JUDD STREET TIMBER BRIDGE, EAST LANSING, MICHIGAN

Thesis for the Degree of B. S.
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Judd Street Timber Bridge,

East Lansing, Michigan

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by

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THESIS

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INTRODUCTION

This thesis covers the complete drawings and design for a timber bridge to connect Judd Street and East Kalamazoo Street located in East Lansing, Michigan.

There is no actual need for a bridge at this place since there is a bridge across the Red Cedar River two blocks east on Harrison Road, however for the purpose of this thesis it will be assumed that a bridge is needed at this point and is to be built in the near future.

Because of the war priority on steel a timber bridge was chosen. A steel bridge would be out of the question, and it would be difficult to obtain the necessary reinforcing for a concrete structure. The use of timber to-day is becoming wide spread and is being recognized as one of the foremost substitutes for steel in roof trusses, bridge trusses and in many other structures. The use of timber connectors has greatly increased the strength of timber joints, and the methods of treating timber have reached the point where if painted and cared for properly, the timber structure will last almost as long as steel.

The advantages of timber for use in bridge construction are many. Below are listed a few.

1. Low first cost.
2. Maintenance cost is low.
3. Simple erection saves time and costs.
4. Less material and hardware is required.
5. Re-use and salvage value is high.
6. Maximum amount of local labor and material is used.

The rapid development of our highway system, resulting in constant changes in location and capacity has caused the consideration of 40 to 50 years as a reasonable life for most bridges. Since past history shows that wood bridges have a life of from 50 to 100 years their economy is quite apparent over a 40 or 50 year period when the above mentioned advantages are given full consideration.

Since this bridge is to be built in Michigan the Michigan State Highway Department Specifications for Highway bridges were followed unless otherwise noted.

The preliminary survey included in this thesis was carried out more for the purpose of determining the span necessary and the position of the bridge relative to the river than for construction purposes.

I would like to express my appreciation to Mr. Charles A. Miller of the Michigan State College Civil Engineering Department, Mr. Gleason of the Michigan State Highway Department, Bridge Design Division and to Mr. E. S. Lank, structural engineer for the Timber Engineering Company for the advice and cooperation given to me during the preparation of this thesis.

COMPUTATIONS FOR THE DESIGN

OF THE 72' HOWE TRUSS.

DATA:

Live load.

H-15 (Michigan State Highway Department Specifications)

Impact

Most all of the latest theorys on timber design state that no account need be taken of impact since the resilience of the wood is supposed to absorb the shock. To be on the safe side the Michigan State Highway Department's impact formula for steel structures was used in this design with I = 30% as a maximum.

$$I = \frac{1 + 20}{61 + 20}$$

Dead load.

Timber is assumed as weighing 60 pounds per cu. ft. Steel as weighing 490 pounds per cu. ft.

Specifications.

Unless otherwise noted the Michigan State Highway specifications for highway bridges were used.

Temperature.

No account was taken for temperature differences as the coefficient of expansion for wood is very small.

Stresses.

The unit stresses used by the Michigan State Highway Department are those recommended by Forest Products Laboratory, U. S. Forest Service, for material complying with structural grades of the American Society for Testing Materials.

The unit stresses given are for three different conditions of exposure during use. The exposure in this case being occasionally wet.

	4" and thinner	3" and thicker
$f_s=$	1,370 #/sq. in.	1,515 #/sq. in.
$f_c=$	265 #/sq. in.	265 #/sq. in.
$H=$	105 #/sq. in.	105 #/sq. in.

Where $f_s=$ Allowable stress in extreme fiber due to bending and axial tension.

$f_c=$ Compression perpendicular to grain.

$H=$ Horizontal shear.

For compression parallel to grain see table below.

RATIO OF LENGTH TO LEAST DIMENSION							
1-10	12	14	16	18	20	25	30
1,165	1,139	1,118	1,083	1,036	971	702	487

Fig. 1

Modulus of elasticity = 1,600,000

These values are for Douglas Fir-dense select structural grade. All lumber shall be of a structural grade (S4S). All lumber shall be fabricated before treatment and then pressure treated with creosote by the empty cell process with a retention of not less than 8 # per cu. ft.

Truss.

Type - Howe

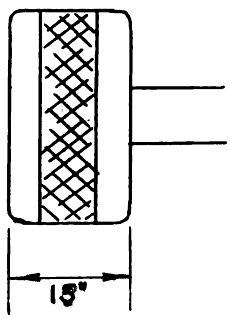
Span - 72' (Four panels)

Width - 23 ' Center to center of trusses.

Height - 15'.

DESIGN OF FLOORING:

15 Ton truck of H-15 loading has rear wheel as shown.



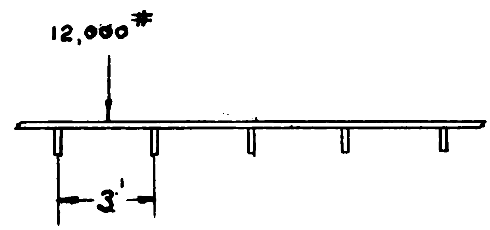


Fig. 2

Say use 2"x6"

Live moment - (occuring at center of outer two stringers)

$$M = 7/40 \times Pl = 7/40 \times 12,000 \times 3 = 6,300'\#$$

Impact.

$$\frac{3+20}{18+20} = 60.5\% \quad \text{so use } 30\%$$

$$.30 \times 6,300 = 1,890'\#$$

Dead load -

$$\left(\frac{15 \times 6}{144}\right) \times 60 = 37.5\#$$

$$M = 1/14\, wl^2 = 1/14 \times 37.5 \times 9 = 24.0'\#$$

Total -

$$6,300 + 24 + 1,890 = 8,214'\#$$

$$S = \frac{Mc}{I}$$

Now $M = 8,214 \times 12 = 98,500"\#$

$c = 3"$

$$I = \frac{bh^3}{12} = \frac{15 \times 216}{12} = 270$$

$$S = 1{,}515\#/\text{sq. in.}$$

$$S = \frac{98{,}500}{270} \times 3 = 1{,}100\#/\text{sq. in.}$$

Since the allowable is 1,515 #/sq. in. and the negative moment will be less, the 2"x6" flooring will be safe.

STRINGER DESIGN:

In accordance with the Michigan State Highway Department Specification, timber stringers shall be of sufficient length to take bearing over the full width of caps or floorbeams, except outside stringers which may have butt joings. Preferably they shall be of two panel lengths placed with staggered joints. The lapped ends of untreated stringers shall be separated at least ½" for air circulation. Stringers shall be adequately secured to caps or floorbeams. Stringers shall be adequately braced by cross bridging in each panel. The bridging shall not be less in size than 2" x 4". The stringers will be arrainged as shown below.

