bending Moment 359,950 in. lb.

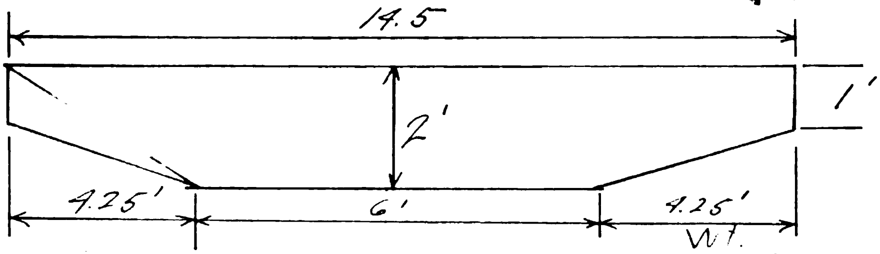
Section modulus required

$\frac{359,950}{16,000}$ = 22.5 $in.^3$

The channels used have a section moculus of 5.0 $in.^3$ They are very badly over-stressed.

Floor Beams Nos. 3, 4, 5, 6, 7.

Web $\frac{5''}{16}$. Flange $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times 3/8''$. Three and one half inch leg horizontal. End stiffeners $2'' \times 2'' \times \frac{1}{4}''$.



Wt.

$29 \times 7.2 = 209$

$14.5 \times 30.60 = 444$

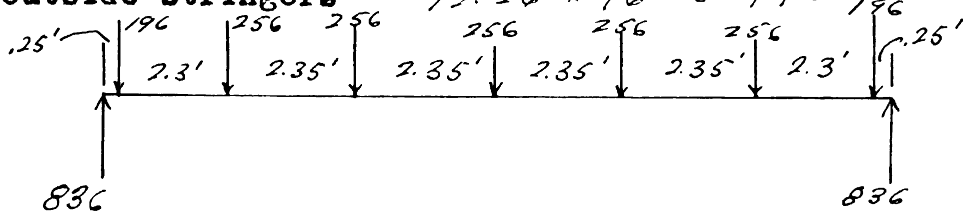
$2 \times 3.9 = 6.38$

659.38

Dead load from stringers

Interior stringers $16.03 \times 16 = 256$

Outside stringers $12.26 \times 16 = 196$



Dead Shear $= 836 + \frac{659.38}{2} = 1165.69\#$

Dead B. M. $= 836 \times 7.25 - 196 \times 7 - 256(4.7 + 2.35) + \frac{659.38}{14.5} \times \frac{(14.5)^2}{8}$

$= 4079$ Ft. lbs.

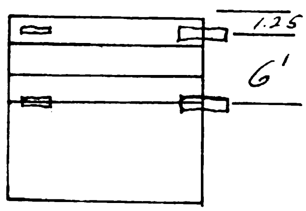
Live load from stringers

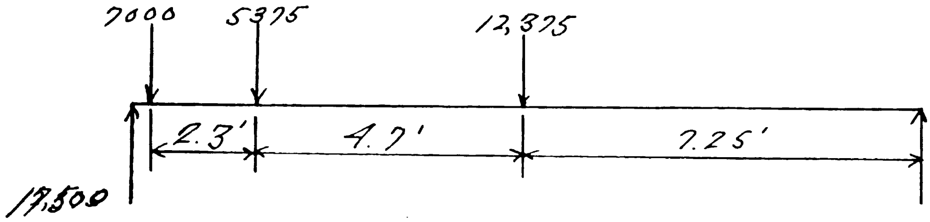
For Shear

$$\frac{2.3 - 1.0}{2.3} \times \left(12{,}000 + \frac{2}{16} \times 3{,}000\right) = 7{,}000\ \#$$

$12{,}375 - 7{,}000 = 5{,}375$

$12{,}000 + 375 = 12{,}375$





Maximum Live Shear

$$\frac{12,375 \times 7.25 + 5,375 \times 11.95 + 7,000 \times 14.25}{14.5} = \underline{17,500} \ \#$$

Impact Shear

$$17,500 \times \frac{50}{32 + 125} = 5,570 \ \#$$

Total Shear

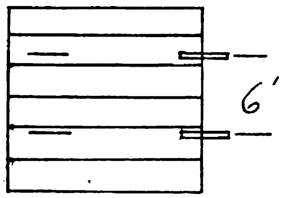
$$17,500 + 5,570 + 1165.7 = 24,235.7 \ \#$$

Stress

$$\frac{24,235.7}{12 \times \frac{5}{16}} = 6480 \ \#/\text{sq. in.}$$

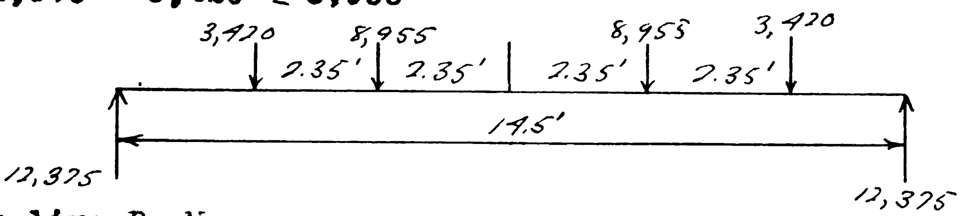
Allowed 12,000 #/sq. in.

Live load from stringers for B. M.



$$12,375 \times \frac{.65}{2.35} = 3,420$$

$$12,375 - 3,420 = 8,955$$



Maximum live B. M.

$$12,375 \times 7.25 - 8,955 \times 2.35 - 3,420 \times 4.7 = 52,550 \text{ ft. } \#$$

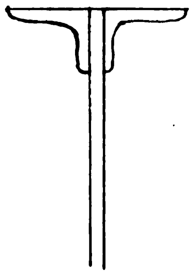
Impact B. M.

$$52,550 \times \frac{50}{32 + 125} = 16,700 \text{ ft. lbs.}$$

Total B. M.

$$52,550 + 16,700 + 4,079 = 73,329 \text{ Ft. lbs}$$

Tension Flange



y for $3\frac{1}{2}" \times 2\frac{1}{2}" \times 3/8" = .66 \quad A = 2.11$

$24 - 2 \times .66 = 22.63$

$$A = \frac{73,329 \times 12}{16,000 \times 22.63} = 2.42 \text{ sq. in.}$$

Area needed $2.42 - 1/8 \times$ web area

$$2.42 - \frac{24 \times 5/16}{8} = 1.48 \text{ sq. in.}$$

Area supplied $2 \times 2.11 - 2 \times 1 \times 3/8 = 3.47$ sq. in.

Compression Flange

Allowable stress $16,000 - 150 \frac{l}{b}$

$$16,000 - 150 \frac{14.5 \times 12}{7.31} = 12,430 \text{ \#/sq. in.}$$

$$A = \frac{73,329 \times 12}{12,430 \times 22.63} = 3.12 \text{ sq. in.}$$

Area needed $3.12 - \frac{24 \times 5/16}{8} = \cancel{3.47}$ 2.18 sq. in.

