PRELIMINARY SURVEY AND COST OF A GRADE SEPARATION

U. S. -16 & P. M. R. R. 3 MILES WEST OF LANSING

THESIS FOR THE DEGREE OF B. S.

E. H. Aue A. E. Heath

1932

Preliminary Survey and Cost Estimate
Of a
Grade Separation
(U.S.-16 & P.M.R.R. 3 miles west of Lansing)

A Thesis Submitted to
The Faculty of
MICHIGAN STATE COLLEGE
of
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By

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THESIS

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The authors wish to take this opportunity to acknowledge their indebtedness to Professor C.L. Allen for his kind advice in matters of good engineering practice; and to other members of the Civil Engineering Department for their cooperation in allowing us the use of various instruments in connection with the work on this paper.

East Lansing
June, 1932

E.H.A.
A.E.H.

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DISCUSSION

The purpose of this Thesis is the determination of the location, type, and cost estimate of the completed separation.

A Topographic map was made to determine the location and type of separation to be constructed. It was found by inspection of the map that the railroad had already approached its maximum grade at this point. Therefore it would be very costly to change the elevation of the railroad.

It was decided after consulting a set of Michigan State Highway specifications, that the curves at the crossing were too sharp—especially if a separation were to be put in. Accordingly, a new reverse curve was run in with longer radii, and the road relocated.

It was also noted from the map that any dirt for a fill would have to be borrowed and hauled from some distant source, while excavation dirt could be wasted nearby; and would be much less than the fill for a separation, since the surface takes a distinct downward slope just to the west of the crossing. The specifications again influence the design when we consider that fourteen feet clearance is required over a Michigan Trunk Line, while a clear height above the tops of the rails of twentytwo feet is required

for railroad passage. There is a seven foot difference in depth of earth work — other factors being equal. Here also the matter of drainage appears in the project. It is comparatively inexpensive, however, and need not be given very much consideration.

Then the bridge problem is brought up. Would it be more economical to build a one-hundred fifty foot by sixteen foot railroad bridge or to build a highway bridge one-hundred seventy ft. wide and with a span of twenty feet. The continuous retaining wall abutments in the latter case would each be one-hundred seventy feet long and twenty six feet high. Retaining wings or head walls would be required in such a design.

The railroad truss bridge would be cheaper than the highway bridge due to cost of materials.

The summation of the above items indicates that the excavated separation would be far more economical to construct than the fill separation.

RELOCATION OF CENTER LINE

P.C. Sta. 0 +00

P.T. Sta. 1 +34.8

I $18^\circ\ 23'\ 00''$

$$R = T \cot.\frac{I}{2} = 833.0'$$

$$\sin\frac{D}{2} = \frac{50}{833} =$$

$$\therefore \frac{D}{2} = 3^\circ\ 26'\ 27''$$

$$\frac{d}{2} = 1^\circ\ 43'\ 13.5''$$

$$L_c = 100\frac{I}{D} = 267.1'$$

This curve is south and east of the rail road.

P.C. Sta. 2 +67.1

P.I. Sta. 3 +73.8

I. $30^\circ\ 12'\ 00''$

$$R = T \cot.\frac{I}{2} = 414.0'$$

$$\sin\frac{D}{2} = \frac{50}{414}$$

$$\therefore \frac{D}{2} = 6^\circ\ 56'\ 00''$$

$$\frac{d}{2} = \frac{D}{4} = 3^\circ 28'\ 00''$$

$$L_c = \frac{100I}{D} = 217.8'$$

This curve is north and ~~east~~ west of the rail road.

SURFACE ELEVATIONS ON RELOCATION

Sta.	50'left	25'left	center line	25'right	50'right.
00 00			852.1		
00 50			851.6		
1 00			851.1		
1 50			850.5		
2 00			850.0		
2 50			849.5		
3 00	847.9	847.2	.49.0	847.2	847.6
3 50	47.1	46.7	48.5	46.5	47.1
4 00	45.9	45.8	47.8	45.5	46.6
4 32.9			P.C.47.5		
4 50	45.8	45.7	47.3	46.1	45.5
5 00	46.6	45.3	46.6	46.6	44.7
5 50	47.1	45.2	44.8	46.4	44.5
6 00	45.3	45.2	45.5	46.3	46.3
6 50	44.5	45.1	45.9	46.3	46.2
R.R. 7 00	47.3	46.2	P. T45.4	44.4	44.5
7 50	44.5	44.9	45.8	45.5	46.7
8 00	42.8	46.5	45.3	46.5	47.0
8 50	44.0	46.4	45.2	45.0	43.5
9 00	42.8	46.9	44.4	44.7	41.0
9 50	44.6	45.7	43.2	42.7	44.2
10 00	44.8	41.0	41.8	43.6	46.5
10 50	42.3	41.3	P.T.42.0	40.8	41.7
11 00			42.5		

FINAL ELEVATIONS OF SUB-GRADE

Sta.	℄ Elv.	Sta.	℄ Elev.
0 00	853.1	6 25	827.7
0 50	P.C. 51.6	6 50	27.0
075	51.2	6 75	26.6
1 00	50.8	7 00	P.I.26.5 R.R.
1 25	50.3	7 25	26.6
1 50	49.7	7 50	27.0
175	49.1	7 75	27.7
2 00	48.3	8 00	28.5
2 25	47.6	8 25	29.6
2 50	P.I.46.7	8 50	P.T. 31.0
2 75	45.7	8 75	P.C. 32.4
3 00	44.7	9 00	33.8
3 25	43.6	9 25	35.0
3 50	42.4	9 50	36.1
3 75	41.2	9 75	37.1
4 00	39.9	10 00	38 0
4 25	38.5	10 25	38.8
4 50	P.T. 37.0	10 50	P.I. 39.5
4 75	35.5	10 75	40.0
5 00	34.0	11 00	40.5
5 25	32.5	11 25	40.9
5 50	P.C.31.0	11 50	41.1
5.75	29.6	11 75	41.2
6 00	28.5	12 00	41.3

FINAL ELEVATIONS OF SUB-GRADE
(Continued)

Sta.	Elevations
12 25	841.2
12 50	P.T. 41.0

NOTE:-

Stations of P.C.,P.T. and P.I. on vertical curves have elevations so marked.