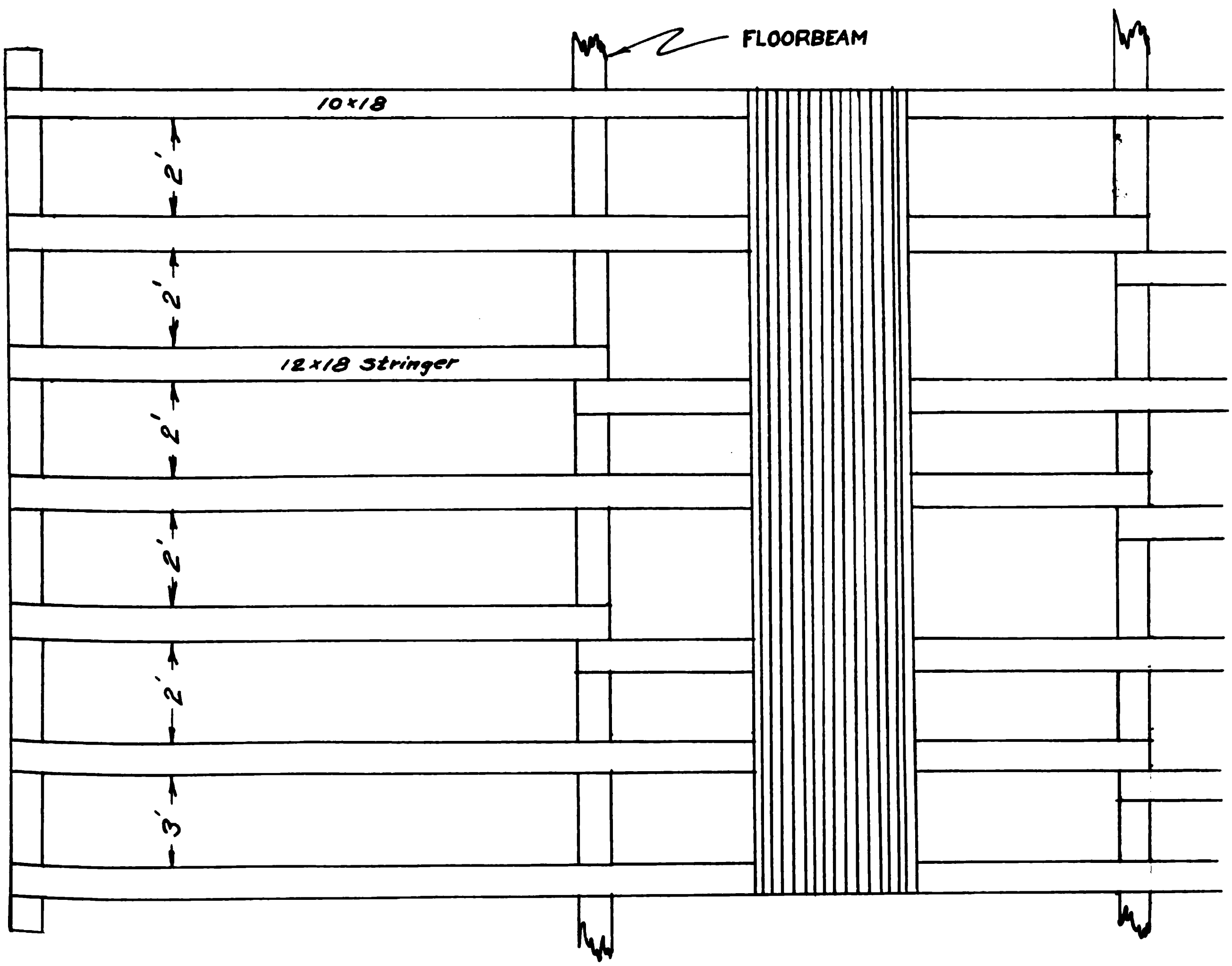


Fig. 3

Each stringer is nailed to the floorbeam and cap with 4-30d nails. 2"x6" bridging at panel points and mid panel points is nailed to stringers with 4-16d nails.

The maximum moment for this 18' stringer will occur when the rear truck wheels are at the center of the panel and the front wheels in the adjacent panel. The rear wheel load for H-15 loading is .4 of the total weight of the truck or 12,000#. The moment due to this load is pl/4 where Pis the load and l the length of the stringer. This moment is increased for impact and the dead moment of 1/8 wl^2 is added.

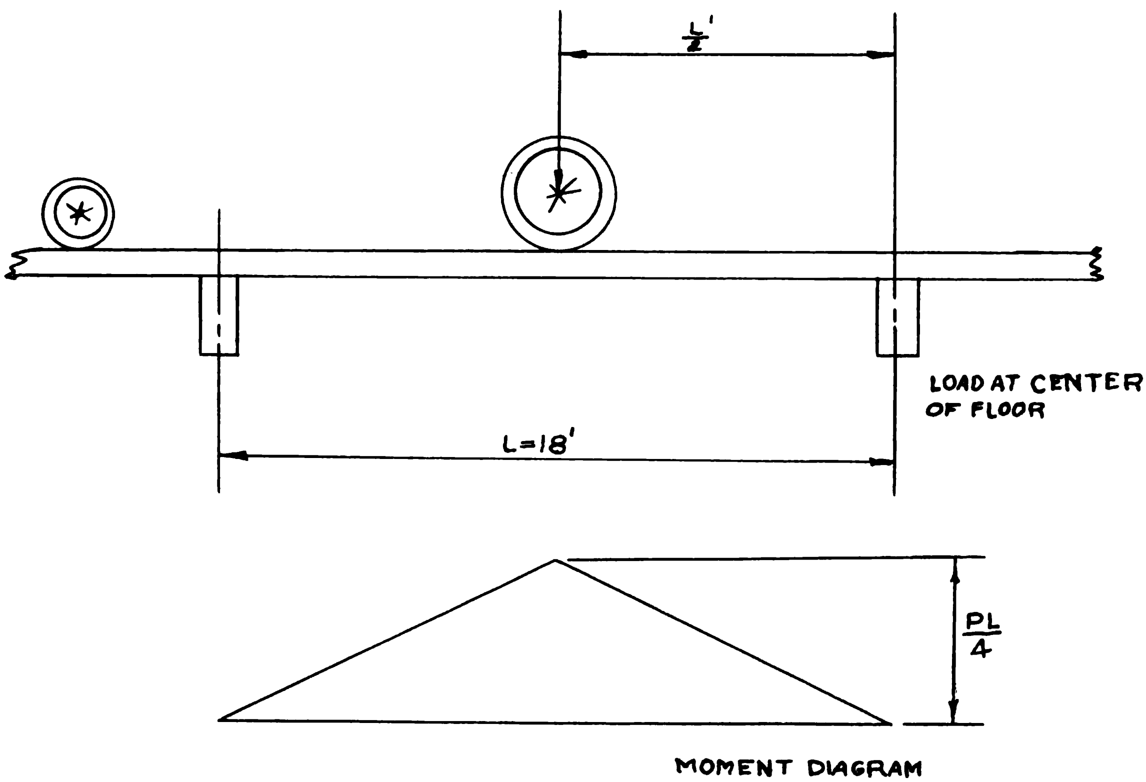


Fig. 4

Dead load

Assume the stringer as being 18" x 10". Then the dead load per foot per stringer will be --

$(19.75 \times \frac{5.5}{12} \times 60) \times \frac{1}{6} = 90.5\#/ft.$ (flooring)

$(\frac{10 \times 18}{144}) \times 60 = 75\#/ft.$ (stringer)

Total $=$ $165.5\ \#/ft.$

$M = 1/8\ wl^2 = 1/8 \times 165.5 \times (18)^2 = 6{,}700'\#$

Live load

The live load is equal to --

$Pl/4 = \frac{12{,}000 \times 18}{4} = 54{,}000'\#$

To which impact is added.

