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Design of a deck type, three chord, space frame railway bridge

Williams, John Paxton

Rensselaer Polytechnic Institute

DESIGN OF A DECK TYPE, THREE CHORD, SPACE FRAME RAILWAY BRIDGE

JOHN P. WILLIAMS AND
FOSTER M. LALOR, JR.

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Submitted to the Faculty of
Rensselaer Polytechnic Institute in
partial fulfillment of the require-
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May, 1948

Troy, New York

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TABLE OF CONTENTS

INTRODUCTION

Since all framed structures have length, breadth, and thickness, all frames actually are space structures. The designer is accustomed to study the stress analysis of a truss from a viewpoint of forces in a plane, but he must take into account the extension of the members into a third dimension when he is designing lacing bars, stay plates, and diaphragms. Portals, sway frames and lateral trusses are analyzed seperately as planar structures. There are however other structures where the entire analysis must be studied in three dimensions. Framed pedestals, towers with three or more legs, framed domes, and bridges having a common chord are examples of space frames. Their analysis requires a knowledge of space statics. The necessary computations are not particularly complicated, but they are more tedious than those involved in the analysis of planar structures.

Preliminary study and investigation of space frame bridges indicated that considerable savings in weight might be effected if economical joints could be designed. In spite of these possible savings, however, the use of space frame bridges in the United States has been negligible. This has been principally due to the relatively low cost of steel, the difficulty of fabrication, and the fact that American engineers, long accustomed to the design of standard planar structures, lack practical skill in space frame design and naturally resist any change. During the recent war when the shortage of steel

was critical, the Army Engineers turned to the three chord space frame bridge for rapid transportation and assembly in war zones. Space frame bridges have been used in Europe where the high cost of steel makes any savings in weight of much greater advantage. In the face of the steadily rising cost of steel and the increasing competition many American engineers are now turning to space frame structures.

From the first one particular advantage of a deck type three chord bridge was apparent. By using the top chords as stringers and designing them for combined stress it would be possible to develop a bridge with a more compact cross section than a plate girder and yet with such rigidity, lacking in the latter, that the three chord bridge could be completely assembled in the fabrication shop and shipped intact to the site. This permits not only such rapidity of assembly in the shop with considerable labor economy both there and at the site but also the more rapid transportation and replacement of such a bridge in an emergency.

The crux of the design of a three chord bridge is the absolute necessity of developing simple, easily fabricated joints. Poor joint design can cancel any possible savings by requiring the use of special sections merely to permit placing of sufficient rivets.

It was decided to design this bridge for the same span (85) loading(Cooper's E-72), and specifications (AREA Specifications for Steel Railroad Bridges) as the railroad plate girder bridge designed in the Bridge Analysis and Design course so that it would be possible to compare the two types.

The initial step in the design consisted of selecting the most advantageous cross section and the most advantageous type truss for a span of 85 feet.

In order to realize all of the advantages of a space frame bridge mentioned in the introduction, it was necessary to select the dimensions of the truss to give the most compact cross section compatible with specifications, member size, and joint design. Both for economy and for ease of transportation it was decided to use the top chords as stringers and to place them as close together as possible. The specifications limited this to 6.5 feet. Choice of the depth was more involved. The shallower the truss the greater the stresses and the more difficult the resulting connections; the greater the depth, the more difficult the transportation. After preliminary investigation in which tentative joints were drawn, stresses computed, and members chosen, it was decided that a depth of ten feet was the best compromise possible. At this depth stresses were reasonable, joint design possible, and transportation practical.

After consideration of the types of space frame trusses available for an 85 foot span, the following seemed to warrant consideration:

(1) Five Panel Howe

(2) Five Panel Pratt

(3) Six Panel Pratt

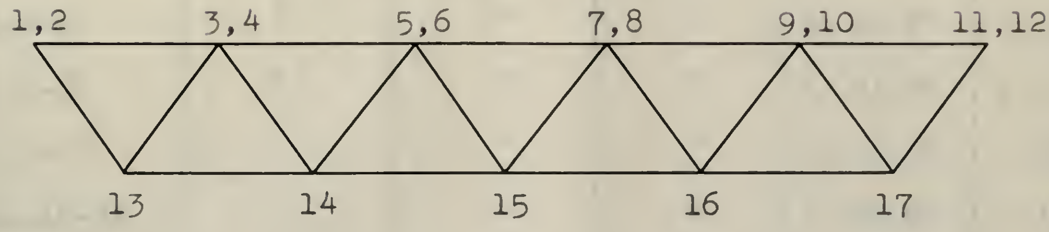
(4) Five Panel Warren

The selection of the truss to be used could best be made by a comparison of the influence lines of the members for each type. The method of solution is a variation of the tension

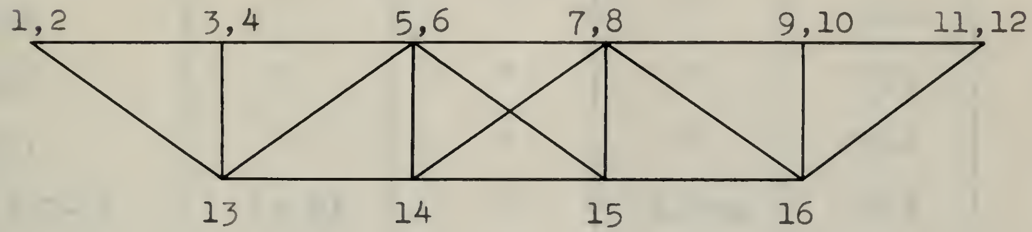
coefficient method proposed by Charles L. Hayen in "Simplified Solution of Space Frames, 1947"

SKETCHES OF SIDE ELEVATIONS OF TRUSSES INVESTIGATED

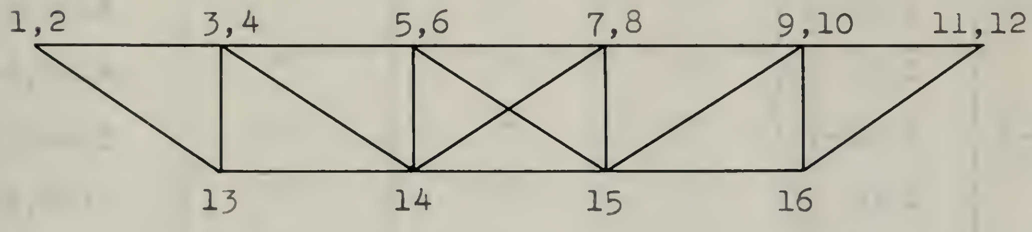
Five Panel Warren



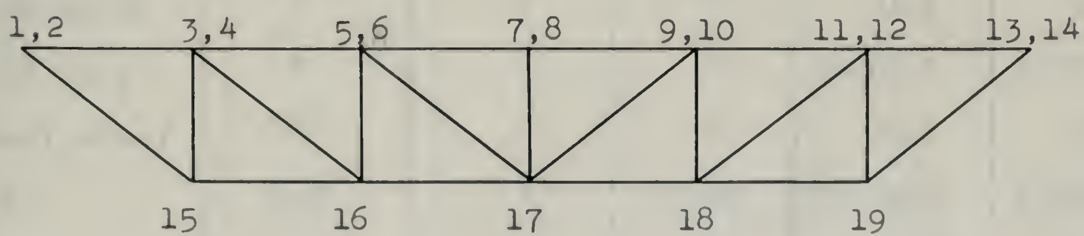
Five Panel Howe



Five Panel Pratt



Six Panel Pratt



INFLUENCE LINES

5 PANEL WARREN TYPE TRUSS   Load at 3-4

Member	Length	h	L/h	ht	Stress L/h ht
1-3,2-4	17.0	10	1.70	(-)0.2	(-)0.34
3-5,4-6	"	"	"	(-)0.35	(-)0.595
5-7,6-8	"	"	"	(-)0.25	(-)0.425
7-9,8-10	"	"	"	(-)0.15	(-)0.255
9-11,10-12	"	"	"	(-)0.15	(-)0.085
13-14	"	"	"	0.8	1.359
14-15	"	"	"	0.6	1.019
15-16	"	"	"	0.4	0.68
16-17	"	"	"	0.2	0.34
1-13,2-13	13.51	"	1.351	0.4	0.541
3-13,4-13	"	"	"	(-)0.4	(-)0.541
3-14,4-14	"	"	"	(-)0.1	(-)0.135
5-14,6-14	"	"	"	0.1	0.135
5-15,6-15	"	"	"	(-)0.1	(-)0.135
7-15,8-15	"	"	"	0.1	0.135
7-16,8-16	"	"	"	(-)0.1	(-)0.135
9-16,10-16	"	"	"	0.1	0.135
9-17,10-17	"	"	"	(-)0.1	(-)0.135
11-17,12-17	"	"	"	0.1	0.135
1-2	6.5	"	0.65	(-)0.2	(-)0.13
3-4	"	"	"	0.25	0.163
5-6,7-8,9-10	"	"	"	0.0	0.0
11-12	"	"	"	(-)0.05	(-)0.0325

INFLUENCE LINES

5 PANEL WARREN TYPE TRUSS   Load at 5-6

Member	Length	h	L/h	ht	Stress L/h ht
1-3,2-4	17.00	10.0	1.70	(-)0.15	(-)0.255
3-5,4-6	"	"	"	(-)0.45	(-)0.765
5-7,6-8	"	"	"	(-)0.50	(-)0.85
7-9,8-10	"	"	"	(-)0.30	(-)0.51
9-11,10-12	"	"	"	(-)0.10	(-)0.17
13-14	"	"	"	0.60	1.019
14-15	"	"	"	1.20	2.04
15-16	"	"	"	0.80	1.358
16-17	"	"	"	0.40	0.68
1-13,2-13	13.51	"	1.351	0.30	0.405
3-13,4-13	"	"	"	(-)0.30	(-)0.405
3-14,4-14	"	"	"	0.30	0.405
5-14,6-14	"	"	"	(-)0.30	(-)0.405
5-15,6-15	"	"	"	(-)0.20	(-)0.27
7-15,8-15	"	"	"	0.20	0.27
7-16,8-16	"	"	"	(-)0.2	(-)0.27
9-16,10-16	"	"	"	0.20	0.27
9-17,10-17	"	"	"	(-)0.20	(-)0.27
11-17,12-17	"	"	"	0.20	0.27
1-2	6.5	"	0.65	(-)0.15	(-)0.0974
3-4,7-8,9-10	"	"	"	0.0	0.0
5-6	"	"	"	0.25	0.163
11-12	"	"	"	(-)0.1	(-)0.065