EARTH WORK DESIGN FROM CROSS SECTIONS

Station		Area of sec. (sq. ft.)	Volume (cu. yd.)	Mass.
0	00	0 00	0 00	0
0	50	0 00		
			20.9	20.9
1	00	22.5		
			81.8	102.7
1	50	65.8		
			187.0	289.7
2	00	136.0		
			314.5	604.2
2	50	203.5		
			342.5	946.7
3	00	166.5		
			393.0	1339.7
3	50	258.5		
			571.8	1911.5
4	00	359.0		
			835.5	2747.0
4	50	543.7		
			1222.0	3969.0
5	00	776.5		
			1656.0	5625.0
5	50	1012.5		
			2040.0	7665.0
6	00	1191.5		
			2368.0	10033.0
6	50	1367.5		
			2566.0	12599.0
7	00	1403.8		
			2550.0	15149.0
7	50	1350.6		
			2370.0	17519.0
8	00	1208.7		
			1976.0	19495.0
8	50	926.0		
			1490.0	20985.0
9	00	683.0		22020.0
			1035.0	~~22912~~
9	50	436.3		
			602.0	22622.0
10	00	215.0		
			290.0	22912.0
10	50	98.5	129.0	
				23041.0
11	00	40.5		
			39.0	23080.0

DESIGN OF EARTH WORK CONTINUED

Station	Area of sec. (sq. ft.)	Volume (cu.yd.)	Mass
11 50	00.0		
		000	23080.0
12 00	00.0		
		000	23080.0
12 50	00.0		
		000	23080.0

Haul:

Station 6 + 50 = mean of Mass

Point of waste is 2000 ft. away.

1000ft. free haul.

∴ Haul =10x23080= 230800 yd. stations.

This is on the safe side since super elevation was not considered on either curve.

Construction of pavement:-

$$\frac{1250x\ 40}{9sq.'/sq.\ yd.} = 5560 \text{ sq. yds. slab.}$$

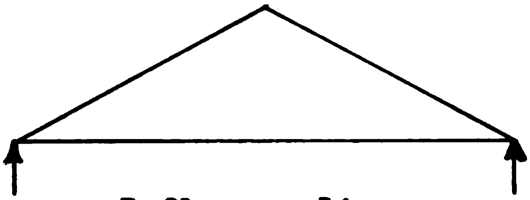
2500 linear feet of curb and gutter.

1 cent. pump and automatic controlled electric motor

2 Catch basins at bottom of cut.

700ft. 10" tile from pump outfit to placement.

DESIGN OF STRINGERS



Influence Line

Maximum Moment Average load method

load (3)	rt. ℄	$\frac{30}{1.25} = 2.40$	$\frac{90}{12.5} = 7.20$	
				reverse
" "	lft. ℄	$\frac{60}{12.5} = 4.80$	$\frac{60}{12.5} = 4.80$	
" (4(	rt. ℄	$\frac{60}{12.5} = 4.80$	$\frac{60}{12.5} = 4.80$	reverse
" "	lft. ℄	$\frac{90}{12.5} = 7.20$	$\frac{30}{12.5} = 24.0$	

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load (3)

$$M = \left(\frac{1245 + 135 \times 2.5 - 15 \times 30.5}{25}\right) 12.5 - 345 + 15 \times 13$$

M = 450 'k.

load (4)

$$M = \left(\frac{1245 + 135 \times 2.5 - 15 \times 25.5}{25}\right) 12.5 - 320 + 15 \times 18$$

M = 450'k.

M = 450'kips

Shear

load (1) lft. reaction

$$R = \frac{1245 + 135 \times 2}{25} = 60.6 \text{ kips}$$

load (2) lft. reaction

$$R = \frac{2460 + 154.5 - 33 \times 15}{25} = 72.5 \text{ kips.}$$

Shear = 72.5 kips

Note: We wish to obtain as shallow a floor system as possible to eliminate excessive earth work.

Web:

$\frac{167.46}{10} = 16.75$ sq. in. Net for Shear.

7/8" rivets $.601 \times 10{,}000 \times 2 = 12{,}020$ #/ rivet in Shear.

$\frac{167{,}460}{12{,}020} = 14$ rivets required

$7/8 \times 7/8 \times 24{,}000 = 18380$ #/rivet in Bearing.

$\frac{167{,}460}{18{,}380} = 10$ rivets required

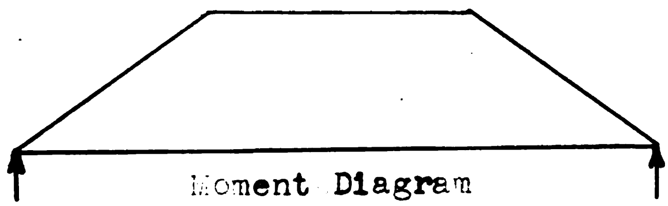
14 Rivets limits the Depth.

A 33x 15¾ x 210# I-Beam is sufficient.

Use a CB section as given.

A built up section from a rough estimate would be several inches deeper and about 10 lbs. heavier than the I-beam. We are striving for a shallow floor system as stated before.

DESIGN OF FLOOR BEAMS



Shear

Dead stringer 210#/ft.

210x25=..........................5250 #

4Hitch angles 6"x6"x29" =

4x19.6x29/12 =.......................189.5#

Track & rails 400/2 x 25 =...........5000 #

Floor beams assumed @300#/ft.

300x8=........2400 #

Total.......12839.5#

Live

Using loads 5 or 14 on beam = ...110.2 kips

Impact " " " " " " " =......94.9 "

Total moving load shear=205.1 "

Total max shear = 217.9 "

Moment

Same loading as for maximum shear

Moving = 205.1x3.25- 205.1x8= ....924.0 kip ft.