$I = \frac{18 + 20}{108 + 20} = 30\%$

$.30 \times 54{,}000'\# = 16{,}200'\#$

Total moment $= 54{,}000 + 6{,}700 + 16{,}200 = 77{,}900'\#$

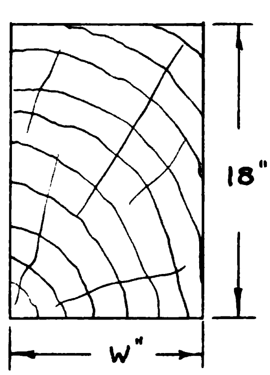


Fig. 5

$S = \frac{Mc}{I}$

$I = \frac{wh^3}{12} = 486w$

$C = 9$

$S = \frac{9.M}{486w}$

or

$w = \frac{9.M}{486.S}$

$W = \frac{9 \times 77{,}900}{486 \times 1{,}515} \times 12 = 11.4"$ So use a 12" x 18" stringer.

Check the above designed stringer for vertical shear.

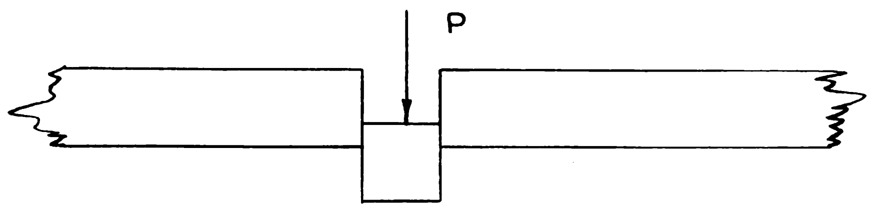


Fig. 6

Place the rear wheel just on the stringer as in fig. 7 .

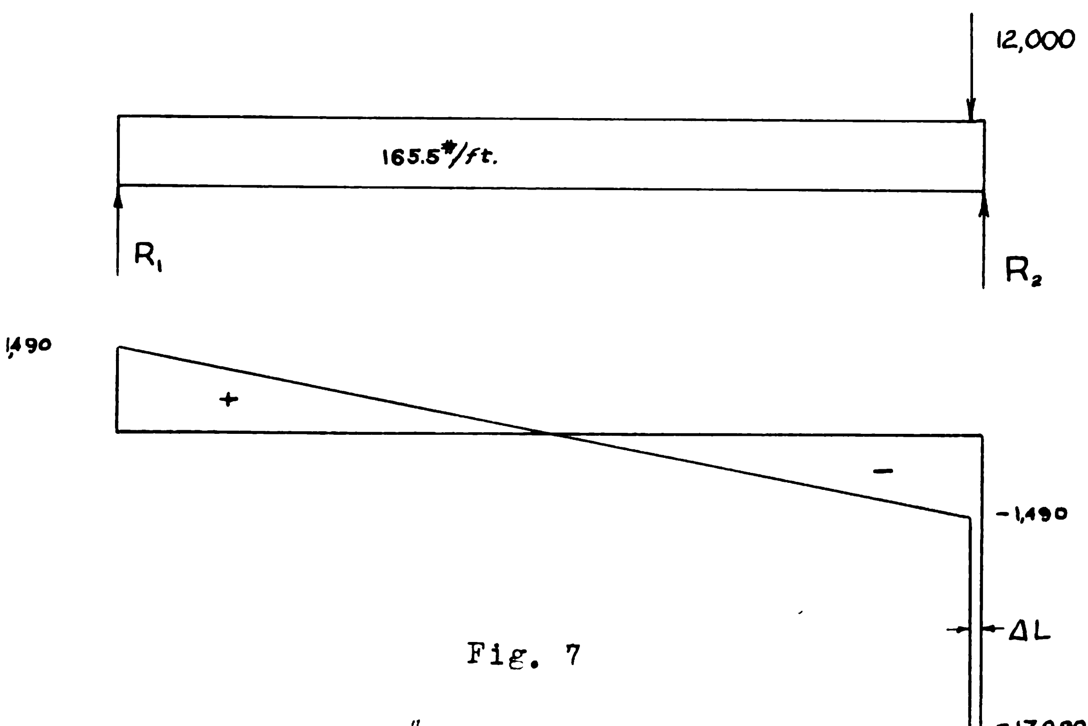


Fig. 7

$R_1 = 165.5 \times 9 = 1490^{\#}$

$R_2 = 1490 + 12,000 + .3 \times 12,000 = 17,090^{\#}$

Shearing stress $= \dfrac{17,090}{18 \times 12} = 79^{\#}$/sq. in.

O.K. Since less than allowable.

Check for horizontal shear.

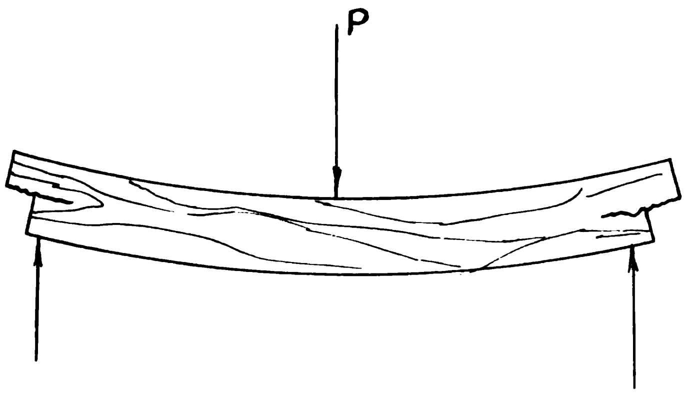


Fig. 8

The maximum horizontal shearing stress in a wood beam is found by the formula --

$q = \frac{VQ}{It}$ Where

$q =$ Maximum horizontal shear stress.

$V =$ External shear.

$I =$ Moment of inertia of section about neutral axis.

$t =$ Width of beam at neutral axis.

$Q =$ Statical Moment of section about neutral axis.

$$q = \frac{17{,}090 \times 440}{5840 \times 12} = 107.0\#/\text{sq. in.}$$

This is satisfactory even though 2 pounds above allowable, since the stringer is actually continuous for two spans of 18'.

Hence it is satisfactory to use a 12" x 18" stringer. The end stringers, that is the ones placed beneath the curbs, normally receive less load than the center stringers and could be made proportionally smaller. This is seldom done though because of the high impact stress they would receive if a truck were to strike the curb.

FLOORBEAM DESIGN:

In accordance with the Michigan State Highway Department Specifications, timber floorbeams shall be sized at bearing points. In floorbeams composed of two or more members, the timbers shall be separated by at least 2" for air circulation.