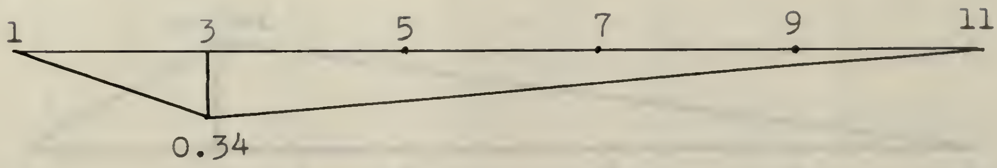
INFLUENCE LINES

5 PANEL WARREN TYPE TRUSS

Chords

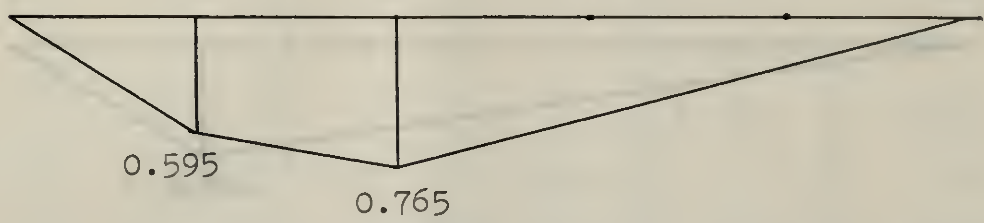
1-3

2-4



3-5

4-6



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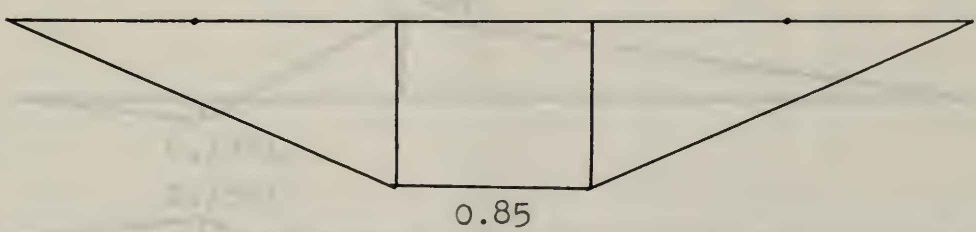
6-8



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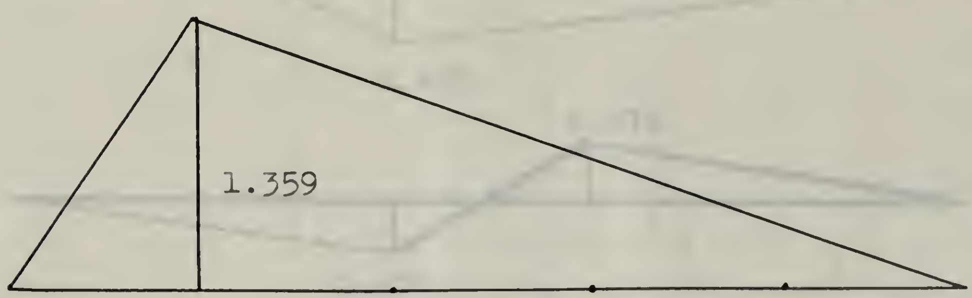

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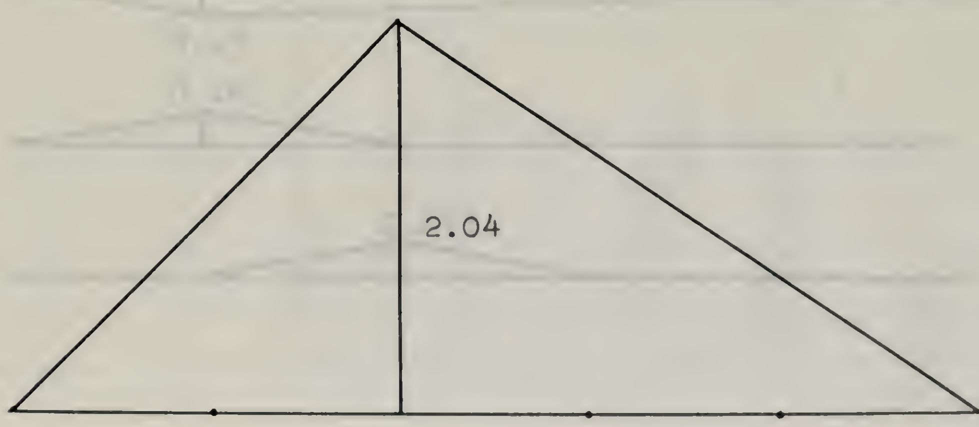
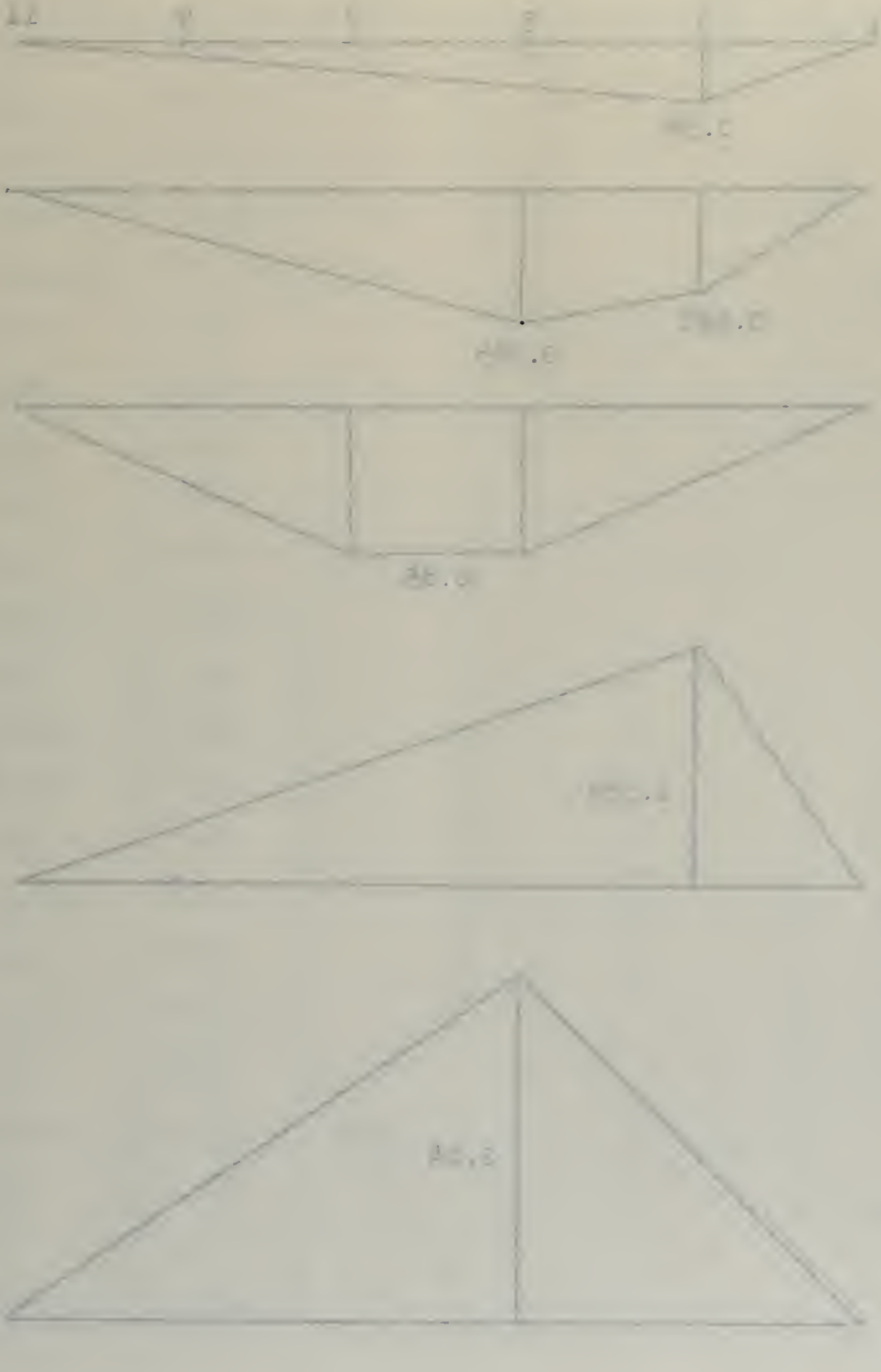
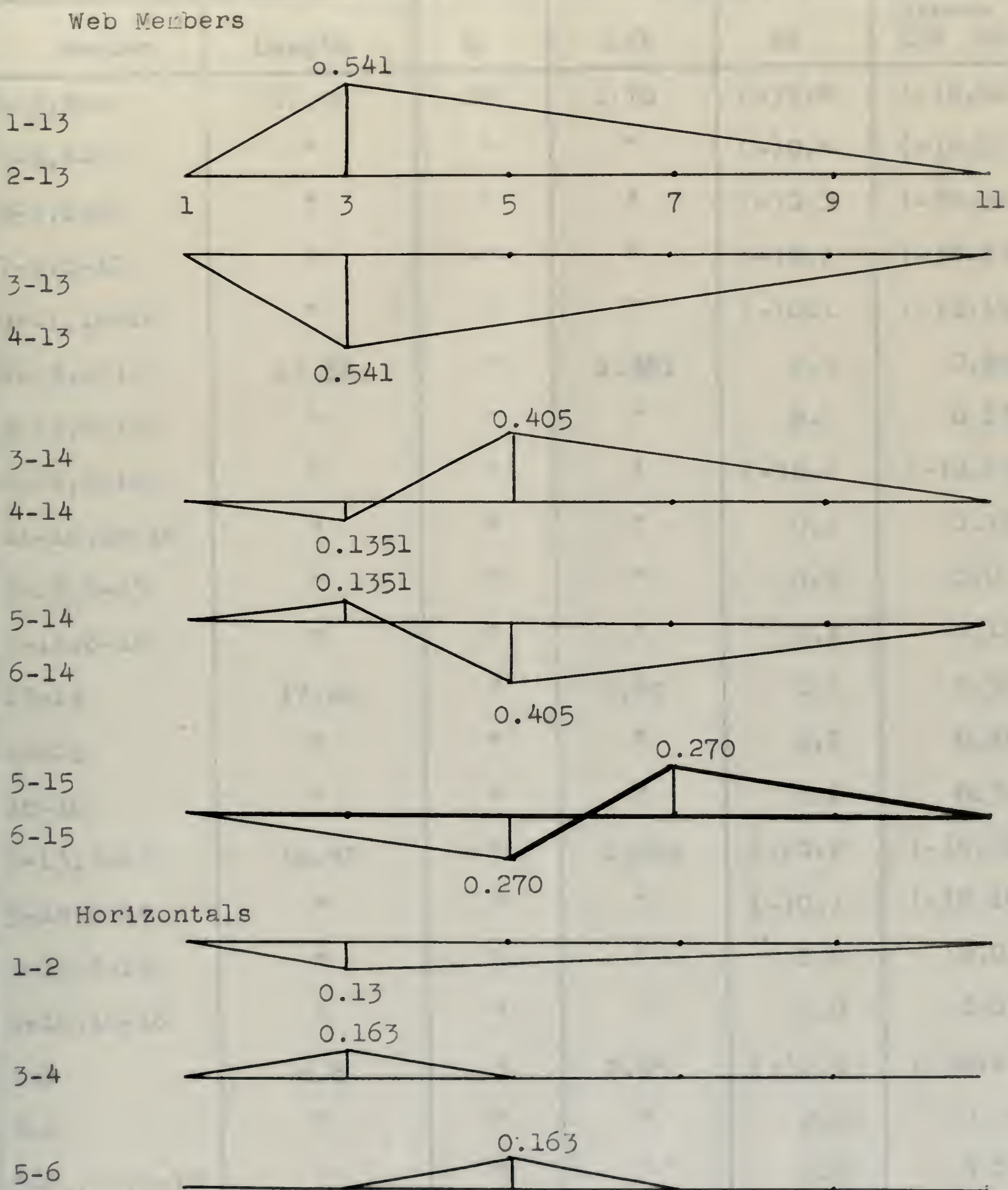