Dead 1284x8- 10.44x3.25 -300x4x8= 59.15 " "

Max Total moment = 983.15 " "

Note: Live loading Cooper's E-60

Rivets

shear $\frac{217.9}{12x.6x2} = 15.12$ rivets necessary.

Assume a 40" webb

$\frac{217.9}{10x40} = .544"$ thickness webb required

to carry shear. Use $\frac{13}{16}$" Webb.

Bearing $\frac{217.9}{\frac{13}{16}x20x\frac{7}{8}} = 15.3$ rivets in bearing.

USE 16 RIVETS.

Flanges

Assumed effective depth = 37"

$\frac{983.15x12}{37x\ 16} = 19.9$ sq. in. needed for tension.

Assume 4 6"x6"x$\frac{7}{8}$"∠s for flanges.

$S_c = 16,000 - \frac{200x14x12}{13.813} = 13570$# per sq. in.

USE 13500 #

$\frac{983.15x12}{\#37x13500} = 23.62$ sq. in. needed for compression.

Area of two 6x6x$\frac{7}{8}$∠s = 19.46 sq.in.

$40x\frac{13}{16}x\frac{1}{8} = 2.70$ sq. in. taken by the webb.

19.46+2.70-1.75=20.41 sq. in. available for tension

and 19.9 are needed.

OK Tension.

19.46+2.70 = 22.16 sq. in. available for compression

and 23.62 are needed.

Change Compression.

DESIGN OF FLOOR BEAMS CONTINUED

We decided to use an angle of larger dimension rather than use a cover plate.

Try 4-6"x6"x1"∠s = gross area for 2 = 22sq. in.

Area needed in compression $= \frac{983.15x12}{36.73x13.5} = 24.45$ sq. in.

Area needed in the angles = 24.45-2.70 = 21.75 sq.in.

We have 22" available. OK COMPRESSION.

Area needed in tension $= \frac{983.15x12}{36.72x13.5} = 20.5$ sq. in.

Area needed in ∠s = 20.5+ 1.75-2.70 = 19.55 sq. in.

We have available 22-1.75 = 20.25 sq. in.

OK tension.

Webb Stiffeners investigation:

$40.5-2x6\frac{5}{8} = 27.25$ in.clear.

$\frac{.8125}{27.25}$ = more than $\frac{L}{60}$

Stiffeners are unnecessary.

Rivet spacing

$\frac{27.25 - 3}{16} = 1.51$"spacing of rivets.

Rivets must be staggered being spaced 3" apart and having 2 rows using total of 17 rivets.

WEIGHT OF FLOOR SYSTEM ON NEXT PAGE.

FLOOR BEAMS CONTINUED

Webb $\frac{13}{16}$ x40x16 = 110.5 x16 = ...1770 #

4∠s 6"x6"x1"x16' = 4x16x37.4 = ...2397 #

8 h∠s 6"x6"x$\frac{1}{2}$" = 8x19.6x$\frac{29}{12}$ = ...379 #

4 " " " " = 4x28.7x$\frac{38}{12}$ = ...364 #

400 rivets @25#/ 100 = ...100 #

Weight of floor beam = ...5010 #

The estimate of 300#/ft. was good

if hitch angles were neglected the wt./ft.

would be 290.3#/ft.

The weight of floor system resting on the

floor beam = ...210+ 200 = 2 x25 = 20,500. #

WEIGHT OF FLOOR SYSTEM PER PANEL = ... 25,510 #

DESIGN OF WARREN TRUSS

The bridge will be 150 ft. long consisting of 6panels at 25 ft. per panel. The trusses will be 29 ft. high and will be spaced 16ft. on centers. The truss will be built with verticals at all panel points.

We assume a dead weight of 1300 lbs. per foot of truss. The dead stresses were figured by the index method. The diagonal forces being multiplied by constants which were figured from proportion of lengths.

$$\text{Diagonal} = \frac{38.29}{25} = 1.53 \times \text{horizontal force.}$$

$$\text{" " } \frac{38.29}{29} = 1.32 \times \text{vertical force.}$$

The members of the bridge are numbered as follows; Starting at the left the lower chord members are numbered $L_1 L_2$ $L_2 L_3$ and so on up to L_6. The upper chord members are numbered starting at the left with $U_1 U_2$ and so on up to U_5. This arrangement causes all vertical members to be numbered with $U_1 L_1$ and so on up to $U_5 L_5$, and the diagonals L_0U_1, U_1L_2 and so on up to U_5L_6.

Dead stresses will be found on the tabulated stress sheet*and on Plate II of the drawings.

We here wish to note that ½ the dead stress was assumed to be carried by the upper chord and ½ by the lower chord.

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*Page -

COMPUTATIONS LIVE AND IMPACT STRESSES

L_0L_1 L_1L_2

Loads at panel point I

load 3 $\frac{\frac{24546+426x29z14.5}{6} - 15x8}{29} = 207.4$ kips

" 4 $\frac{\frac{24546+34x426+34x17x3}{6} - 720}{29} = 209.4$ kips

" 5 $\frac{\frac{24546+39x426+39x195x3}{6} - 1245}{29} = 207.5$ kips

Max Live and Impact stresses (live = 209.4 kips

(impact= $\frac{209.4x300}{445}$ =141.1kips

L_0U_1

Loads to right or left of right reaction.

load 1 rt. $\frac{24546+3x41x20.5}{150}$ = 180.5 kips

" " lt. $\frac{24546+3x41x20.5-15x150}{150}$ =165.5 kips

" 2 rt. $\frac{24546+3x49x24.5+(120+2250)}{150}$ = 171.75kips

MAX Live and Impact stresses (live=180.5 x1.32 =238.1 kips

(impact= $\frac{238.1x300}{450}$ =158.8 "

U_1L_1 U_3L_3 U_5L_5

Loads placed on panel point V

load 3 $\frac{3232.5+174-690}{25}$ = 108.5kips

" 4 $\frac{4276.5-1440}{25}$ = 113.4 kips

" 5 $\frac{5244-2490}{25}$ = 110.16kips

Max Live and Impact stresses (113.4 live

($\frac{113.4x300}{344}$ = 99.2 kips

Impact figured by $\frac{\text{max stress x 300}}{\text{300+loaded length of truss}}$

COMPUTATIONS LIVE AND IMPACT (CONTINUED)

U_1U_2 U_2U_3

Loads placed on panel point II

load 6 $$\frac{\frac{24546+(426\times23)+(3\times23\times11.5)}{3}-2460}{29}=318.8 \text{ kips}$$

" 7 $$\frac{\frac{24546+(426\times28)+(3\times28\times14)}{3}-3232.5}{29}=321.2 \text{ kips}$$

" 8 $$\frac{\frac{24546+(426\times34)+3\times34\times17}{3}-4276.5}{29}=321.0 \text{ kips}$$

Max live stress = 321.2 kips Max impact $=\frac{321.2\times300}{437}=$ 222 kips.