Because of the rather long panel lengths, the floorbeams in this bridge take a larger load than usual. In view of this fact I believe that a built up or trussed floorbeam will be lighter and more satisfactory in this case than a single wood beam. The floorbeam truss was designed so as to give adequate support to the stringers resting upon it. Impact was computed using the formula given under Data, using for l the length of the floorbeam with a maximum of 30%.

Dead load. Consists of flooring, stringers, guards, bridging, railing, railing posts, hardware, bracing and weight of floorbeam itself.

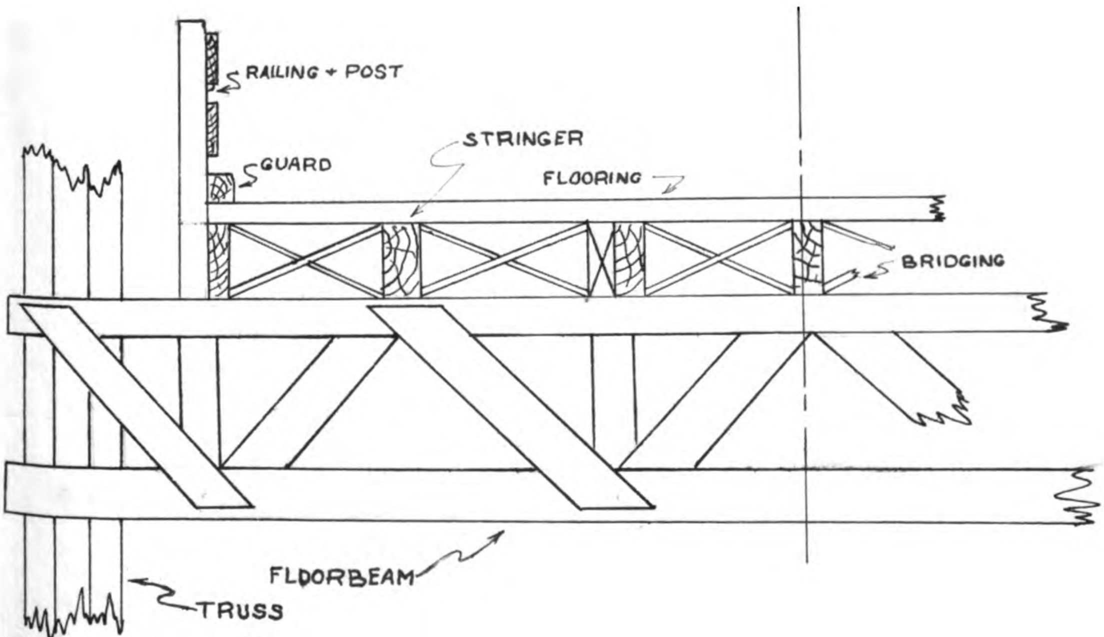


Fig. 9

The load is transmitted to the floorbeam by the stringers at points 1, 2, 3 and 4.

Flooring (2x6) - $(18' \times \frac{5.5}{12} \times 19.75')\ 60 = 9{,}770\#$

Stringer (12x18) - $(18' \times \frac{12 \times 18}{144})\ 60 = 1{,}620\#$

* Guards (10x10) - $(18' \times \frac{10 \times 10}{144})\ 60 = 750\#$

Bridging (2x6) - $(3.3 \times \frac{2 \times 6}{144}) \times 2 \times 6 \times 2 \times 60 = 400\#$

* Railing (3x8) - $(18 \times \frac{3 \times 8}{144}) \times 2 \times 60 = 360\#$

* Railing Posts (6 x 8) - $(5 \times \frac{6 \times 8}{144}) \times 2 \times 60 = 200\#$

Hardware - Taken into account in rounding out loads.

Bracing (3x8) - $(25.4 \times \frac{3 \times 8}{144})\ 60 = 254\#$

Weight of floorbeam itself -

Say $\frac{8 \times 16}{144} \times 3 \times 60 = 160\#/\text{ft.}$

Distribution of Dead Load.

At (1.) 1/6 x 9,770 = 1,630# Flooring.

- 1,620# Stringer.

- 750# Guard.

1/6 x 400 = 67# Bridging.

- 360# Railing.

- 200# Railing posts.

* Only apply to end points. - (1)

1/7 x 254	=	37# Cross Bracing.
3 x 160	=	480# floorbeam.
Total		5,144# Say 5,200#
At (2), (3) & (4)	-	1,630# flooring.
	-	1,620# Stringer.
	-	67# Bridging.
	-	37# Bracing.
	-	480# Floorbeam.
Total		3,834# Say 3,900#

Distribution of Dead Load Stresses.

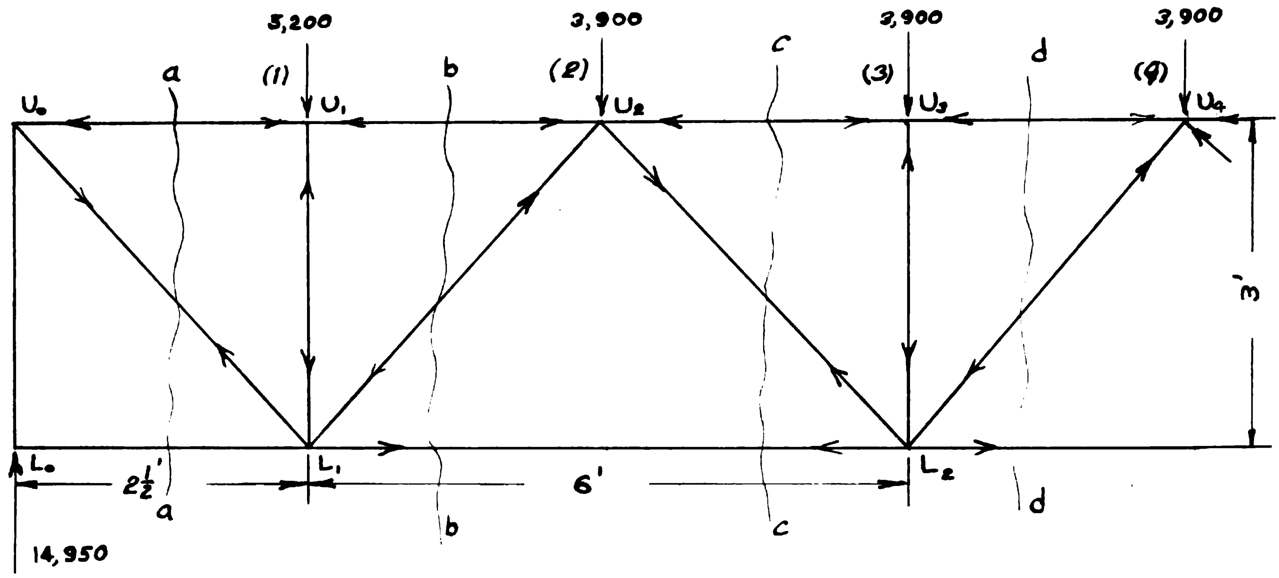


Fig. 10

Section a-a

Vert. comp. of $U_0L_1 =$ 14,950

Stress $U_0L_1 = 14,950 \times \frac{3.92}{3} =$ 19,500# ten.

Joint U_1

Stress $U.L_1 =$ 5,200# comp.