Area supplied $2 \times 2.11 = 4.22$ sq. in.

Pitch of Flange rivets

$p = \frac{rh}{V}$ — r - allowable bearing stress
h - dist. between C. G's.
V - Maximum shear

$$P = \frac{6060 \times 22.63}{24,235.7} = 5.66$$

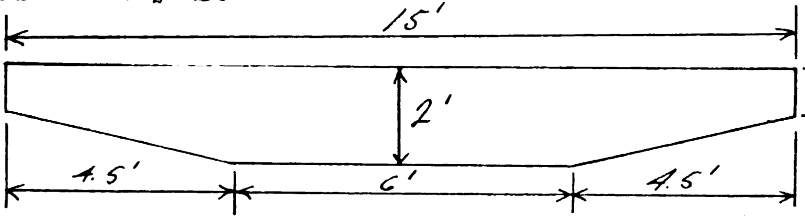
Five inches was used so the pitch is O. K.

End stiffeners $2" \times 2" \times \frac{1}{4}"$ were used.

Intermediate stiffeners

No intermediate stiffeners were used, However they should have been used at points of concentrated loading and at intervals not to exceed two feet.

Floor Beam # 2.



Wt.
30 x 7.2 = 216
15 x 30.6 = 459
2 x 3.19 = 6.38
681.38

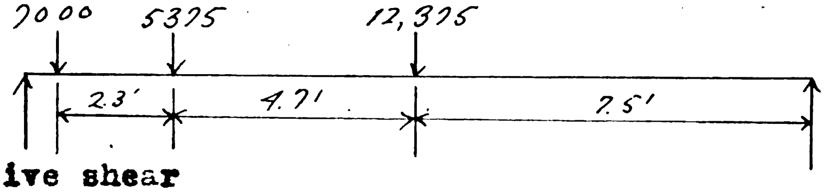
Dead load from stringers same as # 3, 4, 5, 6, 7.

Dead Shear $836 + \frac{681.38}{2} = 1176.7$ #

Dead B. M.

$$836 \times 7.5 - 196 \times 7 - 256(4.7 + 2.35) + \frac{681.38}{15} \times \left(\frac{15}{8}\right)^2$$
$$= 4370 \text{ ft. lbs.}$$

Live load from stringers for Shear



Maximum live shear

$$\frac{12,375}{2} + \frac{5375 \times 12.2}{15} + \frac{7000 \times 14.5}{15} = 17,327 \text{ \#}$$

Impact shear

$$17,327 \times \frac{50}{157} = 5,520 \text{ \#}$$

Total Shear

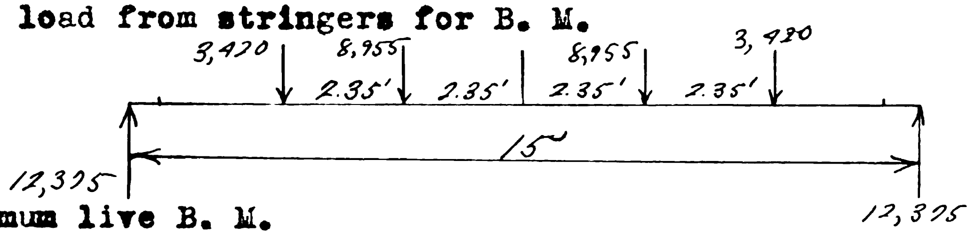
$$17,327 + 5,520 + 1,176.7 = 24,023 \text{ \#}$$

Stress

$$\frac{24,023.7}{12 \times 5/16} = 6400 \text{ \#/sq. in.}$$

Allowable 12,000 #/sq. in.

Love load from stringers for B. M.



Maximum live B. M.

$$12,375 \times 7.5 - 8,955 \times 2.35 - 3,420 \times 4.7 = 55,650 \text{ ft. \#}$$

Impact B. M.

$$55,650 \times \frac{50}{157} = 17,750 \text{ ft. lbs.}$$

Total B. M.

$$55,650 + 17,750 + 4,370 = 77,770 \text{ ft. lbs.}$$

Tension Flange

$$A = \frac{77,770 \times 12}{16,000 \times 22.68} = 2.58 \text{ sq. in.}$$

Area needed 2.58 - .94 = 1.64 sq. in.

Area supplied 2 x 2.11 - 2 x 1 x 3/8 = 3.47 sq. in.

Compression Flange

Allowable stress $16,000 - 150 \frac{l}{b}$

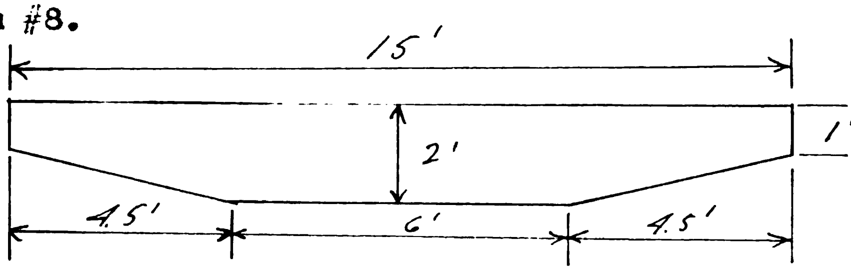
$$16,000 - 150 \frac{15 \times 12}{7.31} = 12,300 \text{ \#/sq. in.}$$

$$A = \frac{77,770 \times 12}{12,300 \times 22.68} = 3.35 \text{ sq. in.}$$

Area needed 3.35 - .94 = 2.41 sq. in.

Area supplied 2 x 2.11 = 4.22 sq. in.

Floor Beam #8.



Dead load from stringers

Interior stringers 16.03 x 8 = 128

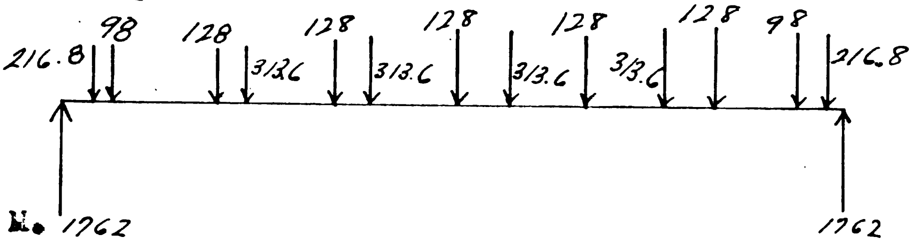
(35 + 1.5 x ~~4.8~~ 2.8) 8 = 313.6

Outside stringers 12.26 x 8 = 98

(25 + 1.5 x 1.4) 8 = 216.8

Dead Shear

$$1,762 + \frac{631.33}{2} = 2,102.69 \ \#$$



Dead B. M.

$$1762 \times 7.5 - 216.8 \times 7 - 98 \times 6.75 - 128 \times 4.7$$

$$- 313.6 \times 4.2 - 128 \times 2.35 - 313.6 \times 1.4 + \frac{631.33}{15} \times \left(\frac{15}{8}\right)^2$$

$$= 9,643 \text{ ft. lbs.}$$

Maximum live shear - same as # 2.