INFLUENCE LINES

5 PANEL WARREN TYPE TRUSS



INFLUENCE LINES

5 PANEL HOWE TYPE TRUSS   Load at 3-4

Member	Length	h	L/h	ht	Stress L/h ht
1-3,2-4	17.00	10	1.70	(-)0.4	(-)0.68
3-5,4-6	"	"	"	(-)0.4	(-)0.68
5-7,6-8	"	"	"	(-)0.3	(-)0.51
7-9,8-10	"	"	"	(-)0.1	(-)0.17
9-11,10-12	"	"	"	(-)0.1	(-)0.17
1-13,2-13	13.51	"	1.351	0.4	0.541
5-13,6-13	"	"	"	0.1	0.135
7-16,8-16	"	"	"	(-)0.1	(-)0.135
11-16,12-16	"	"	"	0.1	0.135
5-15,6-15	"	"	"	0.0	0.0
7-14,8-14	"	"	"	0.1	0.135
13-14	17.00	"	1.70	0.3	0.51
14-15	"	"	"	0.2	0.34
15-16	"	"	"	0.2	0.34
3-13,4-13	10.52	"	1.052	(-)0.2	(-)0.526
5-14,6-14	"	"	"	(-)0.1	(-)0.105
7-15,8-15	"	"	"	0.0	0.0
9-16,10-16	"	"	"	0.0	0.0
1-2	6.5	"	0.65	(-)0.2	(-)0.13
3-4	"	"	"	0.25	0.1625
5-6,7-8,9-10	"	"	"	0.0	0.0
11-12	"	"	"	(-)0.05	(-)0.0325

INFLUENCE LINES

5 PANEL HOWE TYPE TRUSS   Load at 5-6

Member	Length	h	L/h	ht	Stress L/h ht
1-3,2-4	17.00	10	1.70	(-)0.3	(-)0.51
3-5,4-6	"	"	"	(-)0.3	(-)0.51
5-7,6-8	"	"	"	(-)0.6	(-)1.02
7-9,8-10	"	"	"	(-)0.2	(-)0.34
9-11,10-12	"	"	"	(-)0.2	(-)0.34
1-13,2-13	13.51	"	1.351	0.3	0.406
13-5,13-6	"	"	"	(-)0.3	(-)0.406
7-16,8-16	"	"	"	(-)0.2	(-)0.270
11-16,12-16	"	"	"	0.2	0.270
5-15,6-15	"	"	"	-----	-----
7-14,8-14	"	"	"	0.2	0.270
13-14	17.00	"	1.70	0.6	1.02
14-15	"	"	"	0.4	0.68
15-16	"	"	"	0.4	0.68
3-13,4-13	10.52	"	1.052	0	-----
5-14,6-14	"	"	"	(-)0.2	(-)0.210
7-15,8-15	"	"	"	0	-----
9-16,10-16	"	"	"	0	-----
1-2	6.5	"	0.65	(-)0.15	(-)0.0975
3-4	"	"	"	0	-----
5-6	"	"	"	0.25	0.1626
7-8	"	"	"	0	-----
9-10	"	"	"	0	-----
11-12	"	"	"	(-)0.1	(-)0.065

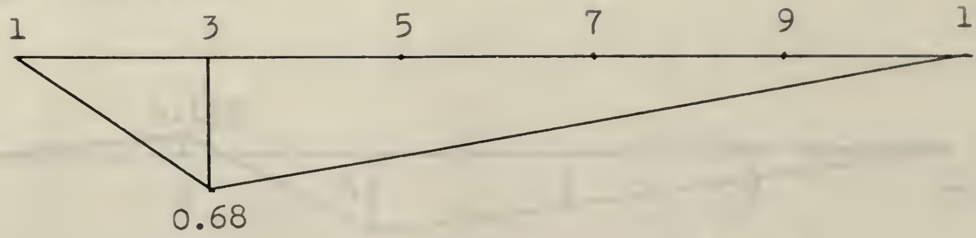
INFLUENCE LINES

5 PANEL HOWE TYPE TRUSS

Chords

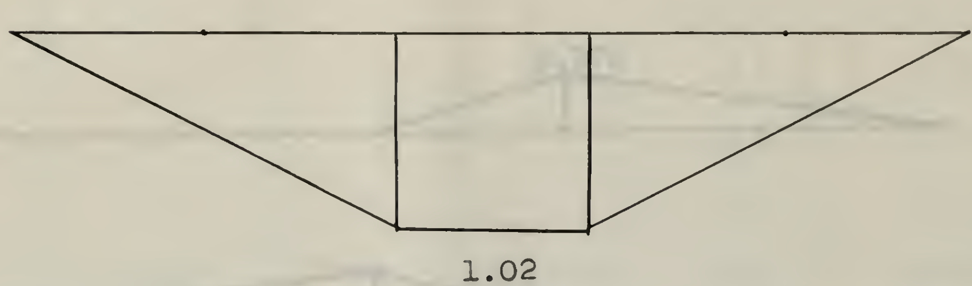
1-3, 2-4

3-5, 4-6



5-7

6-8



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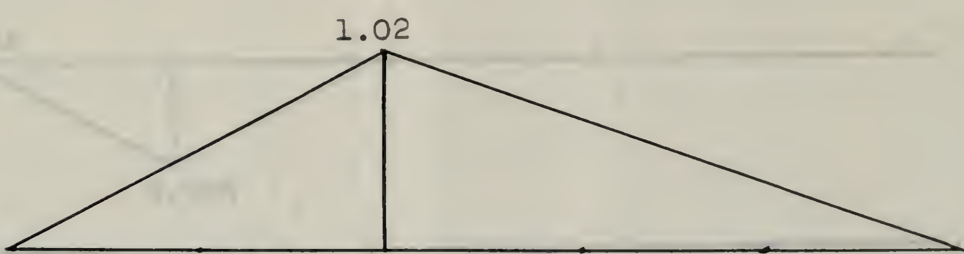

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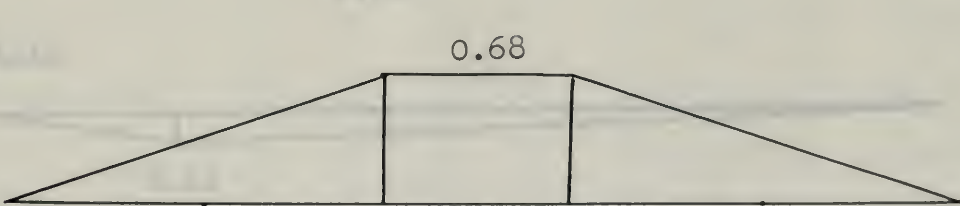
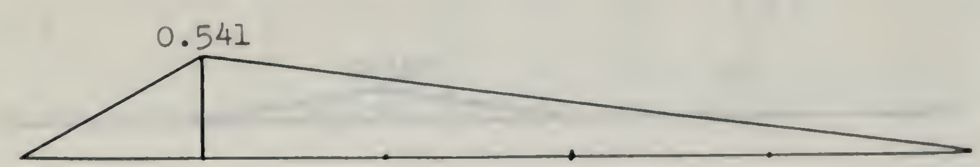