L_2L_3

Loads at panel point III

load 11 $$\frac{\frac{24546+(426\times30)+(3\times30\times15)}{2}-8772}{29}=364.34 \text{ kips}$$

" 12 $$\frac{\frac{24546+(426\times35)+(3\times35\times17.5)}{2}-10062}{29}=364.98 \text{ kips}$$

" 13 $$\frac{\frac{24546+(426\times40)+(3\times40\times20)}{2}-11502}{29}=361.75 \text{ kips}$$

Max live stress = 364.98, Max impact $=\frac{364.98\times300}{444}=$ 246.5 kips

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U_1L_2

Loads on panel point II for tension

load 4 $$\frac{24546+(426\times9)+(9\times4.5\times3)}{150}-\frac{15\times30}{25}-\frac{15\times18}{25}= 161.1 \text{ kips}$$

" 3 $$\frac{24546+(426\times4)+(3\times4\times2)}{150}-\frac{15\times13}{25}-\frac{30\times20}{25}=161.36 \text{ kips}$$

Max live stress = 161.36x1.32 = 213 kips)
 tension
" impact stress $=213\times\frac{300}{413}=155$ kips)

- -

Loads on panel point V for compression

COMPUTATIONS LIVE AND IMPACT (CONTINUED)

U_1L_2

load 2 $\frac{2460+154.5\times 1}{150}+\frac{3\times 15}{25}=12.63$ kips

" 3 $\frac{3232.5+174}{150}+\frac{13\times 15}{25}+\frac{5\times 30}{25}=8.91$ kips

Max live stress= $12.63\times 1.32= 16.7$ kips)
" impact stress= $\frac{16.7\times 300}{413} = 15.04$ kips) compression

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$L2U_3$

Loads on panel point III for compression
load 2

$\frac{13092+34\times 4}{150}-\frac{8\times 15}{25}=91.76$ kips

" 3 $\frac{16224-6}{150}-\frac{13\times 15}{25} = 94.36$ kips

" 4 $\frac{18061.5}{150}-\frac{18\times 15}{25}-\frac{15\times 30}{25}=91.60$ kips

Max live stress= $94.36\times 1.32=124.4$ kips)
compression.
" impact stress = $\frac{124.4\times 300}{388}= 95.75$ kips)

Loads on panel point IV for tension.
load 2

$\frac{6948+(2\times 228)}{150}-\frac{8\times 15}{25}=44.56$ kips

" 3 $\frac{6948(228\times 7)}{150}-\frac{13\times 15}{25}-\frac{5\times 30}{25}= 43.16$ kips

Max live stress= $44.56\times 1.32 = 58.8$ kips)
tension.
" impact stress= $\frac{58.8\times 300}{363} = 48.6$ ")

There will be no live load stress on U_2L_2 as it carries only its own weight and is used more as a brace.

STRESS TABLE FOR TRUSS

All stresses are given in kips and tenths of kips.

Member		Dead	Live	Impact	Total
L_0U_1		-107.2	-238.1	-158.8	504.1 Comp.
U_1L_1		+ 24.4	+113.4	+ 99.2	237.0 Ten.
L_1L_2		+ 70.2	+209.4	+141.1	420.7 Ten.
L_0L_1		"	"	"	" "
U_1U_2		-112.2	-321.2	-222.0	655.5 Comp.
U_2U_3		"	"	"	" "
L_2L_3		+126.4	+364.98	+246.5	737.9 Ten.
U_1L_2	(	+ 64.4	+ 213	+155	432.4 Ten.
	R(	+ 64.4	- 16.7	-15.04	32.7 Ten.
U_3L_2	(	- 21.4	-124.4	- 95.75	241.5 Comp.
	R(	- 21.4	+ 58.8	+ 48.6	86.0 Ten.
U_2L_2		- 8.1			8.1 Comp.

Only the members of half the span have been figured this is allowable as the truss is symetrical about its center line.

DESIGN OF TRUSS MEMBERS

DESIGN L_2L_3

Stress=737,900# tension.

Req'd area $= \frac{737,900}{16,000} = 46.20$ sq. in.

Trial section-

4∠s 3½x3½x5/8"=15.92 sq. in.

4 webs@21"x½" =42.00 " "

Gross area =57.92 " "

-Rivet holes

4 1"x5/8"=2.50 sq. in.

12 1"x½" =6.00 " "

Rivet holes=8.50 " "

57.92-8.50=49.42 sq. in.

Member is OK.

DESIGN L_1L_2

Stress=420,700# tension.

Req'd area $= \frac{420,700}{16,000} = 26.30$ sq. in.

Trial section-

4∠s 3½x3½x9/16"=14.47 " "

2 webs@21"x7/16=18.38" "

Gross area =32.86 " "

-Rivet holes

4 1"x916" = 2.25 " "

2x2x1"x7/16"=1.75 " "

Rivet holes= 4.00 sq. in.

32. 86-4.00=28.86 sq. in.

Member is OK.

DESIGN OF TRUSS MEMBERS CONTINUED

DESIGN U_1L_2

Stress = 432,400# tension.

Req'd area $= \frac{432,400}{16,000} = 27.01$ sq. in.