17,327 lbs.

Impact Shear - same as # 2.

5,520 lbs.

Total Shear

$$17,327 + 5,520 + 2102.69 = 24,949.7 \ \#$$

Stress

$$\frac{24,949.7}{12 \times 5/16} = 6,550 \ \#/\text{sq. in.}$$

Allowable 12,000 #/sq. in.

Maximum live B. M. - same as # 2.

55,650 ft. lbs.

Impact B. M.

17,750 ft. lbs.

Total B. M.

55,650 + 17,750 + 9,643 = 83,043 ft. lbs.

Tension Flange

$$A = \frac{83,043 \times 12}{16,000 \times 22.68} = 2.75 \text{ sq. in.}$$

2.75 - .94 = 1.81 sq. in. - area needed

Area supplied = 2 x 2.11 - 2 x 1 x 3/8 = 3.47 sq. in.

Compression Flange

Allowable stress - same as # 2

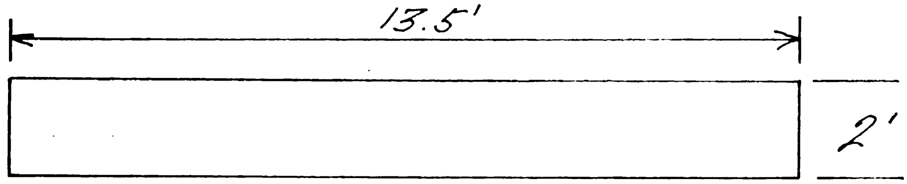
12,300 #/sq. in.

$$A = \frac{83,043 \times 12}{12,300 \times 22.68} = 3.57 \text{ sq. in.}$$

Area needed = 3.57 - .94 = 2.63 sq. in.

Area supplied = 2 x 2.11 = 4.22 sq. in.

Floor Beam # 1.



Wt.
27x7.2 = 194
13.5x30.6 = 413
4x3.19 = 12.8
619.8

Dead load from stringers same as # 3, 4, 5, 6, 7.

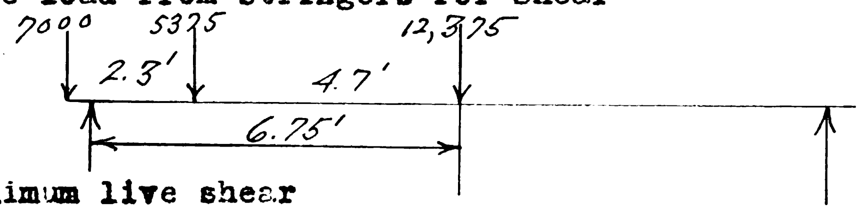
Dead Shear $836 + \frac{619.8}{2} = 1145.9$ #

Dead B. M.

$$836 \times 6.75 - 196 \times 7 - 256(4.7 + 2.35) + \frac{619.8}{13.5} \times \frac{(\cancel{15})\ \overline{13.5}^{2}}{8}$$

$= 3525$ ft. lbs.

Live load from stringers for Shear



Maximum live shear

$$\frac{12,375}{2} + \frac{5375 \times 11.45}{13.5} + \frac{7000 \times 13.75}{13.5} - 7000 = 10,837 \ \#$$

Impact Shear

$$10,837 \times \frac{50}{157} = 3460 \ \#$$

Total Shear

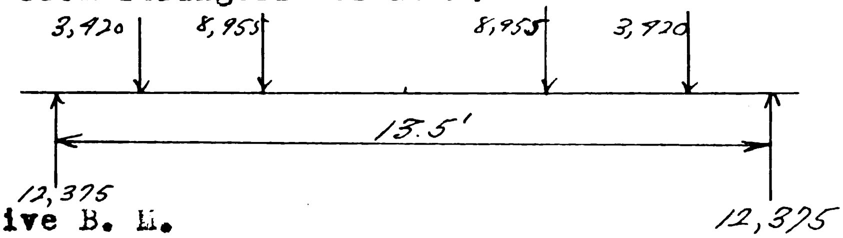
$$10,837 + 3460 + 1146 = 15,493 \ \#$$

Stress

$$\frac{15,493}{24 \times 5/16} = 2070 \ \#/\text{sq. in.}$$

Allowable 12,000 #/sq. in.

Love load from stringers for B. M.



Maximum live B. M.

12,375 x 6.75 - 8,955 x 2.35 - 3,420 x 4.7 = 46,400 ft. #

Impact B. M.

$46{,}400 \times \frac{50}{157} = 14{,}800$ ft. lbs.

Total B. M.

46,400 + 14,800 + 3525 = 64,725 ft. lbs.

Tension Flange

$A = \frac{64{,}725 \times 12}{16{,}000 \times 22.68} = 2.14$ sq. in.

Area needed = 2.14 - .94 = 1.20 sq. in.

Area supplied = 2 x 2.11 - 2 x 1 x 3/8 = 3.47 sq. in.

Compression Flange

Allowable stress $16{,}000 - 150 \frac{l}{b}$

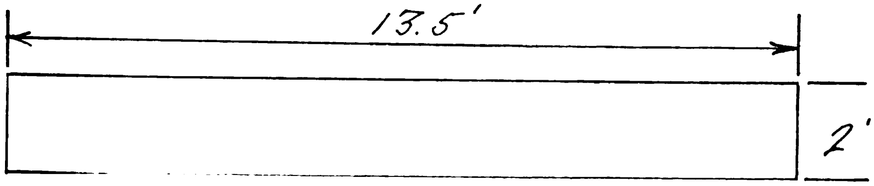
$= 16{,}000 - 150 \frac{13.5 \times 12}{7.31} = 12{,}680$ #/sq. in.

$A = \frac{64{,}725 \times 12}{12{,}630 \times 22.68} = 2.70$ sq. in.

Area needed = 2.70 - .94 = 1.76 sq. in.

Area supplied = 2 x 2.11 = 4.22 sq. in.

Floor Beam # 9.



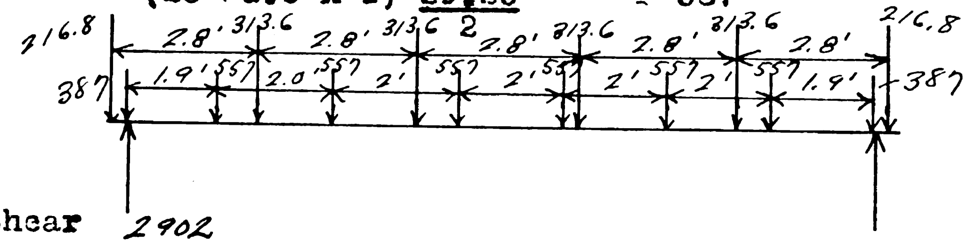
Dead load from stringers

Interior stringers $39.2 \times 8 = 313.6$

$(35 + 1.5 \times 2)\ \dfrac{29.25}{2} = 557$

Outside stringers $27.1 \times 8 = 216.8$

$(25 + 1.5 \times 1)\ \dfrac{29.25}{2} = 387$



Dead Shear

$2902 - 216.8 - 387 + \dfrac{619.8}{2} = 2608$ #

Dead B. M.