Diagonals

1-13

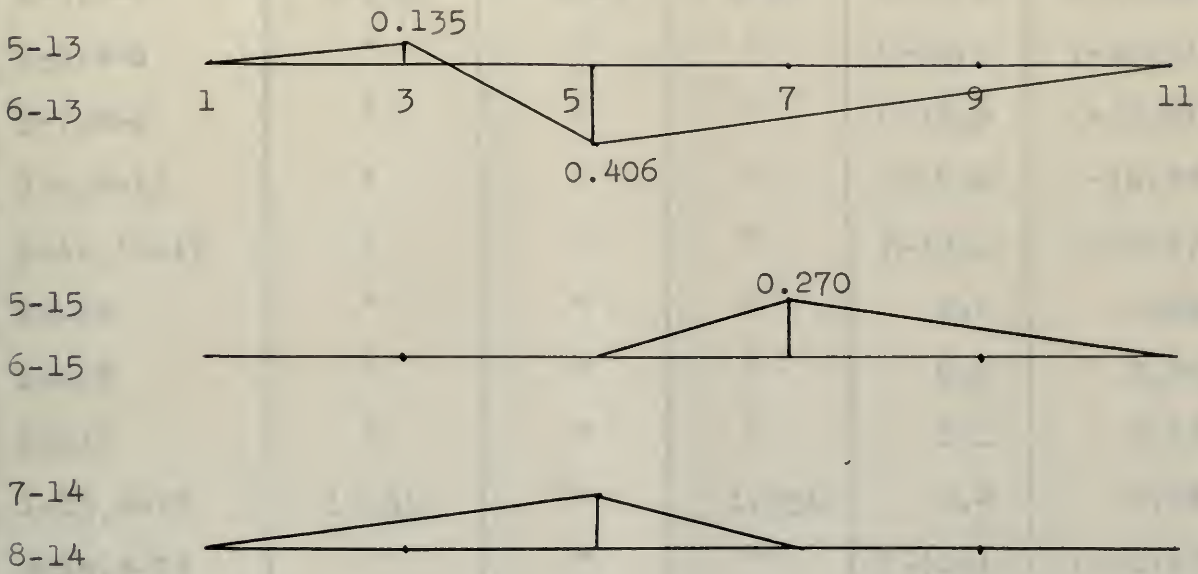
2-13



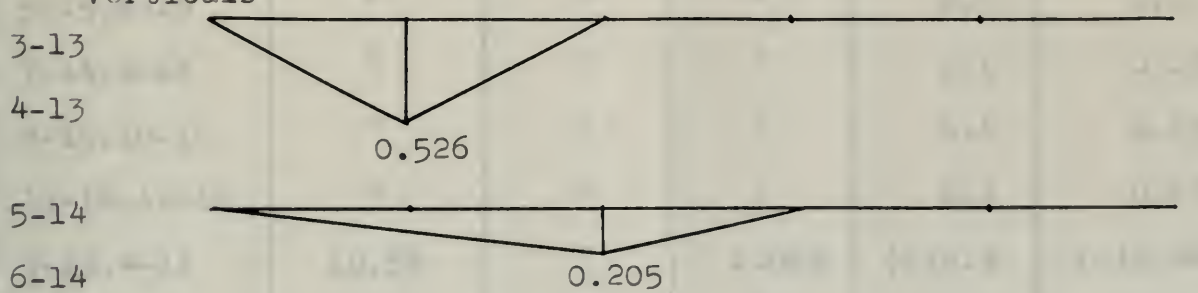
INFLUENCE LINES

5 PANEL HOWE TYPE TRUSS

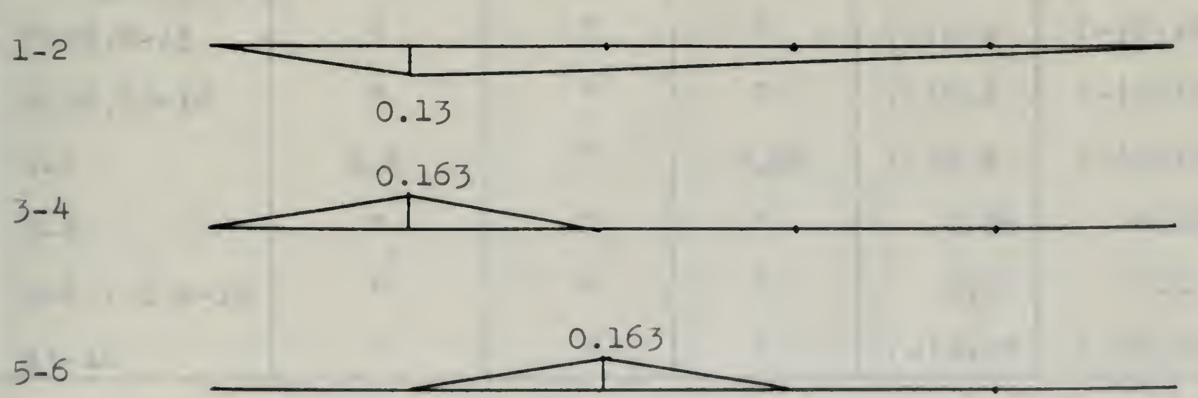
Diagonals (continued)



Verticals



Horizontals



INFLUENCE LINES

5 PANEL PRATT TYPE TRUSS   Load at 3-4

Member	Length	h	L/h	ht	Stress L/h ht
1-3,2-4	17.00	10.0	1.70	(-)0.4	(-)0.68
3-5,4-6	"	"	"	(-)0.3	(-)0.51
5-7,6-8	"	"	"	(-)0.3	(-)0.51
7-9,8-10	"	"	"	(-)0.2	(-)0.34
9-11,10-12	"	"	"	(-)0.1	(-)0.17
13-14	"	"	"	0.4	0.68
14-15	"	"	"	0.2	0.34
15-16	"	"	"	0.1	0.17
1-13,2-13	13.51	"	1.351	0.4	0.54
3-14,4-14	"	"	"	(-)0.1	(-)0.135
5-15,6-15	"	"	"	0.0	0.0
7-14,8-14	"	"	"	0.1	0.135
9-15,10-15	"	"	"	0.1	0.135
11-16,12-16	"	"	"	0.1	0.135
3-13,4-13	10.52	"	1.052	(-)0.4	(-)0.421
5-14,6-14	"	"	"	0.0	0.0
7-15,8-15	"	"	"	(-)0.1	(-)0.105
9-16,10-16	"	"	"	(-)0.1	(-)0.105
1-2	6.5	"	0.65	(-)0.2	(-)0.13
3-4	"	"	"	0.25	0.163
5-6,7-8,9-10	"	"	"	0.0	0.0
11-12	"	"	"	(-)0.05	(-)0.033

INFLUENCE LINES

5 PANEL PRATT TYPE TRUSS   Load at 5-6

Member	Length	h	L/h	ht	Stress L/h ht
1-3,2-4	17.00	10	1.70	(-)0.3	(-)0.51
3-5,4-6	"	"	"	(-)0.6	(-)1.02
5-7,6-8	"	"	"	(-)0.6	(-)1.02
7-9,8-10	"	"	"	(-)0.4	(-)0.68
9-11,10-12	"	"	"	(-)0.2	(-)0.34
13-14	"	"	"	0.3	0.51
14-15	"	"	"	0.4	0.68
15-16	"	"	"	0.2	0.34
1-13,2-13	13.51	"	1.351	0.3	0.405
3-14,4-14	"	"	"	0.3	0.405
5-15,6-15	"	"	"	0.0	0.0
7-14,8-14	"	"	"	0.2	0.27
9-15,10-15	"	"	"	0.2	0.27
11-16,12-16	"	"	"	0.2	0.27
3-13,4-13	10.52	"	1.052	(-)0.3	(-)0.316
5-14,6-14	"	"	"	(-)0.5	(-)0.526
7-15,8-15	"	"	"	(-)0.2	(-)0.21
9-16,10-16	"	"	"	(-)0.2	(-)0.21
1-2	6.5	"	0.65	(-)0.15	(-)0.0975
3-4,7-8,9-10	"	"	"	0.0	0.0
5-6	"	"	"	0.25	0.163
11-12	"	"	"	(-)0.1	(-)0.065

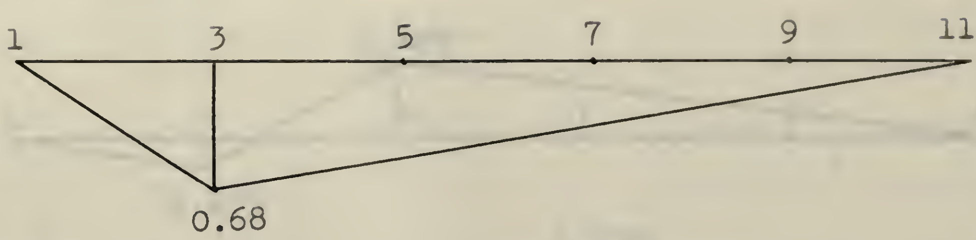
INFLUENCE LINES

5 PANEL PRATT TYPE TRUSS

Chords

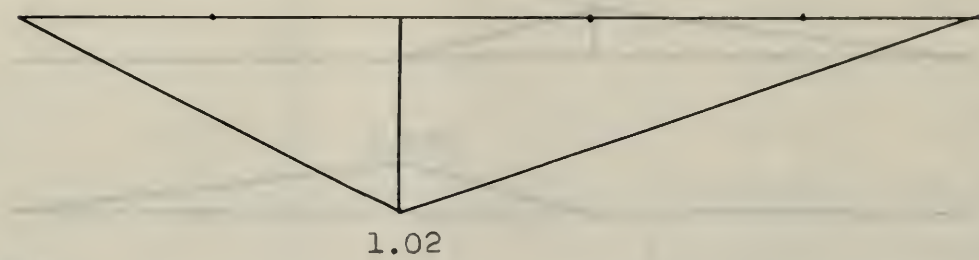
1-3

2-4



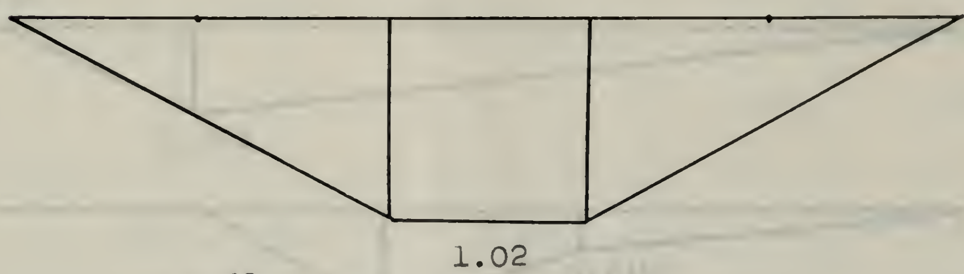
3-5

4-6



5-7

6-8



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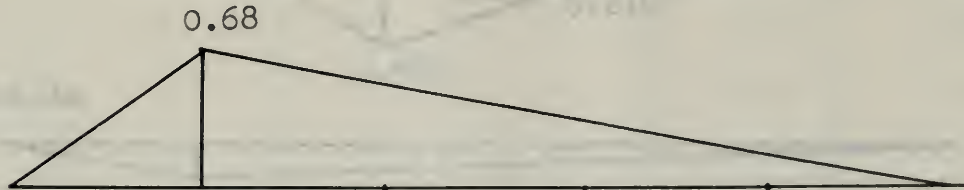

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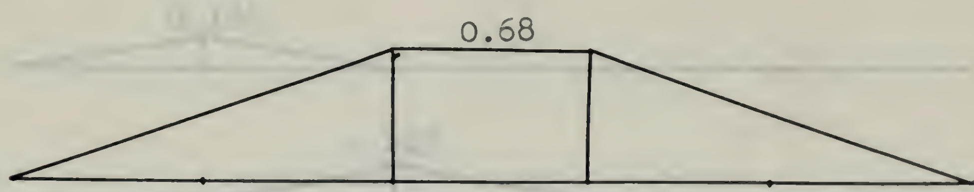
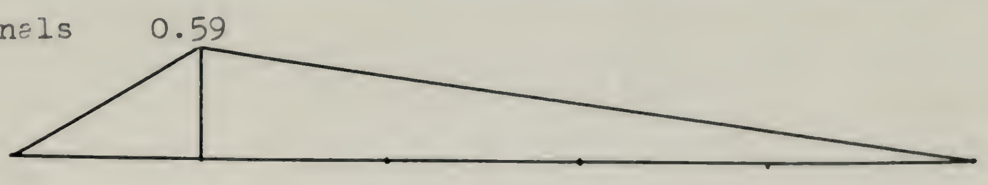