Section b-b

Vert. comp. of $L_1U_2 = 14,950 - 5,200 =$ 9,750

Stress $L_1U_2 = 9,750 \times \sqrt{2} =$ 13,800# Comp.

Section c-c

Vert. comp. of $U_2L_2 = 14,950 - 5,200 - 3,900 =$ 5,850

Stress U_2L_2 $5,850 \times \sqrt{2} =$ 8,300# Ten.

Section d-d

Vert. comp. of $L_2U_4 = 14,950 - 5,200 - 2 \times 3,900 =$ 1,950

Stress $L_2U_4 = 1,950 \times \sqrt{2} =$ 2,760# comp.

Joint U_3

Stress $U_3L_2 =$ 3,900# Comp.

Joint U_1

Stress $U_0U_1 = U_1U_2 = 19,500 \times \frac{2.5}{3.92} =$ 12,420# Comp.

Joint U_2

Stress $U_2U_3 = U_3U_4 = 12,420 + 9,750 + 8,300 =$ 30,470# Comp.

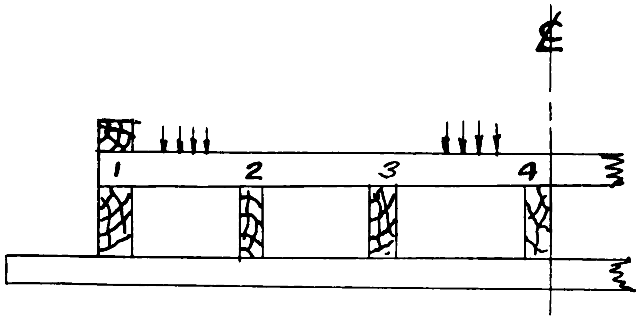
Joint L_1

Stress $L_1L_2 = 12,420 + 9,750 =$ 22,170# Ten.

Joint L_2

Stress $L_2L_3 = 22,170 + 5,850 + 1,950 =$ 29,970# Ten.

Live Load



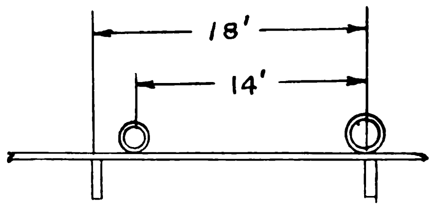


fig. 11

with the truck as in Fig. 11 the distribution of the live load will be as follows. The entire rear wheel loads will be transmitted to the floorbeam and 4/18 of the front wheel load. These loads will be transmitted to the floorbeam by the stringers at points 1, 2, 3 and 4 as shown below.

(1) --- 6,334# say 6,400#

(2) --- 6,334# say 6,400#

(3) --- 6,334# say 6,400#

(4) ---12,667# say 12,700#

Impact $\frac{23+20}{138+20} = 33.6$ use 30%.

(1) --- 6,400 x 1.3 = 8,320#

(2) --- 6,400 x 1.3 = 8,320#

(3) --- 6,400 x 1.3 = 8,320#

(4) --- 12,667 x 1.3 = 16,510#

Distribution of Live Load Stresses.

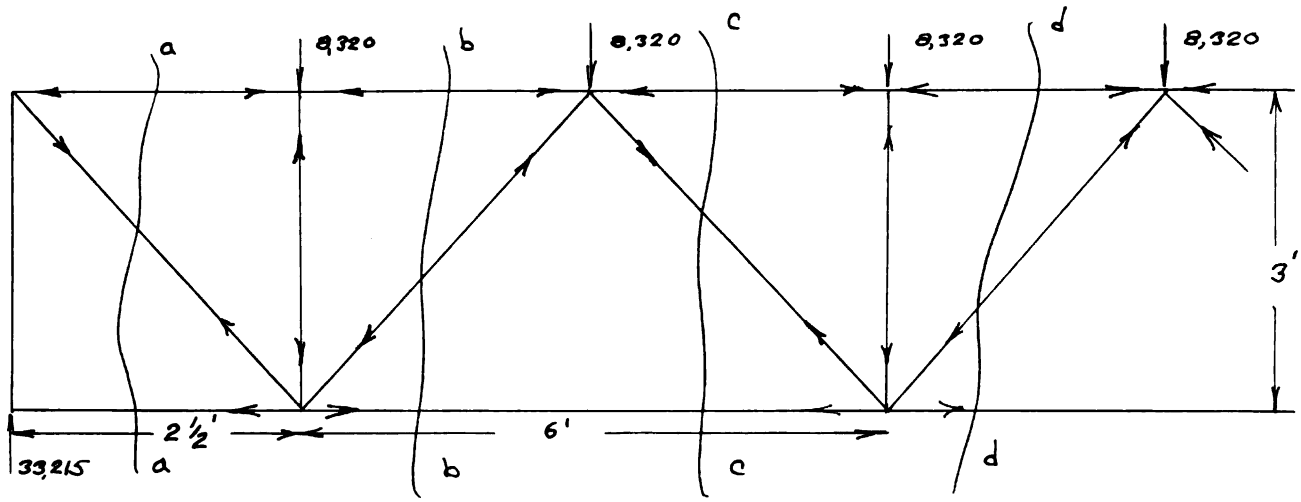


Fig. 12

Section a-a

Vert. Comp. $V_0L_1 = 33{,}215\#$

Stress $V_0L_1 = 33{,}215 \times \frac{3.92}{3}$ $= 43{,}400\#$ Ten.

Joint U_1

Stress U_1L_1 $= 8{,}320\#$ Comp.

Section b-b

Vert. Comp. $L_1U_1 = 33{,}215 - 8{,}320 = 24{,}895\#$

Stress $L_1U_1 = 24{,}895 \times \sqrt{2}$ $= 35{,}200\#$ Comp.

Section c-c

Vert. Comp. $U_2L_2 = 33{,}215 - 2 \times 8{,}320 - 16{,}575\#$

Stress $U_2L_2 = 16{,}575 \times \sqrt{2}$ $= 23{,}400\#$ Ten.

Joint U_3

Stress U_3L_2 $= 8{,}320\#$ Comp.

Section d-d

Vert. Comp. $L_2U_4 = 33{,}215 - 3 \times 8320 = 8{,}255\#$

Stress $L_2U_4 = 8{,}255 \times \sqrt{2}$ $= 11{,}680\#$ Comp.

Joint U_o

Stress $U_oU_1 = U_1U_2 = 42{,}200 \times \frac{2.5}{5.92}$ $= 26{,}900\#$ Comp.

Joint U_2

Stress $U_2U_3 = U_3U_4 = 26{,}900 + 24{,}895 + 16{,}575$ $= 68{,}370\#$ Comp.

Joint L_1

Stress $L_1L_2 = 33{,}215 + 24{,}895$ $= 58{,}110\#$ Ten.

Joint L_2

Stress $L_2L_3 = 58{,}110 + 16{,}575 - 8{,}255$ $= 82{,}940\#$ Ten.