$2902 \times 6.75 - 216.8 \times 7 - 387 \times 6.9 - 557(5+3+1)$

$- 313.6(4.2 + 1.4) + \dfrac{619.8}{13.5} \times \left(\dfrac{13.5}{8}\right) = 9890$ Ft. lbs.

Live Shear- same as # 1.

10,887 #

Impact Shear - same as # 1

3460 #

Total Shear

$10,887 + 3460 + 2608 = 16,953$ #

Stress

$\dfrac{16,953}{24 \times 5/16} = 2260$ #/sq. in.

Allowable 12,000 #/sq. in.

Maximum live B. M. - same as # 1

46,400 ft. lbs.

Impact B. M. - same as # 1.

14,800 ft. lbs.

Total B. M.

46,400 + 14,800 + 9890 = 71,090 ft. lbs.

Tension Flange

$$A = \frac{71,090 \times 12}{16,000 \times 22.63} = 2.35 \text{ sq. in.}$$

Area needed 2.35 - .94 = 1.41 sq. in.

Area supplied 2 x 2.11 - 2 x 1 x 3/8 = 3.47 sq. in.

Compression Flange

Allowable stress - same as # 1.

12,680 #/sq. in.

$$A = \frac{71,090 \times 12}{12,680 \times 22.63} = 2.96 \text{ sq. in.}$$

Area needed 2.96 - .94 = 2.02 sq. in.

Area supplied 2 x 2.11 = 4.22 sq. in.

Trusses

Member	Dead Stress	Live Stress	Impact Stress	Total
L_0U_1	− 20,200	− 28,100	− 5,550	−53,850
L_0L_1	+ 11,800	+ 17,600	+ 3,480	+32,880
U_1L_1	+ 3,000	+ 13,590	+ 4,320	+20,910
U_1U_2	− 21,600	− 26,480	− 5,240	−53,320
U_1L_2	+ 14,400	− 1,930 + 30,500	− 673 + 6,770	+ 51,670
L_1L_2	+ 11,800	+ 17,600	+ 3,480	+32,880
U_2L_2	− 8,250	+ 1,510 −23,800	+ 527 − 5,290	− 37,340
U_2L_3	+ 8,650	− 4,530 +16,600	−1,400 +3,830	+29,080
U_2U_3	− 27,000	− 33,200	− 6,560	− 66,760
L_2L_3	+ 21,600	+ 26,480	+ 5,240	+ 53,320
U_3U_4	− 28,800	− 35,400	− 7,000	− 71,200
U_3L_4	+ 2,300	+ 11,900	+ 3,000	+17,700
L_3U_4	-----	+ 7,850	+ 2,160	+ 10,010
L_3L_4	+ 27,000	+ 33,200	+ 6,560	+ 66,760
U_3L_3	− 3,750	− 9,300 + 6,140	− 2,350 + 1,710	−15,400 + 4,100
U_4L_4	− 1,500	+18,600 −12,280	+ 4,700 − 3,420	+21,800 −17,200

TRUSSES

Loads

The dead load will be determined by the use of the formula $w = \frac{w_{,}}{9} + l$

$w_{,}$ - load brought to each truss per foot

l - span in feet

w - weight of each truss per foot in pounds

$$w_{,} = \left(\frac{19,500}{2} + 240 \times 8 \times 16\right)\left(1 + \frac{50}{128 \quad 125}\right) + 128 \times 7 \times \frac{1}{3} \times 4.5$$

$$+ 12.5 \times 128 \times 3.5 + \frac{620}{2} \times 9 = 58,233$$

$$w = \frac{58,233}{128 \times 9} + 128$$

$$w = 182 \text{ lbs.}/\text{ft.}$$

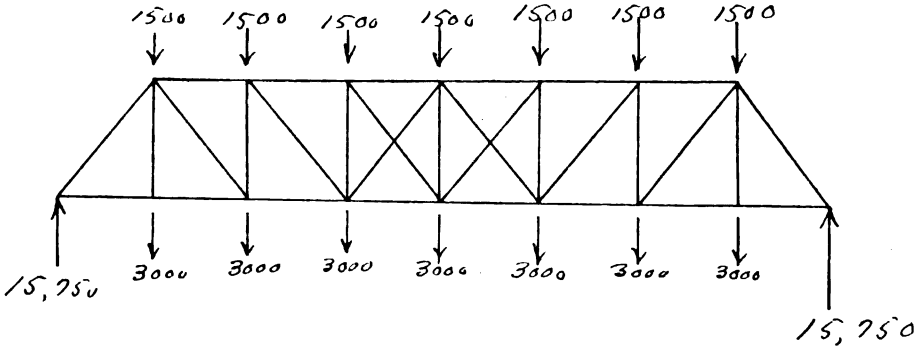
The live load used shall be the equivalent H-15 loading.

Dead panel weight due to truss equals		182 x 16 =	2910 lbs.
Dead panel loads	Upper chord		1450 "
	Lower chord	1220 + 1450 =	2670 "

Assume upper 1500 lbs. and lower 3000 lbs.

TRUSSES

Dead Stresses



L_0U_1

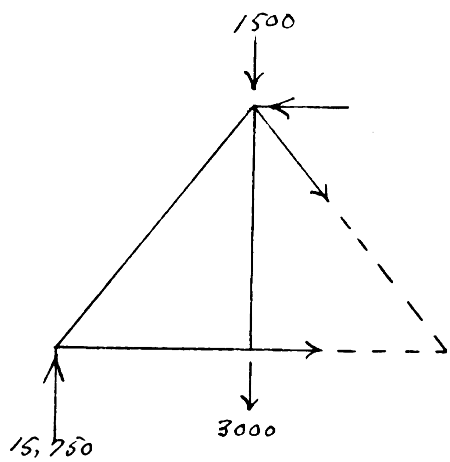
$$15{,}750 \times \frac{\sqrt{(16)^2+(20)^2}}{20} = 20{,}200 \ \#$$

L_0L_1

$$15{,}750 \times \frac{16}{20} = 11{,}800 \#$$

U_1L_1

3000 #



U_1U_2

$$15{,}750 \times 2 - 4500 \times 1 = S \times \frac{5}{4} \qquad S = 21{,}600 \ \#$$