Trial section-

4∠s $3\frac{1}{2} \times 3\frac{1}{2} \times 9/16" = 14.48$ sq. in.

2 webs @ $21" \times 7/16" = 18.38$ " "

Gross area = 32.86 " "

-Rivet holes

4 $1" \times 9/16" = 2.25$ " "

4 $1" \times 7/16" = 1.75$ " "

Rivet holes = 4.00

32.86 - 4.00 = 28.86 sq. in. Net area.

Member is OK.

DESIGN U_1U_2 & U_2U_3

Stress = 655,500# compression.

S allowable $= 16,000 - \frac{70 \times 25 \times 12}{r}$

Assume $S = 13,500$#/sq."

Req'd area $= \frac{655,500}{13,500} = 48.5$ sq. in.

Trial section-

4∠s $3\frac{1}{2} \times 3\frac{1}{2} \times 5/8" = 15.92$ sq. in.

2 webs @ $20" \times 11/16" = 27.50"$ "

1C Plt. @ $24" \times \frac{1}{2}"$ = 12.00" "

Total area = 55.42" "

$Z = \frac{12.00 \times (10\ 1/8 + \frac{1}{4})}{55.42} = 2.25$ "

I (c. plt.) $= 1 \times 24 \times (\frac{1}{2})^3/12 + 12.0 \times (10\ 1/8 + \frac{1}{2} - 2.25)^2 = 344"^4$

I (top ∠s) $= 2(4.3 + 3.98(10\ 1/8 - 1.1 - 2.25)^2)$ = 375 "

DESIGN U_1U_2 & U_2U_3 continued.

$$I(\text{bot.}\angle s) = 2(4.3+3.98(10\ 1/8-1.1+2.25)^2) = 1023''^4$$

$$I(\text{webs}) = 1 \times 1\ 3/8 \times 20^3 + 27.50 \times (2.25)^2 = 1137''$$

$$I = \ldots\ldots\ldots\ldots 3379''^4$$

$$r = \sqrt{\frac{3379}{55.42}} = 7.8 \qquad r = \sqrt{\frac{I}{A}}$$

$$S = 16{,}000 - \frac{70 \times 25 \times 12}{7.8} = 13{,}310\#/\text{sq.}''$$

$$\text{Area} = \frac{655{,}500}{13{,}310} = 49.1 \text{ sq. in.}$$

Also
$$I = C.P. = \frac{\tfrac{1}{2} \times (24)^3}{12} = \ldots\ldots\ldots\ldots 576''^4$$

$$4\angle s = 4(4.3+3.98(7\ 5/8+11/16+1.10)^2) = 1431''^4$$

$$2\text{webs} = 2\left(\frac{20 \times (11/16)^3}{12} + 13.75(7\ 5/8+11/32)^2\right) = 1746''^4$$

$$I = \ldots\ldots\ldots\ldots 3753''^4$$

$$r = \sqrt{\frac{3753}{55.42}} = 8.22$$

$$S = 16{,}000 - \frac{70 \times 25 \times 12}{8.22} = 13{,}450\#/\text{sq.}''$$

$$\text{Area} = \frac{655{,}500}{13\text{m}450} = 48.70 \text{ sq. in.}$$

$$\text{Actual stress} = \frac{655{,}500}{55.42} = 11{,}800\#/\text{sq.}''$$

Latticing-

Double latticing

$$s = \frac{280Ar}{C}$$

$$S = \frac{280 \times 55.42 \times 8.22}{12} = 10{,}620\#$$

$\frac{S}{2} = \frac{10{,}620}{2} =$ 5,310# caried by latticing.

Trial I Double 45° rivet at center.

$$S = \frac{10{,}620\sqrt{2}}{4} = 3760\#/\text{bar.}$$

Try $2\tfrac{1}{2}''\times 9/16''$ bar.

DESIGN OF U_1U_2 & U_2U_3 continued.

$S=\frac{3760}{2\frac{1}{2}x9/16}=2673\#/sq.\ in.$

$r=0.163$

$S=16,000-\frac{70x292}{0.163}=3460\#/\ sq.\ in.$

OK.

Trial II
60^o center riveted $2\frac{1}{2}x9/16"$ bars.

$S=\frac{10,620}{4}x\frac{1}{\sin 60^o}=3065\#/$~~sq.~~ ~~in.~~bar.

$S=\frac{3065}{2\frac{1}{2}x9/16}=2180\#/sq.\ in.$

$r=0.163$

$S=16,000-\frac{70x23.8}{0.163}=5770\#/sq.\ in.$

USE $60^o-2\frac{1}{2}"x9/16"$ bars.

$E=\frac{s}{d}$ $def.=\frac{sl}{E}=\frac{11,800x11.9}{30,000,000}=00.00422$

$def.=23.76136-\sqrt{20,625^2+11.88063^2}=.0025$

$S=\frac{.0025x30,000,000}{23.7614}=3290\#/sq.\ in.$

Total $S=2180+3290=5470\#/sq.\ in.$

OK.

DESIGN U_3L_2

(241,500# compression
Stress= (86,000# tension

Try I

2[s 15"-45# area= 26.34

$S=16,000-\frac{10x38.3x12}{5.33}=9,960\#/sq."$

$Area=\frac{241,500}{9960}=24.31\ sq.\ in.$

$I=2(10.3+13.17(7\ 5/8+0.79)^2)=1886.6"^4$

$r=\sqrt{\frac{1886.6}{26.34}}=8.47$

DESIGN OF TRUSS MEMBERS CONTINUED

DESIGNU_3L_2 continued

$S = 16,000 - \frac{70 \times 38.3 \times 12}{8.47} = 12,200$#/ sq. in.

$Area = \frac{241,500}{12,200} = 19.85$ sq. in.

OK.

Actual $stress = \frac{241,500}{26.34} = 9,180$#/ sq. in.

Check for Reversal-

Tension stress = 86,000

Net area $needed = \frac{86,000}{16,000} = 5.34$ sq. in.

OK.