Floorbeam Data

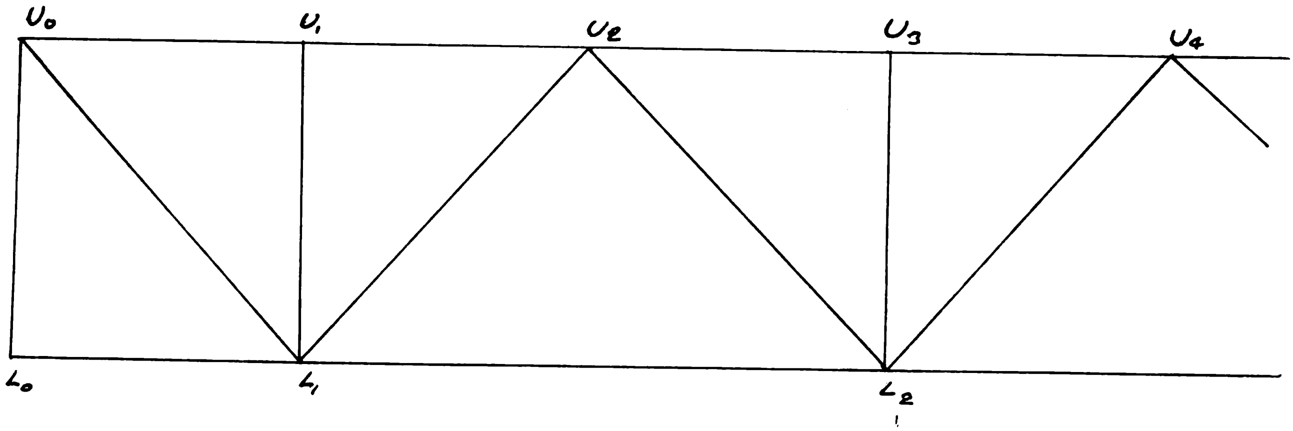


Fig. 13

Member	Total Stress	Length c-c	Area
V_oL_1	62,900# Ten	2.92^1	42 sq. in.
V_1L_1	13,520# Comp	3.00^1	12 sq. in.
L_1U_2	49,000# Comp.	4.24^1	44 sq. in.
U_2L_2	31,700# Ten.	4.24^1	21 sq. in.
U_3L_2	12,220# Comp.	3.00^1	11 sq. in.
L_2U_4	14,440# Comp.	4.24^1	13 sq. in.
U_oU_1	39,320# Comp.	2.50^1	35 sq. in.
U_1U_2	39,320# Comp.	3.00^1	35 sq. in.
U_2U_3	98,840# Comp.	3.00^1	87 sq. in.
U_3U_4	98,840# Comp.	3.00^1	87 sq. in.
L_oL_1	0	2.50^1	0
L_1L_2	80,280# Ten.	6.00^1	55 sq. in.
L_2L_3	112,910# Ten.	6.00^1	74.7 sq. in.

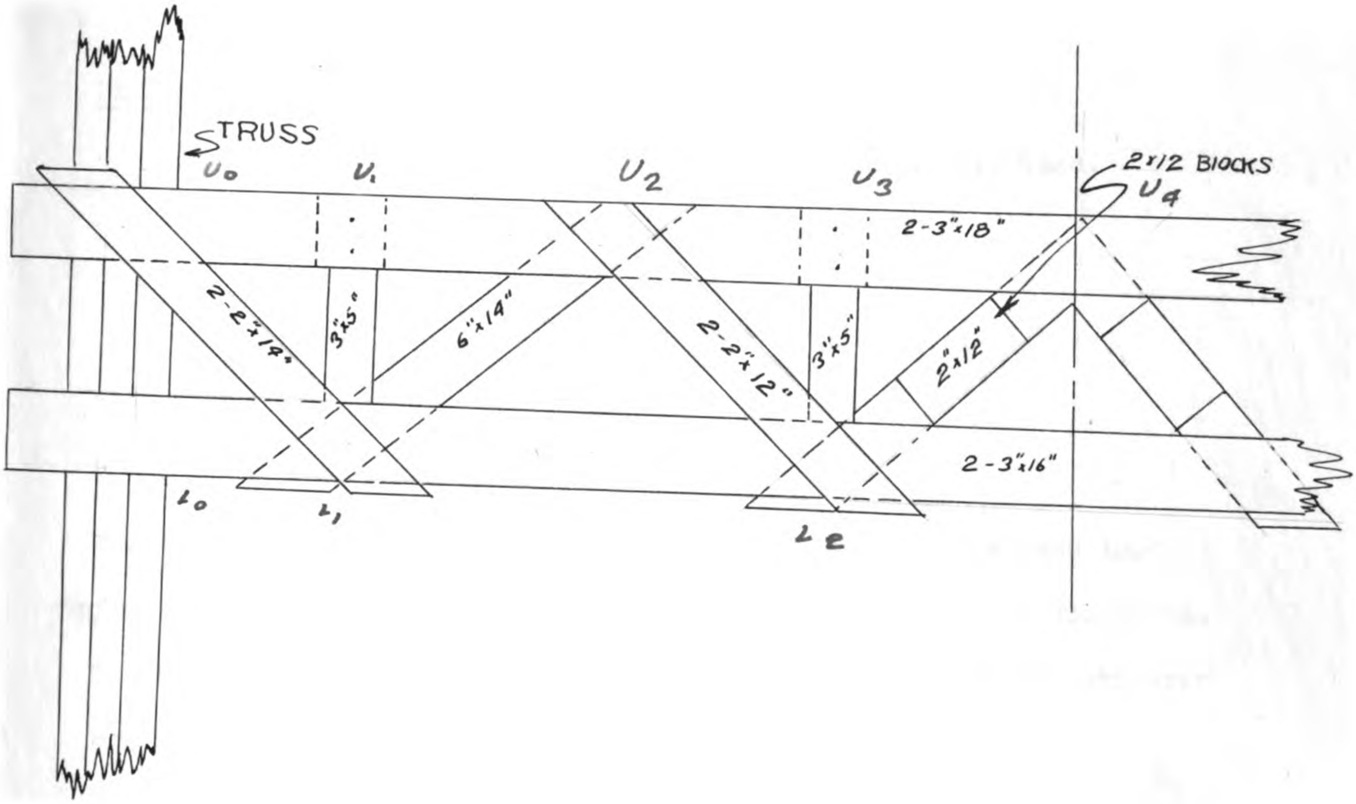


Fig. 14

The floorbeam is to be constructed as shown in Fig. 14. Split ring connectors are to be used in all stressed joints. The table below gives the No. and size of connectors to be used. For details of the joints see the drawings accompanying this thesis.

Member	No. Rings	Ring Size	Bolt Size
V_oL_1	5	4"	3/4"
L_1U_2	10	4"	3/4"
U_2L_2	3	4"	3/4"
L_2U_4	4	4"	3/4"
Truss	8	4"	3/4"

The floorbeam is checked for shear.

$$\frac{50,000}{200} = 250 \text{ lbs. per sq. in.}$$

The bolt holes for the 3/4" bolts shall be 11/16".

DESIGN OF 72' HOWE TRUSS:

In the truss design the loading shown in Fig. 15 was used.