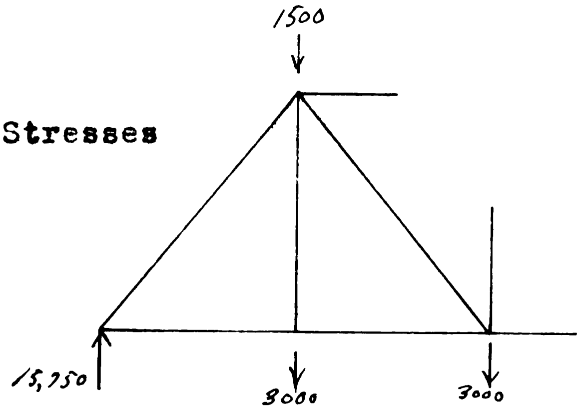
U_1L_2

$$(15{,}750 - 4500)\ \frac{25.6}{20} = 14{,}400 \ \#$$

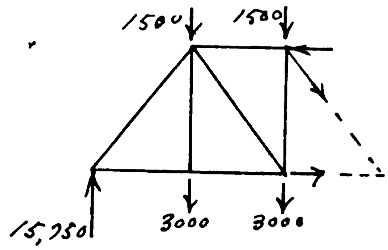
L_1L_2

11,800 #

Dead Stresses



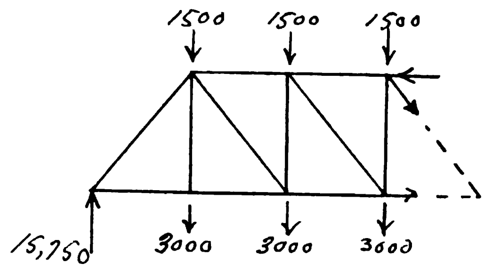
U_2L_2 15,750 - 7,500 = 8,250#



U_2L_3 $(15,750 - 9000)\ \dfrac{25.6}{20} = 8,650$ #

U_2U_3 $15,750 \times 3 - 4500 \times 2 - 4500 \times 1 = S \times \dfrac{5}{4}$ S = 27,000 #

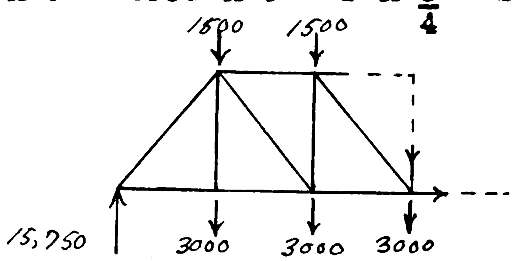
L_2L_3 $15,750 \times 2 - 4500 \times 1 = S \times \dfrac{5}{4}$ S = 21,600 #



U_3U_4 $15,750 \times 4 - 4500 \times 6 = S \times \dfrac{5}{4}$ S = 28,800 #

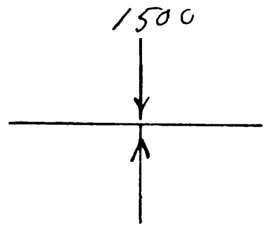
U_3L_4 $(15,750 - 9000 - 4500)\ \dfrac{25.6}{20} = 2,800$ #