Diagonals

1-13

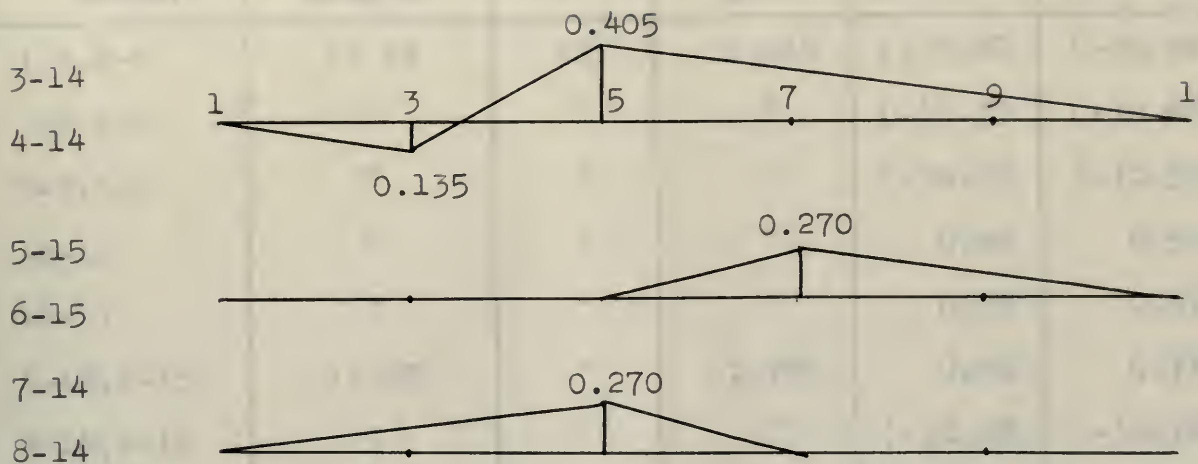
2-13



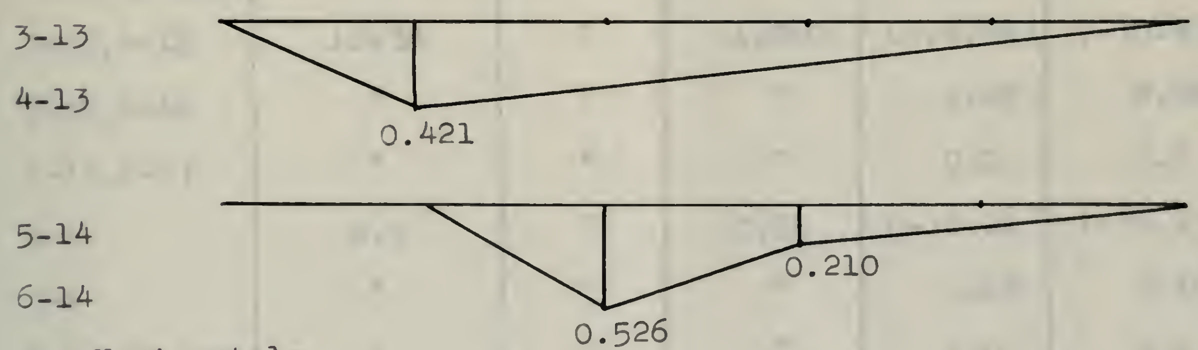
INFLUENCE LINES

5 PANEL PRATT TYPE TRUSS

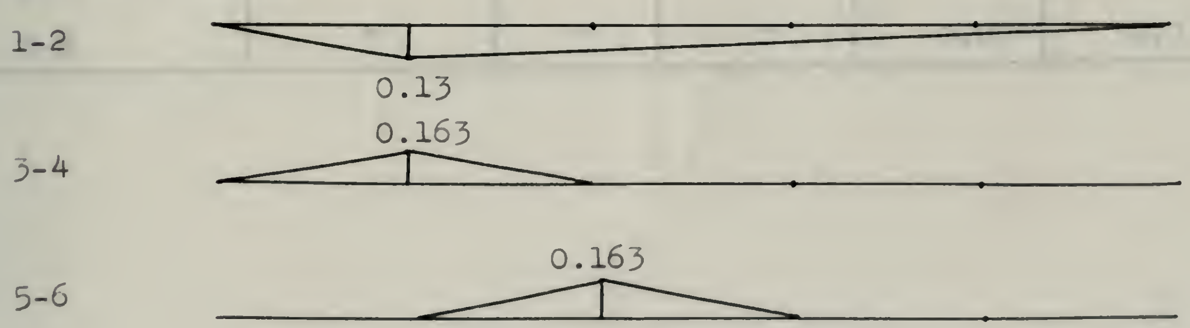
Diagonals (continued)



Verticals



Horizontals



INFLUENCE LINES

6 PANEL PRATT TYPE TRUSS   Load at 3-4

Member	Length	h	L/h	ht	Stress L/h ht
1-3,2-4	14.18	10	1.418	(-)0.42	(-)0.59
3-5,4-6	"	"	"	(-)0.33	(-)0.472
5-7,6-8	"	"	"	(-)0.25	(-)0.354
15-16	"	"	"	0.42	0.59
16-17	"	"	"	0.33	0.472
1-15,2-15	17.65	"	1.765	0.42	0.735
3-16,4-16	"	"	"	(-)0.08	(-)0.147
5-17,6-17	"	"	"	(-)0.08	(-)0.147
3-15,4-15	10.50	"	1.050	(-)0.42	(-)0.438
5-16,6-16	"	"	"	0.08	0.088
7-17,8-17	"	"	"	0.0	0.0
1-2	6.5	"	0.65	(-)0.21	(-)0.136
3-4	"	"	"	0.25	0.163
5-6	"	"	"	0.0	0.0
7-8	"	"	"	0.0	0.0

INFLUENCE LINES

6 PANEL PRATT TYPE TRUSS   Load at 5-6

Member	Length	h	L/h	ht	Stress L/h ht
1-3,2-4	14.18	10.0	1.418	(-)0.33	(-)0.472
3-5,4-6	"	"	"	(-)0.67	(-)0.945
5-7,6-8	"	"	"	(-)0.5	(-)0.709
15-16	"	"	"	0.33	0.472
16-17	"	"	"	0.67	0.945
1-15,2-15	17.65	"	1.765	0.33	0.588
3-16,4-16	"	"	"	0.33	0.588
5-17,6-17	"	"	"	(-)0.17	(-)0.294
3-15,4-15	10.50	"	1.050	(-)0.33	(-)0.35
5-16,6-16	"	"	"	(-)0.33	(-)0.35
7-17,8-17	"	"	"	0.0	0.0
1-2	6.5	"	0.65	(-)0.17	(-)0.1083
3-4	"	"	"	0.0	0.0
5-6	"	"	"	0.25	0.163
7-8	"	"	"	0.0	0.0

INFLUENCE LINES

6 Panel Pratt Type Truss   Load at 7-8

Member	Length	h	L/h	ht	Stress L/h ht
1-3,2-4	14.18	10.0	1.418	(-)0.25	(-)0.355
3-5,4-6	"	"	"	(-)0.50	(-)0.709
5-7,6-8	"	"	"	(-)0.75	(-)1.064
15-16	"	"	"	0.25	0.355
16-17	"	"	"	0.50	0.709
1-15,2-15	17.65	"	1.765	0.25	0.441
3-16,4-16	"	"	"	0.25	0.441
5-17,6-17	"	"	"	0.25	0.441
3-15,4-15	10.52	"	1.052	(-)0.25	(-)0.263
5-16,6-16	"	"	"	(-)0.25	(-)0.263
7-17,8-17	"	"	"	(-)0.50	(-)0.526
1-2	6.50	"	0.65	(-)0.125	(-)0.0813
3-4	"	"	"	0.0	0.0
5-6	"	"	"	0.0	0.0
7-8	"	"	"	0.25	0.163