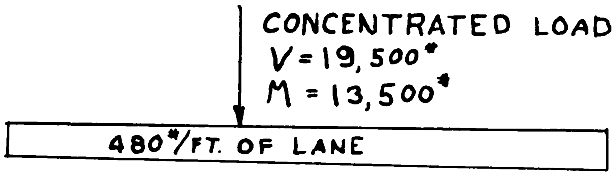


Fig. 15

The dead load transmitted at the panel points will be the dead load figured in the floorbeam design plus the true weight of the floorbeam. The dead load stresses are figured directly and the live load stresses are figured by means of influence lines.

Dead Load Stresses.

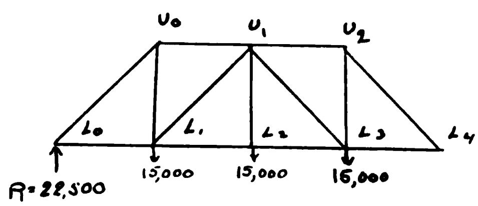


Fig. 16

$$\text{Stress } L_0U_0 = 22{,}500 \times \frac{23.42}{15} = 35{,}300\# \text{ Comp.}$$

$$\text{Stress } L_1U_1 = 7{,}500 \times \frac{23.42}{15} = 11{,}710\# \text{ Comp.}$$

$$\text{Stress } U_0L_1 = 15{,}000 + 7{,}500 = 22{,}500\# \text{ Ten.}$$

$$\text{Stress } L_0L_1 = 35{,}300 \times \frac{18}{23.42} = 27{,}200\# \text{ Ten.}$$

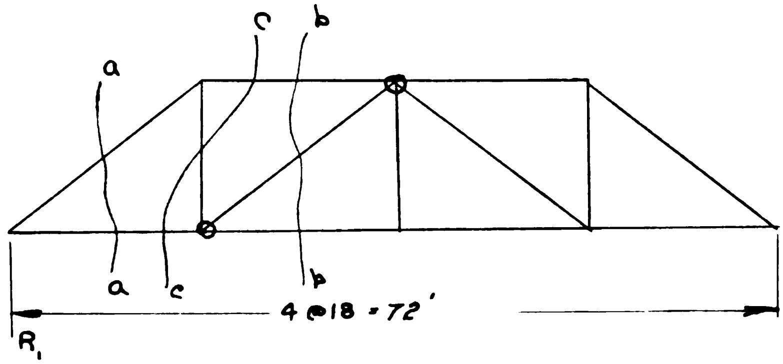
$$\text{Stress } L_1L_2 = 27{,}200 + 11{,}710 \times \frac{18}{23.42} = 36{,}200\# \text{ Ten.}$$

$$\text{Stress } U_1L_2 = 15{,}000\# \text{ Ten.}$$

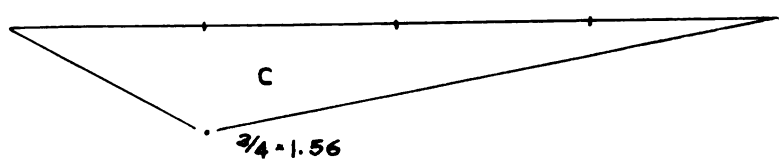
$$\text{Stress } U_0U_1 = 35{,}300 \times \frac{18}{23.42} = 27{,}200\# \text{ Comp.}$$

Live Load Stresses.

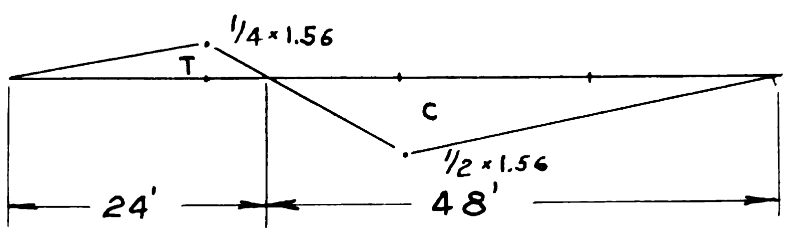
The influence lines used in finding the live load stresses are shown below.



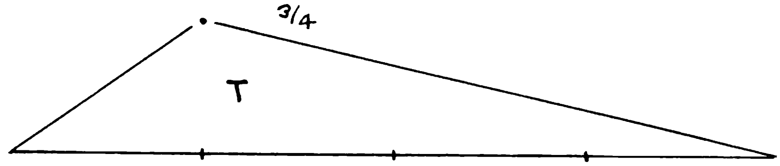
STRESS $L_0U_0 = \frac{2342}{15} \times R_1$ IN COMP. (SHEAR aa)



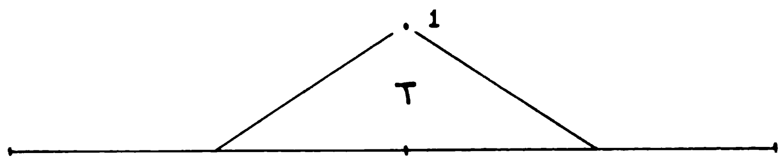
STRESS L_1U_1 = VALUE SHOWN × 1.56 (SHEAR bb)



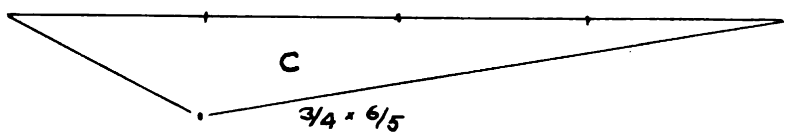
STRESS $U_0L_1 = R_1$ (SHEAR cc)



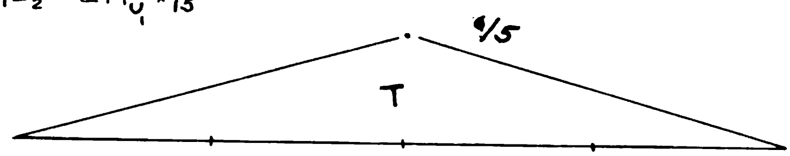
VERT. U_1L_2



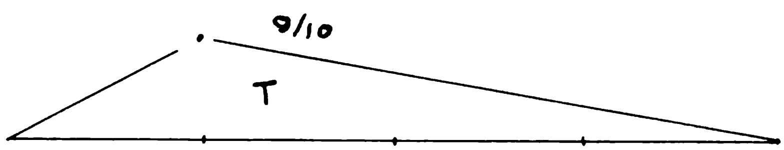
STRESS $U_0U_1 = R_1 \times 6/5$ COMP. (MOM. L_1)



STRESS $L_1L_2 = \Sigma M_{U_1} \times \frac{1}{15}$



STRESS $L_0L_1 = R_1 \times 6/5$



By means of the influence lines the live load stresses listed below were found.

Stress $L_oU_o = 72 \times 1.17 \times 1/2 \times 480 + 1.17 \times 19{,}500$ $= 43{,}100\#$ Comp. no reversal

Stress $L_1U_1 = 48 \times .78 \times 1/2 \times 480 + .78 \times 19{,}500$ $= 24{,}250\#$ comp.

$= 24 \times .39 \times 1/2 \times 480 + .39 \times 19{,}500$ $= 9{,}870\#$ Ten.