L_3L_4 $15,750 \times 3 - 4500 \times 3 = S \times \dfrac{5}{4}$ S = 27,000 #



U_3U_3 15,750 - 9000 - 3000 = 3,750 #

Dead Stresses

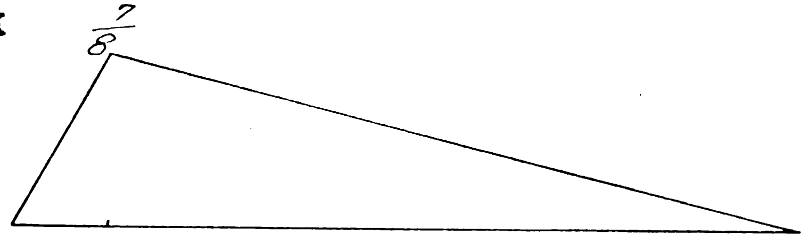


$U_1 L_1$

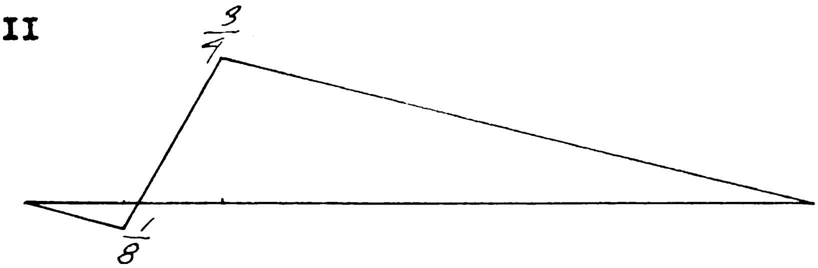
1500 #

INFLUENCE LINES

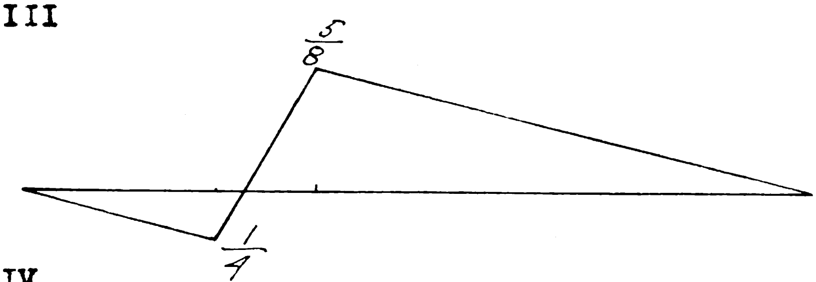
Shear Panel I



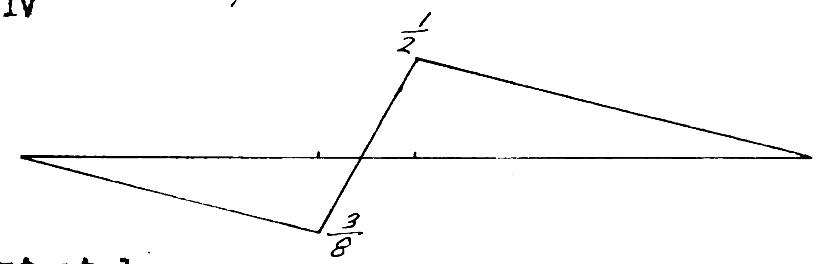
Shear Panel II



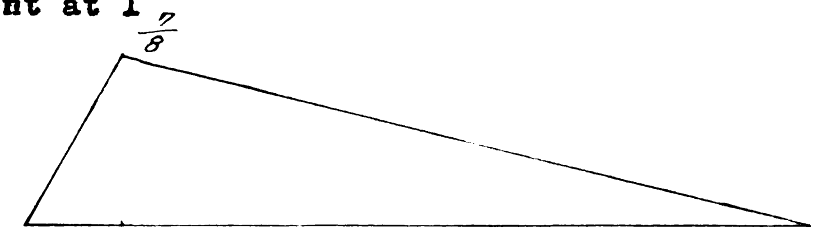
Shear Panel III



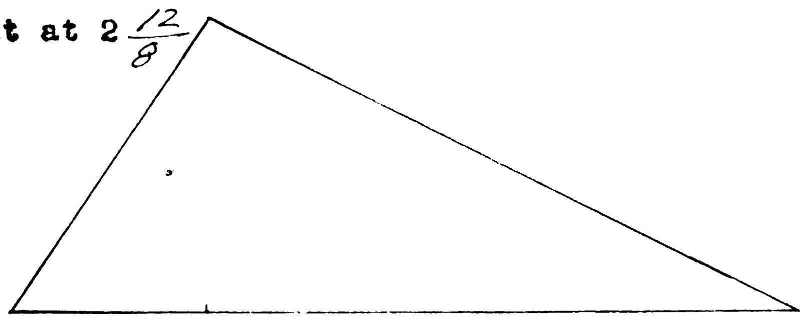
Shear Panel IV



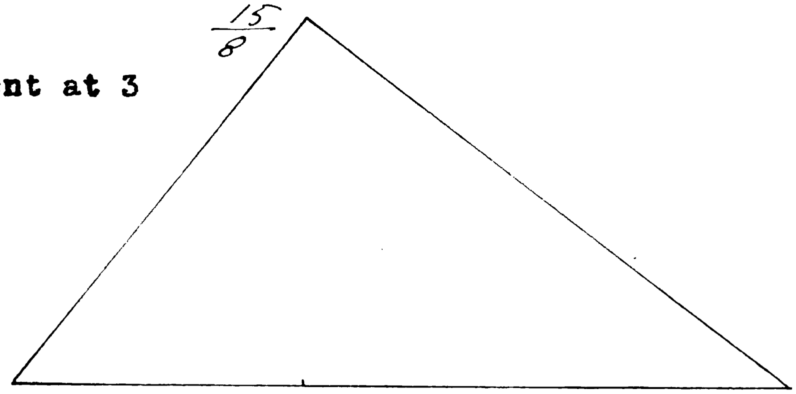
Bending Moment at 1



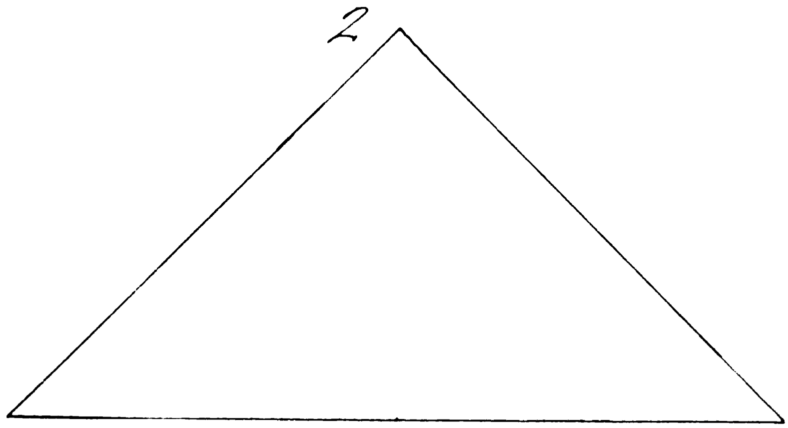
Bending Moment at 2



Bending Moment at 3



Bending Moment at 4



Live Stresses

L_0U_1 from shear panel I

$$\left(\frac{19,500}{2} \times \frac{7}{8} + \frac{7}{8} \times \frac{128}{2} \times 240\right) \frac{25.6}{20} = -28,100$$

L_0L_1

$$\left(\frac{19,500}{2} \times \frac{7}{8} + \frac{7}{8} \times \frac{128}{2} \times 240\right) \frac{16}{20} = +17,600$$

U_1L_1

$$\frac{19,500}{2} + 16 \times 240 = +13,590$$

U_1U_2 from B. M. at 2

$$\left(\frac{13,500}{2} \times \frac{12}{8} + \frac{12}{8} \times \frac{128}{2} \times 240\right) \frac{4}{5} = -26,480$$

U_1L_2 from shear panel II

$$\left(\frac{19,500}{2} \times \frac{1}{8} + \frac{1}{8} \times \left(\frac{16}{2} + \frac{16}{2x7}\right) \times 240\right) \frac{25.6}{20} = -1,930$$

$$\left(\frac{19,500}{2} \times \frac{3}{4} + \frac{3}{4} \times \left(\frac{96 + 16 \times \frac{6}{7}}{2}\right) 240\right) \frac{25.6}{20} = +30,500$$

L_1L_2

$L_0L_1 \quad L_1L_2 \qquad = + 17,600$

U_2L_2

$$1,930 \times \frac{20}{25.6} = + 1,510$$

$$30,500 \times \frac{20}{25.6} = - 23,800$$

U_2L_3 from shear panel III

$$\left(\frac{19,500}{2} \times \frac{1}{4} + \frac{1}{4} \left(\frac{32 + 16 \times \frac{2}{7}}{2}\right) 240\right) \frac{25.6}{20} = -4,530$$

$$\left(\frac{19,500}{2} \times \frac{5}{8} + \frac{5}{8} \left(\frac{80 + 16 \times \frac{5}{7}}{2}\right) 240\right) \frac{25.6}{20} = +16,600$$

U_2U_3 from B. M. at 3

$$\left(\frac{13,500}{2} \times \frac{15}{8} + \frac{15}{8} \times \frac{128}{2} \times 240\right) \frac{4}{5} = -33,200$$

L_2L_3 from B. M. at 2

Same as U_1U_2 $= +26,480$

U_3U_4 from B. M. at 4

$$\left(\frac{13,500}{2} \times 2 + 2 \times \frac{128}{2} \times 240\right) \frac{4}{5} = -35,400$$

U_3L_4 from Shear panel IV

$$\left(\frac{19,500}{2} \times \frac{1}{2} + \frac{1}{2} \times \frac{64 + \frac{4}{7} \times 16}{2} \times 240\right) \frac{25.6}{20} = +11,900$$

U_4L_3 from Shear panel IV

$$\left(\frac{19500}{2} \times \frac{3}{8} + \frac{3}{8} \times \frac{48 + \frac{3}{7} \times 16}{2} \times 240\right) \frac{25.6}{20} = +7,850$$