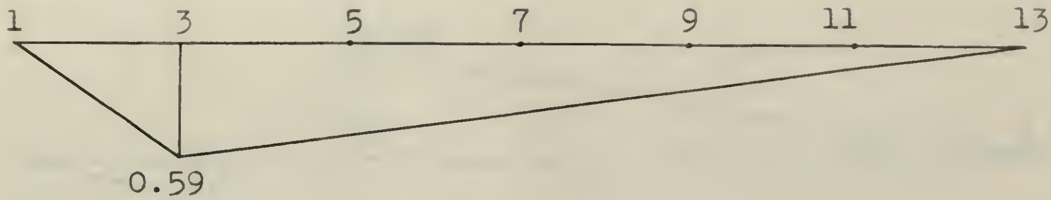
INFLUENCE LINES

6 PANEL PRATT TYPE TRUSS

Chords

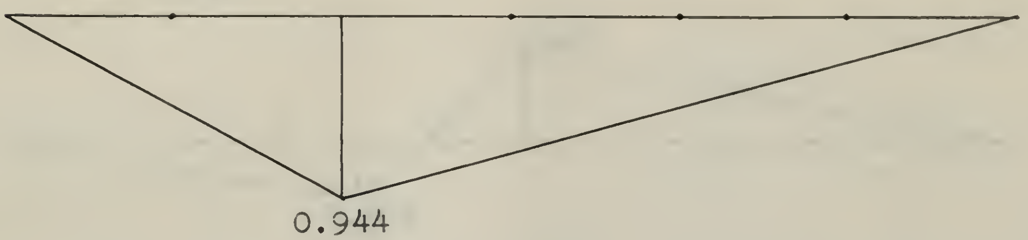
1-3

2-4



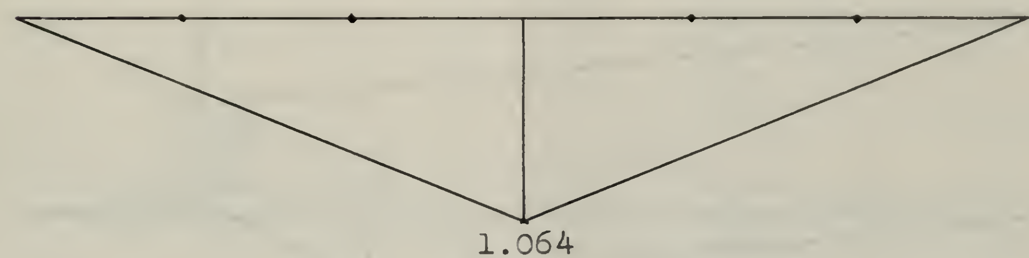
3-5

4-6

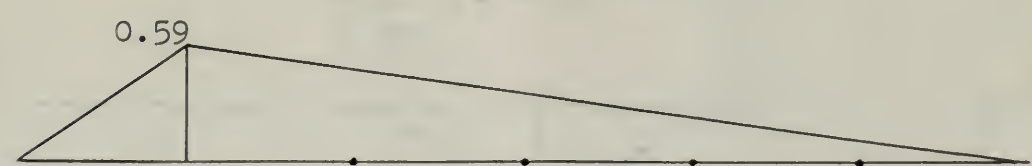


5-7

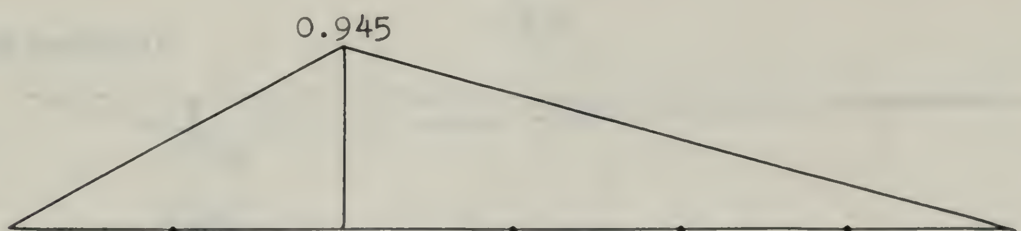
6-8



15-16



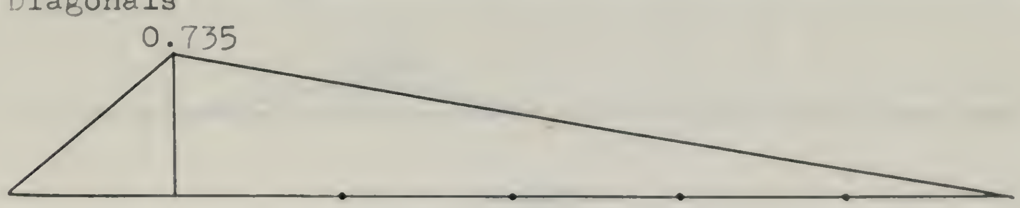
16-17



Diagonals

1-15

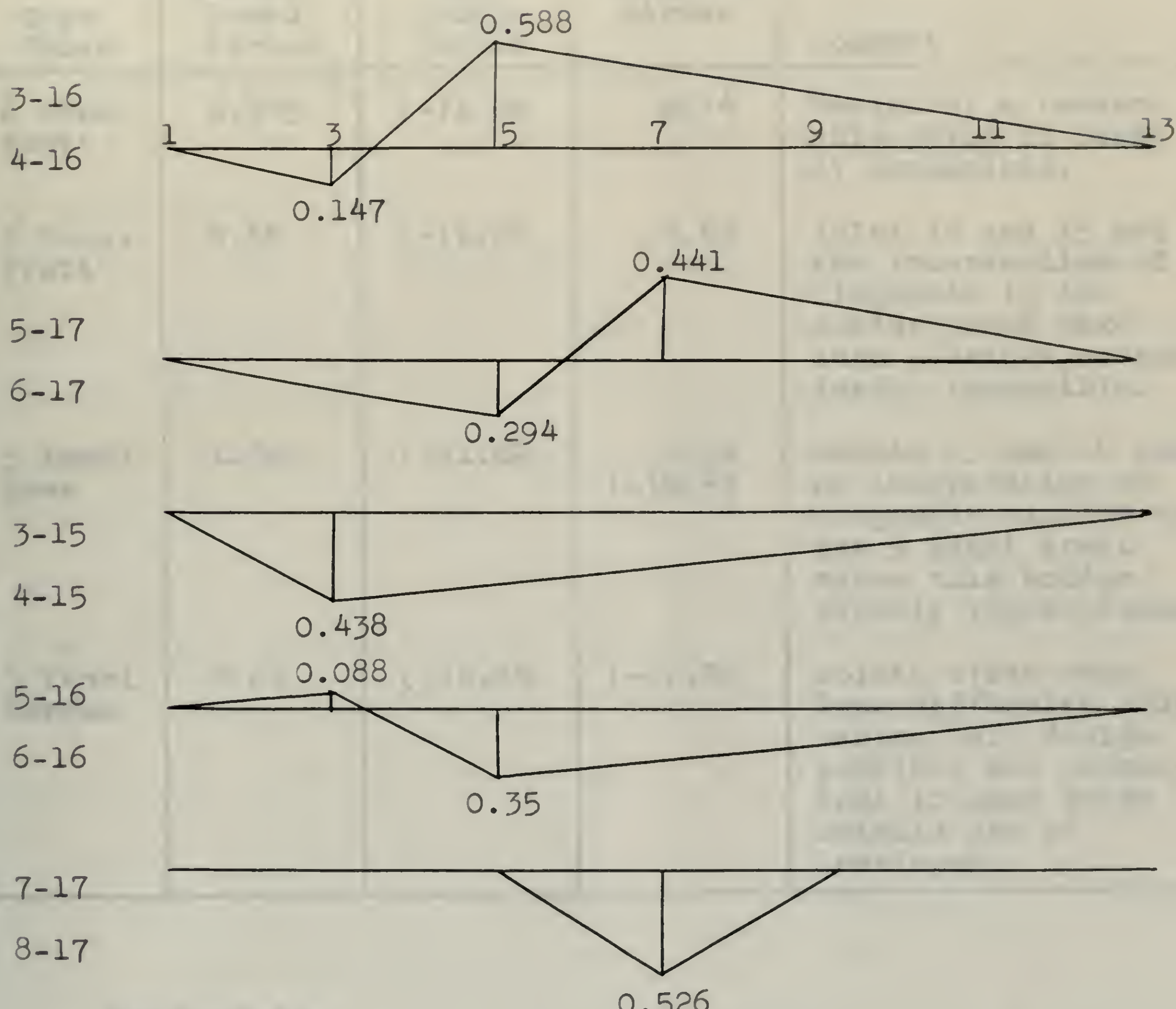
2-15



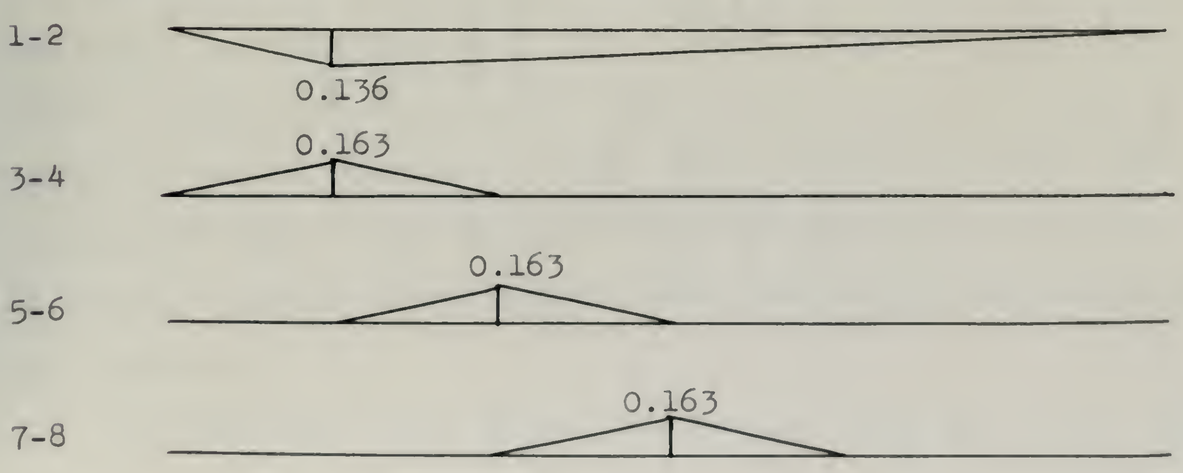
INFLUFNCE LINES

6 PANEL PRATT TYPE TRUSS

Diagonals (continued)



Horizontals



SUMMARY OF INFLUENCE LINE DATA

Type Truss	Maximum Bottom Chord Stress	Maximum Top Chord Stress	Maximum Diagonal Stress	Comment
6 Panel Pratt	0.945	(-)1.06	0.74	Design of a reasonable joint at point 17 impossible.
5 Panel Pratt	0.68	(-)1.02	0.54	Joint 14 and 15 and the intersection of diagonals in the center panel make such a bridge economically impossible.
5 Panel Howe	1.02	(-)1.02	0.54 (-)0.53	Joints 13 and 16 and an intersection of diagonals similar to the 5 Panel Pratt makes this bridge equally impracticable.
5 Panel Warren	2.04	(-)0.85	(-)0.54	Joints offer much less difficulty thus making this design possible and economical if good joint details can be developed.

A study of the preceding tabular condensation of the influence lines and results of investigation of the joint problem for each bridge showed that although the Warren has the highest stresses of the four it is the only bridge in which satisfactory joints can be developed. Experimentation showed that only by the use of eccentric joints could any of the Howe or Pratt types be used.

The apparently larger stress in the Warren truss are deceiving. Although its bottom chord stress is about twice the a average of the other three, its top chord stress is about 18 per cent less, and as the length of the two top chords is 2½ times that of the single bottom chord, the final analysis shows the Warren onlt slightly larger in stresses than the others. By a summation of stress times length for all members of each bridge, the following relative factors were obtained:

5 Panel Warren Bridge	1.000
5 Panel Howe Bridge	0.857
6 Panel Pratt Bridge	0.828
5 Panel Pratt Bridge	0.777

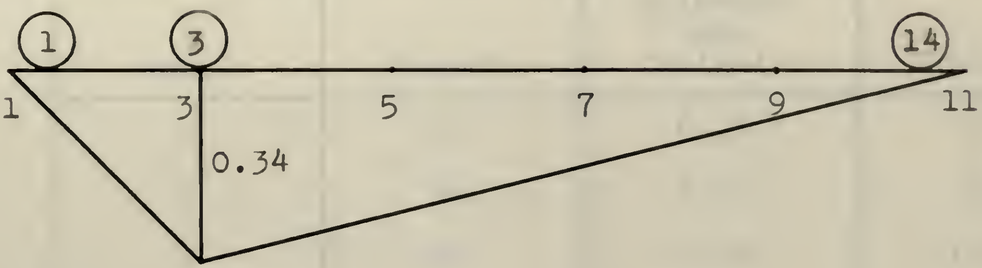
The economical if not structural impossibility of developing joints for the Howe and Pratt bridges, however, made the Warren bridge the only possible type to use. The fact that its apparent lack of economy is only slight has been stressed only to indicate that if the Howe or Pratt bridges were used, the inevitably excessive fabrication costs would make the latter much more expensive.

Following are the computations for the stresses in all members:

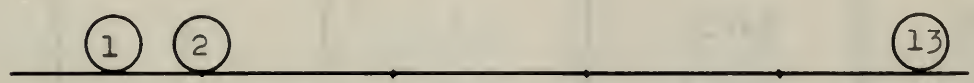
WARREN TYPE SPACE FRAME RAILWAY BRIDGE

CALCULATIONS FOR LIVE LOAD STRESSES: E-72 LOADING

Members 1-3 and 2-4



Position I: Wheel "2" at point 3

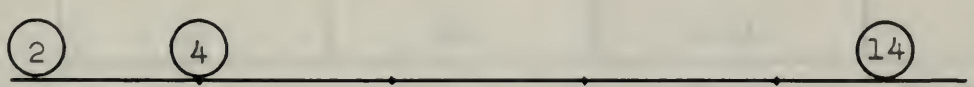


Position II: Wheel "3" at point 3

Loss = 45(5)(0.02) = 4.5

Gain = 303(5)(0.0005) = 7.58

Position III: Wheel "4" at point 3



Loss = 75(5)(0.02) = 7.5

Gain = 273(5)(0.005) = 6.82

Therefore use Position II

LIVE LOAD STRESSES

Member 1-3 and 2-4

Wheel	Weight	Influence Line Ordinate	Stress
1	15	0.08	1.2
2	30	0.24	7.2
3	30	0.34	10.2
4	30	0.314	9.4
5	30	0.291	8.7
6	19.5	0.245	4.8
7	19.5	0.221	4.3
8	19.5	0.19	3.7
9	19.5	0.165	3.2
10	15	0.125	1.9
11	30	0.85	2.5
12	30	0.60	1.8
13	30	0.35	1.0
14	30	0.01	0.3
		Sum	= 60.2

Stress = 2(60.2)(1.2) = (-)144.5K.