Stress $U_oL_1 = 72 \times .75 \times 1/2 \times 480 + .75 \times 19{,}500$ $= 27{,}700\#$ Ten. no reversal

Stress $L_oL_1 = 72 \times .90 \times 1/2 \times 480 + .90 \times 19{,}500$ $= 33{,}200\#$ Ten. no reversal

Stress $L_1L_2 = 72 \times 1.2 \times 1/2 \times 480 + 1.2 \times 13{,}500$ $= 37{,}000\#$ Ten. no reversal

Stress $U_1L_2 = 36 \times 1/2 \times 480 + 19{,}500$ $= 28{,}150\#$ Ten. no reversal.

Stress $U_oU_1 = 72 \times 1/2 \times .90 \times 480 + .90 \times 13{,}500$ $= 27{,}800\#$ Comp. no reversal

Using the impact formula and adding dead and live load stresses the table shown in Fig. 17 was computed. Since the dead load compressive stress in member L_1U_1 is greater than the live load tension obtained by the stress reversal no counters are needed. The compressive members must be designed as columns keeping in mind the ratio of (l/d).

Member	Total Stress	Length c-c	Area
L_oU_o	89,300# Comp.	23.42'	114 sq. in.
L_1U_1	43,310# Comp.	23.42'	63 sq. in.
U_oL_1	56,000# Ten.	15.00'	38 sq. in.
L_oL_1	69,000# Ten.	18.00'	46 sq. in.
L_1L_2	83,000# Ten.	18.00'	56 sq. in.
U_1L_2	51,800# Ten.	15.00'	35 sq. in.
U_oU_1	57,200# Comp.	18.00'	122 sq.in.

Fig. 17

The date shown in Fig. 19 is determined from the above stresses, taking into account the angle of the load to the grain. The number of rings shown is the minimum and in most cases has had to be increased in order to balance the load. The cross sectional area of the members given in Fig. 17 has also been increased to allow for net section, placing of connectors and in compression members to obtain a l/d ratio of less than 30.

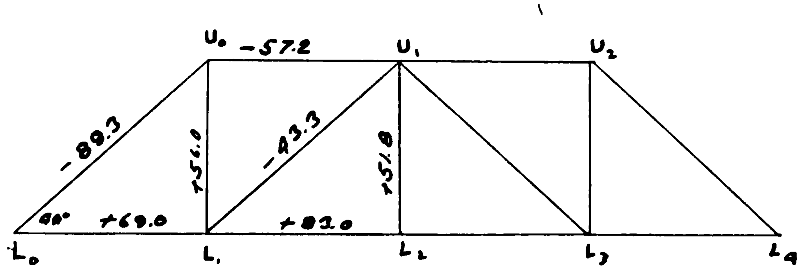


Fig. 18

In figuring the connectors for L_oU_o the vertical component of the stress was assumed as passing directly to the reaction. The horizontal component is equal and opposite to the stress in L_oL_1,. Using 4" Connectors the strength per connector in timber 3" and thicker is as follows.

Angle to grain	strength
0°	6,400#
40°	[illegible]
50°	5,400#
90°	4,600#

Member	No. Rings	Angle to grain	Bolt size
L_oU_o	16	40^o	3/4"
L_1U_1	8	40^o	3/4"
U_oL_1	12	90^o	3/4"
L_oL_1	11	0^o	3/4"
L_1L_2	13	0^o	3/4"
U_1L_2	12	90^o	3/4"
U_oU_1	9	0^o	3/4"

Fig. 19

Details of all joints and splices are shown on the drawings accompanying this thesis. The number of connectors in the vertical at joint L_1 may be greatly reduced, since the floor beam load is applied above this joint and the member is in tension.

Camber:

The bridge is to be cambered as follows:

Increase the lengths of top chord members U_oU_1 each by 1/4".

Raise floor beam at L_2U_1 by 1/4".

Lower end caps by 1/4".

Bracing:

There is no need for knee bracing since the floor beam acts as a knee brace itself. Since the truss is shallow and of a fairly short span the wind load will be small. 3"x8" bracing as shown is used as wind braces.

Truss Members:

Members L_oU_o, U_oU_1 and L_1U_1 carry compressive loads and will be designed as columns.

Take member L_oU_o first. Since this member carries quite a large stress it will be built up of three timbers and designed as a solid column. Member L_oU_o will be considered as an intermediate column where the l/d ratio is between 11 and K. K is a constant depending on E and C of the timber.

$K = .64\sqrt{\frac{E}{C}}$ Where E = Modulus of elasticity 1.6×10^6

C = Allowable unit stress parallel to grain

= 1,100#/sq. in.

$$K = .64\sqrt{\frac{1.6\times 10^6}{1,100}} = 24.4$$

For intermediate columns

$$P/A = c\left[1-\frac{1}{3}\left(\frac{l}{Kd}\right)^4\right]$$

According to the data in Fig. 17, computed from unit stresses, L_0U_0 should have a cross sectional area of 114 sq. in. Two 3" x 14" and one 6" x 8" would give on area of 119 sq. in. and a d of 14". Rearranging the above formula we get.

$$A = P \times \frac{1}{c\left[1-\frac{1}{3}\left(\frac{l}{Kd}\right)^4\right]}$$

Solving for A we get

A = 116 sq. in. which is satisfactory.

16 connectors will be needed between the center member and the outside members to develop the full strength of the member.

Member L_1U_1 has a K of 24.4 and trying two 3" x 12" and one 6" x 8" we have a d of 12". Substituting in the above formula for A we get.

A = 50 sq. in. which is satisfactory.

Member U_0U_1 will be designed as a spaced column as it would otherwise collect water.

For intermediate spaced columns

$$A = P \times \frac{1}{c\left[1-\frac{1}{3}\left(\frac{l}{Kd}\right)^4\right]}$$

Where $K_2 = 1.5811 \times K = 38.6$

Try two 6" x 16" timbers separated by a 8" block. $d = 6"$ Then for A we get $A = 60$ sq. in. Well within limits.

The remaining members carry tensial stresses and the areas of Fig. 17 must be adhered to. These timber members as well as the compressive timber members may have to be increased in size to allow for connector spacing at the joints and also for net section reduction due to bolt holes.

Deflection:

For deflection of the compressive members: $\delta = \dfrac{Pl^2}{2EI}$

for L_oU_o

$$\delta = \frac{89{,}300 \times (23.42)^2 \times 144}{2 \times (1.6 \times 10^6) \times 3{,}200} = .345".$$

for L_1U_1

$$\delta = \frac{43{,}310 \times (23.42)^2 \times 144}{2 \times (1.6 \times 10^6) \times 2{,}640} = .404"$$

for U_oU_1

$$\delta = \frac{57{,}200 \times (18)^2 \times 144}{2 \times (1.6 \times 10^6) \times 2{,}050} = .406"$$

The floorbeams are to be connected to the verticals by split rings. The maximum floorbeam reaction is 50,000# requiring 8 rings.

All bolts will have washers under the head and nut in accordance with Michigan State Highway Department Specifications.

For general notes see specifications for the Design of Highway Bridges adopted by the Michigan State Highway Department, January, 1936 numbers 129 through 149.

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