L_3L_4 from B. M. at 3

Same as U_2U_3 $= +33,200$

U_3L_3 Method of joints, upper

$$11,900 \times \frac{20}{25.6} = -9,300$$

$$7,850 \times \frac{20}{25.6} = +6,140$$

U_4L_4 Method of joints, upper

$9,300 \times 2$ $= +18,600$

$6,140 \times 2$ $= -12,280$

Impact Stresses

L_0U_1

$$23{,}100 \times \frac{50}{123 + 125} = -5{,}550$$

L_0L_1

$$17{,}600 \times \frac{50}{128 + 125} = +3{,}430$$

U_1L_1

$$13{,}590 \times \frac{50}{\cancel{123}\,32 + 125} = +4{,}320$$

U_1U_2

$$26{,}430 \times \frac{50}{123 + 125} = -5{,}240$$

U_1L_2

$$1{,}930 \times \frac{50}{13.3 + 125} = -673$$

$$30{,}500 \times \frac{50}{110 + 125} = +6{,}770$$

L_1L_2

$$17{,}600 \times \frac{50}{253} = +3{,}430$$

U_2L_2

$$1{,}510 \times \frac{50}{143.3} = +527$$

$$23{,}300 \times \frac{50}{110 + 125} = -5{,}290$$

U_2L_3

$$4{,}530 \times \frac{50}{36.6 + 125} = -1{,}400$$

$$16{,}600 \times \frac{50}{91.5 + 125} = +3{,}830$$

U_2U_3

$$33{,}200 \times \frac{50}{253} = -6{,}560$$

L_2L_3

$$26{,}430 \times \frac{50}{253} = +5{,}240$$

U_3U_4

$$35{,}400 \times \frac{50}{253} = -7000$$

U_3L_4

$$11{,}900 \times \frac{50}{73.1 + 125} = +3{,}000$$

U_4L_3

$$\overset{7{,}850}{\cancel{33{,}200}} \times \frac{50}{54.9 + 125} = +2{,}160$$

L_3L_4

$$33{,}200 \times \frac{50}{253} = +6{,}560$$

U_3L_3

$$9{,}300 \times \frac{50}{73.1 + 125} = -2{,}350$$

$$6{,}140 \times \frac{50}{54.9 + 125} = +1{,}710$$

U_4L_4

$$13{,}600 \times \frac{50}{73.1 + 125} = +4{,}700$$

$$12{,}230 \times \frac{50}{54.9 + 125} = -3{,}420$$

L_0U_1

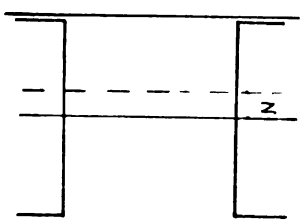
A compression member composed of two channels 8" x $2\frac{1}{4}$" - $\frac{1}{4}$" , seven inches back to back and a cover plate 12" x $\frac{5"}{16}$ riveted together every five inches.

Load 53,850 # compression

Area

Channels	2 x 3.36 =	6.72 sq. in.
cover plate		3.75
		10.47 sq. in.

Stress $\frac{53,850}{10.47} \approx$ 5,150 #/sq. in.



$$z = \frac{\frac{60}{16}\left(4 + \frac{5}{32}\right)}{10.47} = 1.49''$$

I about horizontal axis

C. P. $\frac{1}{12} \times 12 \times \left(\frac{5}{16}\right)^3 + \frac{60}{16}(2.66)^2 = 26.60$

Channels $2\left(37.3 + 3.36(2.66)^2\right) = 113.6$

140.2

$$K = \sqrt{\frac{I}{A}} = \sqrt{\frac{140.2}{10.47}} = 3.66$$

I about vertical axis

C. P. $\frac{1}{12} \times \frac{5}{16} \times (12)^3 = 45.00$

channels 2×1.3 $\quad 2.6$

47.6

$$K = \sqrt{\frac{I}{A}} = \sqrt{\frac{47.6}{10.47}} = 2.13$$

Allowable stress = 15,000 - 50$\left(\frac{25.6 \times 12}{2.13}\right)$ = 7,800 #/sq. in.

This member is O. K. and will stand a load of

$7,800 \times 10.47 = 81,600$ #

L_0L_1

A tension member composed of two eye-bars 2" x $\frac{3}{4}$"

Load 32,830 # tension

Stress $\frac{32,830}{2 \times 2 \times \frac{3}{4}} = 10,960$ #/sq. in.

Allowable 16,000 #/sq. in.

This member is O. K. and will stand a load of

$16,000 \times 3 = 48,000$ #

U_1L_1

A tension member composed of two one inch square bars.

Load 20,910 # tension

Stress $\frac{20,910}{2} = 10,455$ #/sq. in.

Allowable 16,000 #/sq. in.

This member is O. K. and will stand a load of

$16,000 \times 2 = 32,000$ #

U_1U_2

A compression member the same as L_0U_1 except that it is of different length.

Load 53,320 # comp.

Stress $\frac{53,320}{10.47} = 5100$ #/sq. in.

Allowable stress $15,000 - 50\left(\frac{16 \times 12}{2.13}\right) = 11,500$ #/sq. in.

This member is O. K. and will stand a load of

$11,500 \times 10.47 = 120,300$ #

U_1L_2

A tension member composed of two, one inch by one and three quarter inches, bars.

Load 51,670 # tension

Stress $\frac{51,670}{2x1x1\frac{3}{4}}$ = 14,900 #/sq. in.

Allowable 16,000 #/sq. in.

This member is O. K. and will stand a load of

16,000 x 3.5 = 56,000 #

L_1L_2

A tension member composed of two eye-bars 2" x $\frac{3}{4}$".

Load 32,830 # tension

Stress $\frac{32,830}{2x2x\frac{3}{4}}$ = 10,960 #/sq. in.

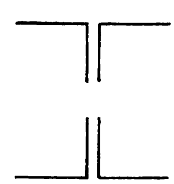
Allowable 16,000 #/sq. in.

This member is O. K. and will stand a load of

16,000 x 3 = 48,000 #

U_2L_2 A compression member composed of four 3" x 2½" x $\frac{5}{16}$", six and one half inches back to back - the three inch leg outstanding - and fastened together with lacing bars.

Load 37,340 # compression



Stress $\frac{37,340}{4 \times 1.62}$ = 5,760 #/sq. in.

I about horizontal axis

$$4(.9 + 1.62(3.25 - .68)^2) = 46.4$$

$$K = \sqrt{\frac{I}{A}} = \sqrt{\frac{46.4}{6.48}} = 2.67$$

I about vertical axis

$$4\left(1.4 + 1.62\left(.93 + \frac{5}{32}\right)^2\right) = 13.29$$

$$K = \sqrt{\frac{13.29}{6.48}} = 1.43$$

Allowable stress $15{,}000 - 50\left(\frac{20 \times 12}{1.43}\right) = 6{,}600$ #/sq. in.

This member is O. K. and will stand a stress or load of $6{,}600 \times 6.48 = 42{,}800$ #

U_2L_3

A tension member composed of two bars, one and three quarters inches by three quarters inches.

Load 29,080 # tension

Stress $= \frac{29{,}080}{2 \times 1\frac{3}{4} \times \frac{3}{4}} = 11{,}100$ #/sq. in.

Allowable 16,000 #/sq. in.

This member is O. K. and will stand a load of

$16{,}000 \times 2.62 = 41{,}900$ #

U_2U_3

A compression member the same as U_1U_2.

Load 66,760 # compression

Stress $= \frac{66{,}760}{10.47} = 6{,}390$ #/sq. in.

Allowable stress 11,500 #/sq. in.