Latticing -

$S = \frac{280 \times 26.34 \times 8.47}{11.243} = 5550$

Try double @45°

$S = \frac{5550}{4} \times \frac{1}{\sin 45^\circ} = 2175$#/ bar

Try $2\frac{1}{2}$"x$\frac{1}{2}$"bars.

$S = \frac{2175}{2\frac{1}{2} \times \frac{1}{2}} = 1740$#/sq. in. actual

$r = \frac{1}{2} \times 0.288 = 0.144$

$S = 16,000 - \frac{70 \times 28}{0.144} = 2400$#/sq. in. allowable.

$e = \frac{sl}{E} = \frac{9000 \times 22.486}{30,000,000} = .0067458$

$27.9306 - \sqrt{19.75^2 - 19.7432^2} = .0053$

$S = \frac{.0053 \times 30,000,000}{27.9306} = 570$#/sq. in.

$S = 1740 + 570 = 2310$#/sq. in. actual

OK.

DESIGN U_3L_2 continued

Check flanges:-

$$r = \frac{3.618}{\sqrt{2}} = 1.05$$

$$\frac{l}{r} = \frac{19.75}{1.05} = 18.9$$

$$\frac{38.3 \times 12}{8.47} = 54.3$$

OK.

DESIGN L_2U_3

Stress = 8,125

$$\text{Area} = \frac{8,125}{13,500} = 0.602 \text{ sq. in.}$$

Fro m former structures designed

4 ∠s 3½x3½x3/8" = 9.92sq.in.

1 web@13½"x3/8" = 5.06 sq. in.

Total area = 14.98 sq.in.

OK.

Use ¾" connecting plates at the jointsof the trusses.

DESIGN OF U_1L_0

Stress=504,100 # compression

$$S \text{ allowable} = 16,000 - \frac{70 \times 38.29 \times 12}{r}$$

S assumed=13,500#/ sq. in.

$$\text{Area req'd} = \frac{504,100}{13,500} = 37.34 \text{ sq. in.}$$

DESIGN OF TRUSS MEMBERS CONTINUED

DESIGN OF U_1L_0 continued.

Trial section:-

4∠s 3½"x3½"x½"	=13.00 sq. in.	
2 webs@20"x½"	=20.00"	"
1 C.Plt.24"x½"	=12.00 "	"
	45.00	
Total area	=~~54~~.00 "	"

$$Z = \frac{9.00 \times (10\ 1/8 + \tfrac{1}{2})}{42.00} = 2.21''$$

I -C. Plt. $= 1 \times 24 \times \tfrac{1}{2}^3 + 9.0 \times (10\ 1/8 + \tfrac{1}{2} - 2.21)^2 =$ 602"^{4}

-Top∠s $= 2(3.6 + 3.25(10\ 1/8 - 1.06 - 2.21)^2 =$ 312"^{4}

Bot∠s $= 2(3.6 + 3.25(10\ 1/8 - 1.06 + 2.21)^2 =$ 834"^{4}

Webs $= 1 \times 1 \times 20^3 + 20 \times (2.21)^2 =$ 765"^{4}

I total=.............2513"^{4}

$$r = \sqrt{\frac{2513}{45.00}} = 7.47$$

$$S = 16{,}000 - \frac{70 \times 38.29 \times 12}{747} = 11{,}580''/\text{sq. in.}$$

$$\text{Area req'd} = \frac{504{,}100}{11{,}580} = \underline{43.53}\ \text{sq. in.}$$

OK.

Also I

C. Plt, $= \tfrac{1}{2} \times \frac{24^3}{12} =$ ....576"^{4}

4∠s $= 4(3.6 + 3.25(7\ 5/8 + \tfrac{1}{2} + 1.06)^2) =$ 1112"^{4}

2 webs $= 2(20 \times \tfrac{1}{2}^3 + 10.00(7\ 5/8 + \tfrac{1}{4})^2)$ = 1241"^{4}

I total=2929"^{4}

$$r = \sqrt{\frac{2929}{45.00}} = 8.07$$

DESIGN OF TRUSS MEMBERS CONTINUED

DESIGNOF U_1L_0 continued.

$S = 16{,}000 - \frac{70 \times 38.29 \times 12}{8.07} = 12{,}020$#/sq. in.

Area $= \frac{504{,}100}{12{,}020} = 41.94$

OK.

Actual stress $= \frac{504{,}100}{45.00} = 11{,}200$#/sq. in.

Laticing:-

Double latticing.

$S = \frac{280Ar}{c}$

$S = \frac{280 \times 45.00 \times 8.07}{12} = 8475$#

$\frac{S}{2} = 4240$# carried by lattice.

Trial I

~~Double at 45° center/riveted.~~

~~$S = \frac{8475}{4} \times 2 = 3000$#/ bar.~~

~~Try 2½" bar~~

~~$t = \frac{20\ 5/8 \times 2}{60} = .472$ or ½" thick.~~

~~$S = \frac{3000}{2\frac{1}{2} \times \frac{1}{2}} = 2400$#/sq." R = .144~~

~~$S = 16000 - \frac{70 \times 29.2}{.144} = 2800$#/sq.in.~~

Trial II

Try 60° center riveted.

$S = \frac{8475 \times 1}{4 \times \sin 60^\circ} = 2447$#/bar.

Use $2\frac{1}{2}$"$\times\frac{1}{2}$" bars

$S = \frac{2447}{2\frac{1}{2} \times \frac{1}{2}} = 1757$#/sq. in.

$r = 0.144$

$S_{allowable} = 16{,}000 - \frac{70 \times 23.8}{.144} = 4430$#/sq. in.

OK.

DESIGN OF TRUSS MEMBERS CONTINUED

DESIGN OF U_1L_0 continued

$$Def. = \frac{sl}{E} = \frac{11,200 \times 11.9}{30,000,000} = .0004443$$

$$Def. = 23.81639 - \sqrt{20.625^2 + 11.90820^2} = .00052$$

$$s = \frac{.00053 \times 30,000,000}{23.81639} = 655\#/sq.in.$$

S total = 1957 + 655 = 2610#/sq. in.