Equivalent Uniform Load* = 8340 lbs./ft. of track

Stress = 0.5(0.34)(85)(8.34)(1.2) = (-)144.5 K.

*Steinman's E-60 Chart-Transactions ASCE Vol.LXXXVI

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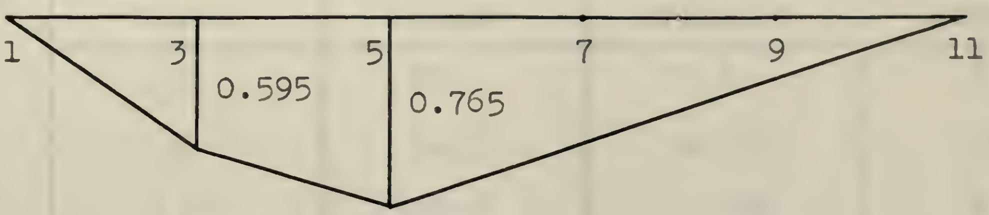
LIVE LOAD STRESSES

Members 3-5 and 4-6



Position I: Wheel "12" reversed at 5

Position II: Wheel "13" reversed at 5

Gain = 58.5(0.035)(5) + 19.5(0.01)(5) = 14.2

Loss = 213(0.015)(5) = 16.0

Position III: Wheel "14" reversed at 5

Gain = 39(0.035)(5) + 69(0.01)(5) = 10.3

Loss = 213(0.015)(5) = 16.0

Therefore use Position **II**

LIVE LOAD STRESSES

Members 3-5 and 4-6

Wheel	Weight	Influence Line Ordinate	Stress
5	30	0.0	0
6	19.5	0.136	2.7
7	19.5	0.210	4.1
8	19.5	0.300	5.9
9	19.5	0.376	7.3
10	15	0.496	6.8
11	30	0.616	18.5
12	30	0.690	20.8
13	30	0.765	22.9
14	30	0.715	21.5
15	19.5	0.625	12.2
16	19.5	0.525	10.4
17	19.5	0.315	6.2
18	19.5	0.140	2.7
		Sum	= 142.0

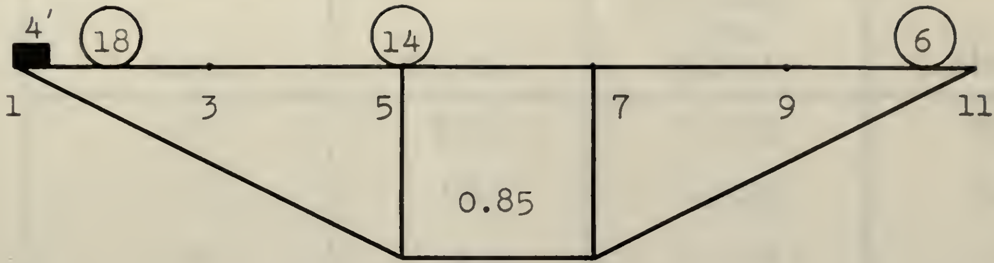
Stress = (-)2(142.0)(1.2) = (-)341 K.

Uniform Load = 8030 lbs./ft. of track

$$\text{Stress} = 1.2\left[(-)0.5(17)(8.03)(0.595) - 0.5(51)(0.765)(8.03) - 0.5(0.595 + 0.765)(8.03)(17)\right] = (-)347 \text{ K.}$$

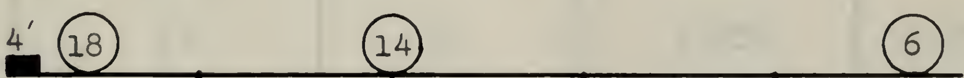
LIVE LOAD STRESSES

Members 5-7 and 6-8



Position I: Wheel "13" reversed at 5

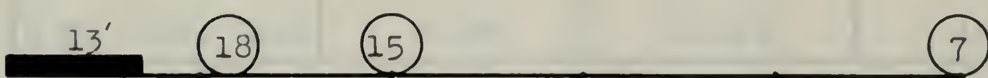
Position II: Wheel "14" reversed at 5



$$\text{Gain} = 108(0.025)(5) = 13.5$$

$$\text{Loss} = 93(0.025)(5) = 11.6$$

Position III: Wheel "15" reversed at 5



$$\text{Gain} = 78(0.025)(9) + 0.5(9)^2(0.025)(3) = 20.6$$

$$\text{Loss} = 153(0.025)(9) = 34.5$$

Therefore use Position II

LIVE LOAD STRESSES

Members 5-7 and 6-8

Wheel	Weight	Influence Line Ordinate	Stress
6	19.5	0.100	2.0
7	19.5	0.225	4.4
8	19.5	0.375	7.3
9	19.5	0.500	9.7
10	15	0.700	10.5
11	30	0.850	25.6
12	30	0.850	25.6
13	30	0.850	25.6
14	30	0.850	25.6
15	19.5	0.625	12.2
16	19.5	0.500	9.7
17	19.5	0.350	6.8
18	19.5	0.350	4.4
4 ft Unif. Load	3 KPF	0.225	0.6
		Sum	= 170.0

Stress = 2(170.0)(1.2) = (-)408 K.

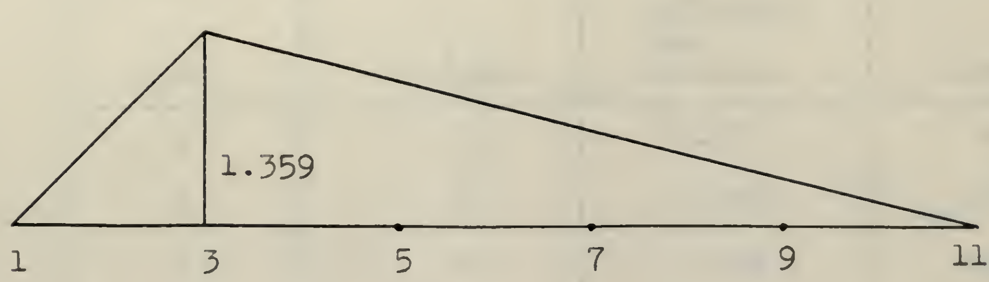
Uniform Equivalent Load = 8000 lbs./ft. of track

$$\text{Stress} = \left[1.2\right]\left[(-)(0.5)(2)(34)(0.85)(8.0) - (0.85)(17)(8.0)\right]$$

$$= (-)416 \text{ K.}$$

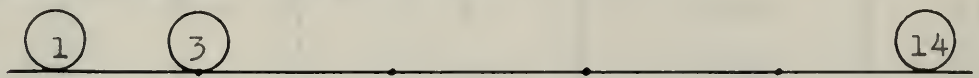
LIVE LOAD STRESSES

Member 13-14



Position I: Wheel "2" at point 3

Position II: Wheel "3" at 3



Gain = 5(0.01995)(303) = 30.2

Loss = 5(0.0799)(45) = 18.0

Position III: Wheel "4" at 3

Gain = 5(0.01995)(273) = 27.2

Loss = 5(0.0799)(75) = 30.0

Therefore use Position II

LIVE LOAD STRESSES

Member 13-14

Wheel	Weight	Influence Line Ordinate	Stress
1	15	0.32	4.8
2	30	0.958	28.6
3	30	1.359	40.8
4	30	1.255	37.8
5	30	1.153	34.6
6	19.5	0.976	19.1
7	19.5	0.876	17.1
8	19.5	0.756	14.8
9	19.5	0.638	12.4
10	15	0.498	7.5
11	30	0.339	10.1
12	30	0.239	7.2
13	30	0.141	4.2
14	30	0.040	1.2
		Sum	= 240.2

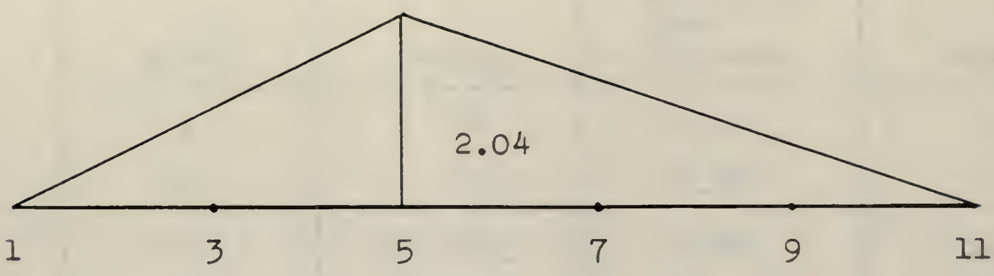
Stress = 2(240.2)(1.2) = 576 K.

Equivalent Uniform Load = 8.33 K./ft. of track

Stress = $\frac{1}{2}$(1.359)(85)(8.33)(1.2) = 576 K.

LIVE LOAD STRESSES

Member 14-15



Position I: Wheel "12" reversed at point 5

Position II: Wheel "13" reversed at point 5

$$\text{Gain} = 138(5)(0.06) = 41.3$$

$$\text{Loss} = 183(5)(0.04) = 36.6$$

Position III: Wheel "14" reversed at point 5

$$\text{Gain} = 78(5)(0.06) + 0.5(4)^2(0.06)(3) = 25.0$$

$$\text{Loss} = 183(5)(0.04) = 36.6$$

Therefore use Position II

LIVE LOAD STRESSES

Member 14-15

Wheel	Weight	Influence Line Ordinate	Stress
5	30	0.0	0.0
6	19.5	0.36	7.0
7	19.5	0.56	10.9
8	19.5	0.799	15.6
9	19.5	1.0	19.5
10	15	1.32	19.7
11	30	1.65	49.1
12	30	1.84	55.3
13	30	2.04	61.2
14	30	1.74	52.1
15	19.5	1.20	23.4
16	19.5	0.895	17.5
17	19.5	0.540	10.5
18	19.5	0.240	4.7
		Sum	= 346.4

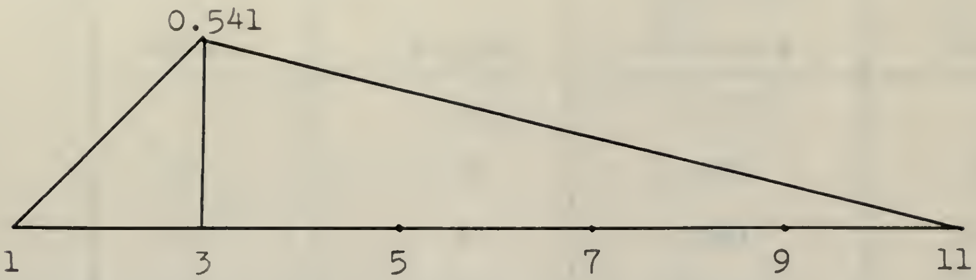
Stress = 2(346.4)(1.2) = 831 K.