This member is O. K. and will stand a load of

$11{,}500 \times 10.47 = 120{,}500$ #

L_2L_3

A tension member composed of two eye-bars, three inches by three quarters inches.

Load 53,320 # tension

Stress $= \frac{53{,}320}{2 \times 3 \times \frac{3}{4}} = 11{,}900$ #/sq. in.

Allowable 16,000 #/sq. in.

This member will stand a load of $16{,}000 \times 4.5 = 72{,}000$ #

U_3U_4

A compression member the same as U_1U_2.

Load 71,200 # compression

Stress $\frac{71,200}{10.47}$ = 6,800 #/sq. in.

Allowable 11,500 #/sq. in.

This member is O. K. and will stand a load of

11,500 x 10.47 = 120,300 #

L_3U_4

This member is a counter member designed to take the reversal of stress in U_3L_4 and acts as a tension member. It is a seven eighths inch round bar.

Load 10,010 # tension

Stress $\frac{10,010}{.6013}$ = 16,700 #/sq. in.

Allowable 16,000 #/sq. in.

This member is slightly overstressed but will not fail due to the factor of safety.

L_3L_4

A tension member composed of two, three and one half inch by one inch eye-bars.

Load 66,760 # tension

Stress $\frac{66,760}{2 \times 3\frac{1}{2} \times 1}$ = 9,550 #/sq. in.

Allowable 16,000 #/sq. in.

This member is O. K. and will stand a load of

16,000 x 7 = 112,000 #

U_3L_4

A tension member composed of two one and one half inch by one half inch bars.

Load 17,700 # tension

Stress $\frac{17,700}{2 \times 1\frac{1}{2} \times \frac{1}{2}} = 11,800$ #/sq. in.

Allowable 16,000 #/sq. in.

This member is O. K. and will stand a stress of 16,000 x 1.5 = 24,000 #

U_3L_3

This member has a reversal of stress but was designed as a compression member. It is composed the same as U_2L_2

Load 15,400 # compression 4,100 # tension

Stress $\frac{15,400}{6.48} = 2,380$ #/sq. in. Allowable 6,600 #/sq. in.

$\frac{4,100}{6.48-1.2} = 777$ #/sq. in. Allowable 16,000 #/sq. in.

This member is O. K. and will stand a load of 6,600 x 6.48 = 42,800 # compression and 16,000 x 5.28 = 84,500 # tension.

U_4L_4

This member has a reversal of stress. It is composed the same as U_2L_2.

Load 21,800 # tension 17,200 # compression

Stress $\frac{21,800}{6.48-1.2} = 4,130$ #/sq. in. Allowable 16,000 #/sq. in.

$\frac{17,200}{6.48} = 2,660$ #/sq. in. Allowable 6,600 #/sq. in.

TOP LATERALS

These are designed to resist a moving load of 150 lbs./ft.

Diagonals

These are tension members and are 3/4" round bars. The maximum stressed will be those in the first panel and will be due to shear.

Load per panel $16 \times 150 = 2400$ #

Shear $2.5 \times 2400 = 6000$ #

$$6000 \times \sqrt{\frac{(15)^2 + (16)^2}{15}} = 8300 \text{ \#}$$

Stress $\frac{8300}{.4413} = 19,900$ #/sq. in.

Allowable 16,000 #/sq. in.

The diagonals are slightly over-stressed but they will not fail due to the factor of safety.

Lateral Strut

Takes no computed stress. Composed of four angles, 2½" x 2" - 5/16", 2½" leg outstanding. Eight inches back to back. Connected by lacing bars riveted every foot.

For unsupported length of 15' the radius of gyration should equal $\frac{1}{140} = \frac{15 \times 12}{140} = 1.29"$

I about horizontal axis $- 4(.45 + 1.31(3.44)^2) = 63.8$

$$r = \sqrt{\frac{63.8}{5.24}} = 3.5" \quad O. K.$$

BOTTOM LATERALS

These are designed for a moving load of 300 #/ft. The diagonals are one inch square bars. The maximum stressed will be those in the first panel and will be due to shear.

Load per panel $= 16 \times 300 = 4800$ #

Shear $3.5 \times 4800 = 16,800$ #

$$16,800 \times \sqrt{\frac{(15)^2 + (16)^2}{15}} = 24,600 \text{ \#}$$

Stress $\frac{24,600}{1} = 24,600$ #/sq. in.

Allowable 16,000 #/sq. in.

These are over-stressed quite a little but they will not fail.

LACING BARS

For L_0U_1, U_1U_2, U_2U_3, U_3U_4.

Bars used are 2" x 5/16".

$$R = \frac{Pl}{4000 \times y} = \frac{11,500 \times 10.47 \times 16 \times 12}{4000 \times 5.49} = 1050 \#$$

$$\text{Stress } \frac{1050}{2 \times 5/16} = 1680 \#/\text{sq. in.}$$

Allowable 12,000 #/sq. in.

For U_2L_2, U_3L_3, U_4L_4.

Bars used are $1\frac{3}{4}$" x 5/16".

$$R = \frac{Pl}{4000y} = \frac{6,600 \times 6.43 \times 20 \times 12}{4000 \times 3.25} = 790 \#$$

$$\text{Stress } \frac{790}{1\frac{3}{4} \times 5/16} = 1450 \#/\text{sq. in.}$$

Allowable 12,000 #/sq. in.

For upper cross members.

Bars used are 1" x 5/16".

$$S = \frac{P}{A} = 15,000 - 50\left(\frac{15 \times 12}{1.24}\right) = 7,740 \#/\text{sq. in.}$$

$$R = \frac{Pl}{4000y} = \frac{7,740 \times 5.24 \times 15 \times 12}{4000 \times 3.5} = 520 \#$$

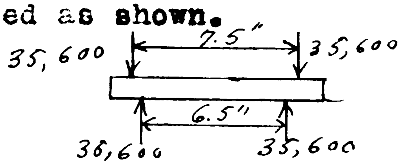
$$\text{Stress } \frac{520}{1 \times 5/16} = 1660 \#/\text{sq. in.}$$

Allowable 12,000 #/sq. in.

PINS

The pins used are all of the same size, $1\frac{3}{4}$" in diameter. The maximum allowable Bending Moment is 12,600 in.-lbs.

The pin which is the worst stressed has a load of 71,200 # placed as shown.



Bending Moment = 35,600 x 3.75 - 35,600 x 3.25

= 17,800 in. lbs.

This particular pin is over-stressed but most of them are not because they are not loaded so heavy.

PORTAL BRACING

Portal bracing is used to strengthen the bridge and also for appearance. The stresses encountered are too small to be calculated. The portal bracing used is 3" x $2\frac{1}{2}$" - 5/16" angles.

CONCLUSIONS

1- The stringers are not strong enough for the H - 15 loading.

2- The Truss members are O. K.

3- Both the top and bottom laterals are over-stressed.

4- The Lacing bars are O. K.

5- A few of the pins are over-stressed.

6- This type of structure gives one the false impression of great strength.

ROOM USE ONLY