Member as designed and use latticing at 60°

2½"x½" center riveted.
Inside clear distance between webs = 15¼"

DESIGN OF U_1L_1 & U_3L_3

Stress = 237000# tension.

$$\text{Req'd area} = \frac{237,000}{16,000} = 14.81 \text{ sq. in.}$$

Trial section:-

4∠s 3½"x3½"x3/8"	= 9.92 sq. in.
2 webs @20"x3/8"	= 15.00 " "
Gross area	24.92

-Rivets

4x1"x3/8" = 1.50 sq. in.

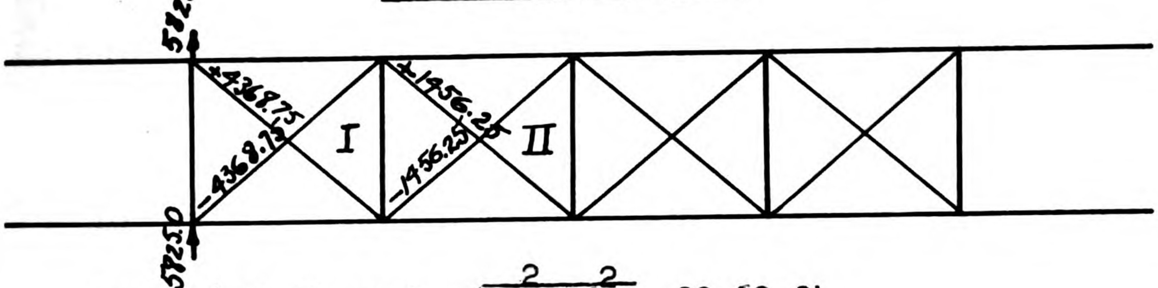
2x2x1"x3/8" = 1.50 sq. in.

Rivet holes = 3.00 " "

Net area = ...24.92-3.00=21.92 sq. in.

Ok.

TOP INDEX STRESSES



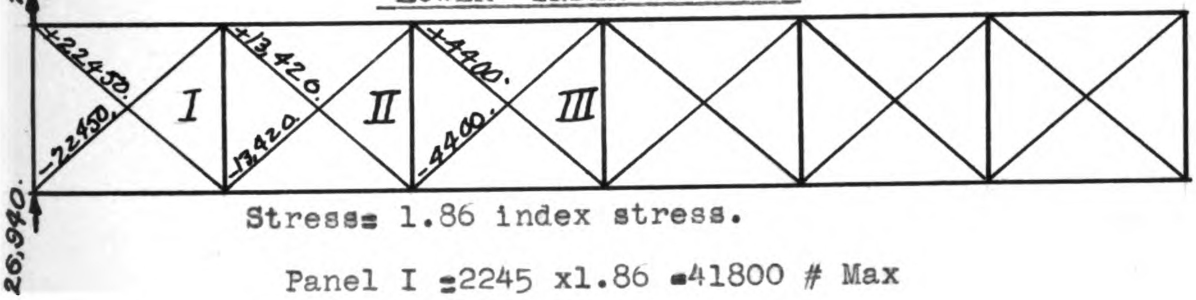
Length of diagnal= $\sqrt{25^2+16^2}$ =29.69 ft.

$\frac{29.69}{16}$ =1.86 x index stress = actual stress.

Stresspanel I = 4368.75 x1.86 = 8125 # Max.

" " II= 1456.25 x1.86 =2710 # Max.

LOWER INDEX STRESSES



Stress= 1.86 index stress.

Panel I =2245 x1.86 =41800 # Max

" II =13420 x1.86 =2500 # Max

" III= 4490 x1.86 =8350 # Max

TOP; Panel I.

$3\frac{1}{2}$"x $3\frac{1}{2}$"x3/8" ∠s area =2.48 sq. in.

$\frac{8125}{13500}$ = ;60 sq. in. needed.

16000-$\frac{70x15x12}{1.07}$ =4200 # allowable.

$\frac{8125}{4200}$ = 1.96 sq. in. needed.

Actual stress= $\frac{8125}{2.48}$ = 3270 #

No computations will be made for other panels.

BOTTOM

Panel I

Stress = 41800 #

Try 5"x5"x7/16" ∠s   Area = 4.18 sq. in.

r = 1.55

$16000 - \frac{70 \times 8.85 \times 12}{1.55} = 11390$ #/sq. in.

$\frac{41800}{11390} = 3.68$ sq. in needed.

Actual stress. $= \frac{41800}{418} = 10000$ #/sq. in.

Ok panel I

Panel II

Stress = 25000 #

Try 4"x4"x3/8" ∠s   area = 2.86 sq. in.

r = 1.23

$16000 - \frac{70 \times 8.85 \times 12}{1.23} = 9960$ #/sq. in. allowable.

$\frac{25000}{9960} = 2.485$ sq. in. needed.

$\frac{25000}{2.86} = 8750$ # actual stress.

Ok panel two.

Panel III

Stress = 8350 #

Try 3½"x3½"x3/8" ∠s   area = 2.48 sq. in.

r = 1.07

$16000 - \frac{70 \times 8.85 \times 12}{1.07} = 9050$ #/sq. in. allowable.

Continued next page.

DESIGN OF LATERAL BRACING CONTINUED

Panel III lower bracing.

$\frac{8350}{9050}$ = .924 sq. in. needed.

$\frac{8350}{2.48}$ = 33370 #/sq. in. actual stress.

Ok panel III

Note:

The design of the pedestals was considered unnecessary in a cost estimate.

DESIGN OF CONCRETE ABUTMENT

Assume bearing power of the soil p= 5,000#/ sq. ft.

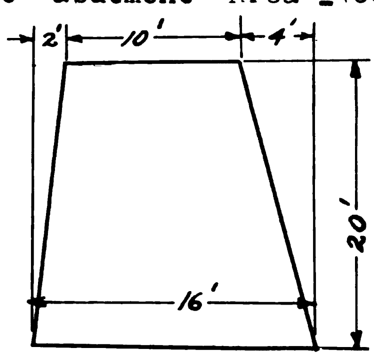
Max bridge load = 381,900x2 = 763,800#

Load from weight of abutment=20'x150= 3000#/sq. ft.

Available pressure p= 5000-3000= 2000#/ sq. ft.

$\frac{763,800}{2,000}$ = 391.9 sq. ft. needed.

Use a 25'x16' abutment Area =400sq. ft. base.



Volume concrete=$\dfrac{2\left(\frac{10+16}{2}\right)\times 20\times 25)}{27}$ =485 cu. yd. for both abutments

WEIGHT OF TRUSS FIGURES

U_1L_0

	wt./ft.		
1 C.Plt.(24"x½")x41'	40.8 =	1675	#
4 webs(20"x½") x41'	34.0	2790	#
4 ∠s(3½"x3½"x½")x41'	11.1	1820	#
		6285	# wt. of one.

4x6285= 25140# for like members.

U_1U_2 & U_2U_3

1 C. Plt.(24"x½")x24'	40.8 =	980	#
4 ∠s 3½"x3½"x5/8")x24'	13.6	1305	#
2 webs (20"x11/16")x24'	46.8	2246	#
		4531	# wt. of one.

4x2x4531= 36248 #for like members.

L_2L_3

4 ∠s (3½"x3½"x5/8")	x24'	13.6 =	1306	#
4 webs (21"x½")	x24	35.7	3428	#
			4734	# wt. of one.

4x4734= 18936 # for like members

L_1L_2 & L_0L_1

4 ∠s (3½"x3½"x9/16)	x24	12.4 =	1191	#
4 webs (21"x7/16")	x24	31.24	3000	#
			4191	#

2x4x4191 =33528 #for like members.

U_1L_1

2 webs (20"x3/8")	x30'	25.5 =	1530	#
4 ∠s (3½"x3½"x3/8")x30'		8.5 =	1020	#
			2550	#

6x2550= 15300 # for like members.

WEIGHT OF TRUSS FIGURES CONTINUED

U_1L_2 wt./ft.

4 ∠s (3½"x3½"x9/16")x34' 12.4 = 1685 #

2 webs (21"x7/16") x34' 31.24= 4250 #
5935 # per member
4x5935=23740#for like members.

U_2L_2

4 ∠s (3½"x3½"x3/8") x30' 8.5=102.0 #

1 web (13½x3/8") x30' 17.21=2065 #
3085 # per member

4 x3085 = 12340 # for like members.

L_2U_3

2 ∠s ~~[illegible]~~ 15"-45# x34' 3060 #per member

3060 x4=;2240# for like members.

Portal bracing per portal.
1 web 57"x3/8")x6' 72.7= 472.5#

2 webs (72"x3/8") x6' 91.8=1101.6#

2 ∠s (3½"x5"x3/8")x18' 10.4 382 #

4 " (5"x3½"x3/8") 4.2' 10.4 174.8#

2∠s (5"x3½"x3/8") 14' 10.4 297.0#

4 ∠s (5"x3½"x3/8")6.5' 10.4 270.5#
~~330~~
2698.4#
2x 2698.4=5396.8# for portals.

Lateral bracing for top.

4 ∠s (3½"x3½"x3/8") 13' 8.5 442 #

2 ∠s (3½"x3½"x3/8" 27' 8.5 459 #
901 #

4x901=3604 # for upper brac-
-ing.

WEIGHT OF TRUSS CONTINUED

Lateral bracing for lower cords.

Panel I wt./ft.

1 ∠ (5"x5"x7/16")x25'	14.3	=357.8 #
2 ∠s(5"x5"x7/16")x12'	14.3	=343.0 #
		700.8 #

2x700.8 =1401.6# for end panels.

Panel II

1 ∠ (4"x4"x3/8")x27.5'	9.8	=269.5 #
2 ∠s(4"x4"x3/8")x13.5'	9.8	=264.5 #
		534.0 #

2x534 =1068 # for second and fifth panels.

Panel III

1 ∠ (3½"x3½"x3/8")x27.5'	8.5	=233.9 #
2 ∠s(3½"x3½"x3/8")x13.5'	8.5	=229.5 #
		563.4 #

2x563.4=1126.8# forpanels three and four.

Sway bracing

2 ∠s (3½"x3½"x3/8")x14'		8.5	= 238#
4 " " " "	x4.5'	"	= 153 #
2 " " " "	x7.25'	"	= 123.2 #
4 " " " "	x3'	"	= 102 #
			616.2 #

3x616.2 = 1848.6# total weight of sway bracing.

WEIGHT OF TRUSS COMPLETED

Weight of rivet heads estimated at	13,020
Latticing of all members	12,632
Gusset plates	
20 - 6'x4'x$\frac{3}{4}$" = 122.4#/ft.	14,700
20 - 6'x2'x$\frac{3}{4}$" = 61.2#/ft.	7,350
End Plates taken at	5,000
Weight of floor system	74,257.5#
TOTAL WEIGHT OF STEEL IN BRIDGE	280,985.3#

The weight of ties and rails will add 400#/ ft. to the dead weight of trusses.

Load per foot of truss $= \frac{280985.3}{300} + 200 = 1{,}103.6$#

We assumed at the start of the design a load of 1,300#

OUR DESIGN IS SAFE.

COST SHEET

Pavement	5569 yd. @ $1.10/yd.	$ 6,116.00
Curb & Gutter	2500ft. $0.65/ft.	1,625.00
Catch Basins	2 @ $50.00	100.00
Pumping outfit (Installed)		195.00
Tile	700 ft.@ $0.42/ft.	294.00
Placing tile	700 ft. @ $0.50	350.00
Abutments		3,637.50
Excavation, Haul, etc.		22,538.50
Bridge		14,050.00
Cost of Material & Labor		$48,906.00
Engineering @ 5%		2,445.30
Total		$51,351.30
Contractor Profits @ 20%		10,270.26
Total cost		$61,621.56

TOTAL COST OF COMPLETED SEPARATION $61,621.56