Equivalent Uniform Load = 8.04 K./ft. of track

Stress = $\frac{1}{2}$(2.04)(85)(8.04)(1.2) = 835 K.

LIVE LOAD STRESSES

Members 1-13 and 2-13



Position I: Wheel "2" at point 3

Position II: Wheel "3" at point 3

Gain = 5(0.000796)(303) = 9.9

Loss = 5(0.0319)(45) = 5.9

Position III: Wheel "4" at point 3

Gain = 5(0.00796)(273) = 9.0

Loss = 5(0.0319)(75) = 9.9

Therefore use Position II

LIVE LOAD STRESSES

Members 1-13 and 2-13

Wheel	Weight	Influence Line Ordinate	Stress
1	15	0.1275	1.9
2	30	0.382	11.9
3	30	0.541	16.3
4	30	0.502	15.1
5	30	0.461	13.8
6	19.5	0.390	7.6
7	19.5	0.351	6.8
8	19.5	0.303	5.9
9	19.5	0.263	5.1
10	15	0.199	3.0
11	30	0.1355	4.1
12	30	0.095	2.9
13	30	0.056	1.7
14	30	0.016	0.5
		Sum	= 96.4

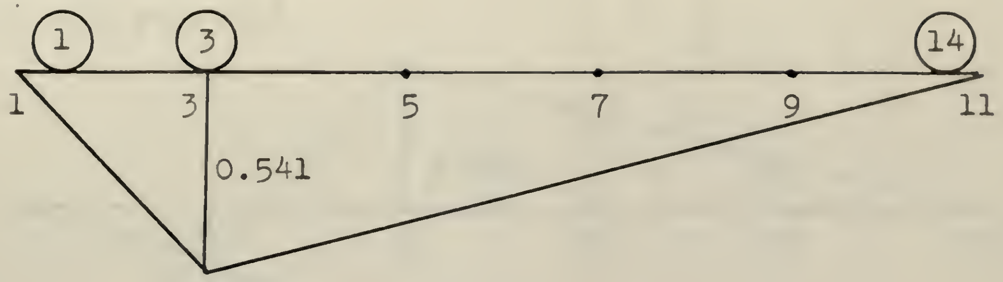
Stress = 2(96.4)(1.2) = 231 K.

Equivalent Uniform Load = 8.33 K./ft. of track

Stress = $\frac{1}{2}$(85)(0.541)(8.33)(1.2) = 230 K.

LIVE LOAD STRESSES

Members 3-13 and 4-13

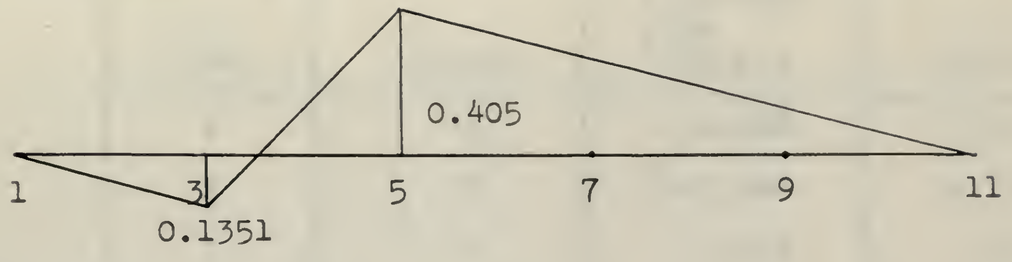


Place wheel "3" at point 3 (Same as loading for 1-13 and 2-13)

Stress = (-)231 K. (Same magnitude as 1-13 and 2-13; opposite sign)

LIVE LOAD STRESSES

Member 3-14 and 4-14



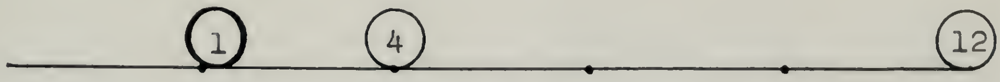
Position I: Wheel "2" at point 5

Position II: Wheel "3" at point 5

Gain: = 5(183)(0.00795) = 6.02

Loss = 5(45)(0.0315) = 5.91

Position III: Wheel "4" at point 5



Gain = 183(5)(0.00795) = 6.02

Loss = 75(5)(0.0315) = 11.8

Therefore use Position II

LIVE LOAD STRESSES

Members 3-14 and 4-14

Wheel	Weight	Influence Line Ordinate	Stress
1	15	(-)0.008	(-)0.1
2	30	(+)0.246	(+)7.4
3	30	0.405	12.1
4	30	0.366	11.0
5	30	0.326	9.8
6	19.5	0.254	5.0
7	19.5	0.214	4.2
8	19.5	0.167	3.2
9	19.5	0.127	2.5
10	15	0.063	1.0
11	30	0.0	0.0
		Sum	= 56.1

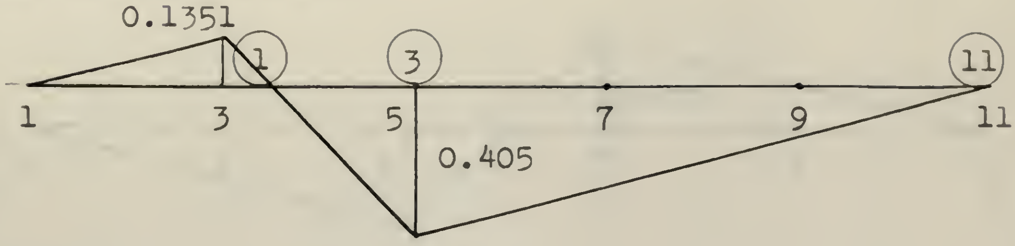
Stress = 2(56.1)(1.2) = 134.6 K.

Equivalent Uniform Load = 8820 lbs./ft. of track

Stress = $\frac{1}{2}$(0.405)(63.9)(8.82)(1.2) = 137 K.

LIVE LOAD STRESSES

Members 5-14 and 6 -14

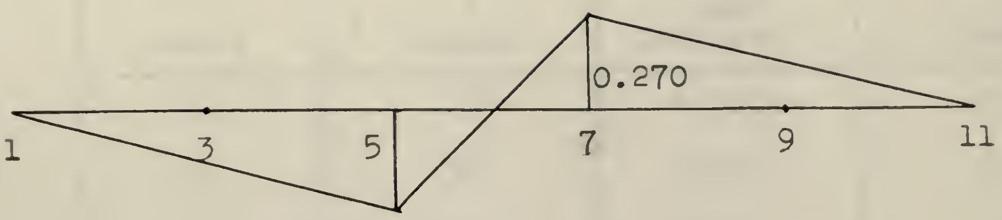


Place wheel "3" at point 5 (Same as loading for 3-14 and 4-14; opposite sign)

Stress = (-)134.6 K.

LIVE LOAD STRESSES

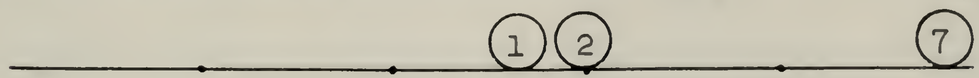
Members 5-15 and 6-15



For Tension

Position I: Wheel "1" at point 7

Position II: Wheel"2" at point 7



Gain = 8(0.00795)(159) = 10.1

Loss = 8(0.0318)(15) = 3.8

Position III: Wheel "3" at point 7

Gain = 5(0.00795)(148.5) = 5.9

Loss = 5(0.0318)(45) = 7.15

Therefore use Position II

For Compression

Place wheel "2" reversed at 5 (Same as above)

LIVE LOAD STRESSES

Members 5-15 and 6-15

Wheel	Weight	Influence Line Ordinate	Stress
1	15	0.0159	0.2
2	30	0.270	8.1
3	30	0.230	6.9
4	30	0.191	5.7
5	30	0.151	4.5
6	19.5	0.0795	1.6
7	19.5	0.0397	0.8
		Sum	27.8

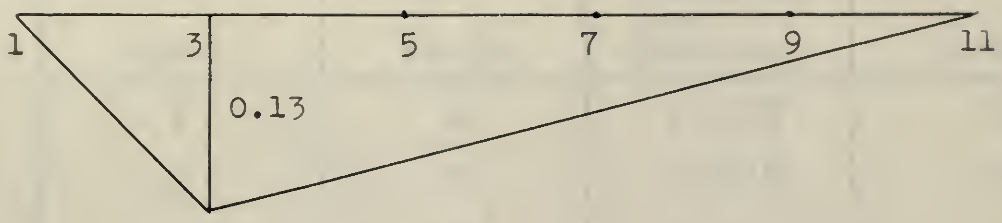
Stress = 2(27.8)(1.2) = (±)66.7 K.

Equivalent Uniform Load = 9880 lbs./ft. of track

Stress = 0.5(0.270)(42.5)(9.88)(1.2) = (±)68 K.

LIVE LOAD STRESSES

Member 1-2



Position I: Wheel "2" at point 3

Position II: Wheel "3" at point 3

Gain = 45(5)(0.00765) = 1.72

Loss = 303(5)(0.001911) = 2.89

Position III: Wheel "4" at point 3

Gain = 273(5)(0.001911) = 2.62

Loss = 75(5)(0.00765) = 2.87

Therefore use Position II

LIVE LOAD STRESSES

Member 1-2

Wheel	Weight	Influence Line Ordinate	Stress
1	15	0.0306	0.5
2	30	0.0917	2.7
3	30	0.130	3.9
4	30	0.120	3.6
5	30	0.111	3.3
6	19.5	0.0936	1.8
7	19.5	0.0841	1.6
8	19.5	0.0725	1.4
9	19.5	0.063	1.2
10	15	0.0478	0.7
11	30	0.0325	1.0
12	30	0.0229	0.7
13	30	0.0133	0.4
14	30	0.0038	0.1
		Sum	= 22.9

Stress = 2(22.9)(1.2) = (-)55.0 K.

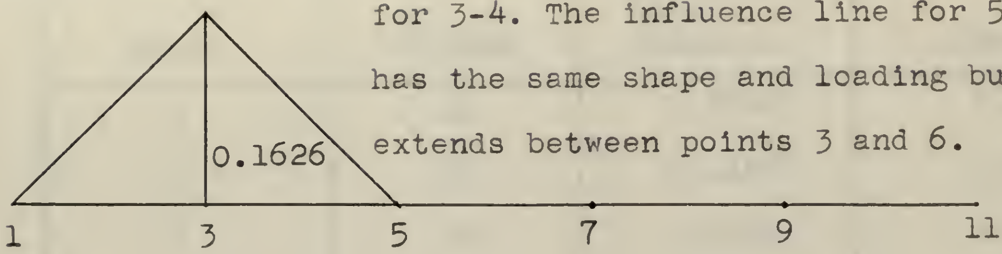
Equivalent Uniform Load = 8330 lbs./ft. of track

Stress = 0.5(85)(0.13)(8.33)(1.2) = (-)55.3 K.

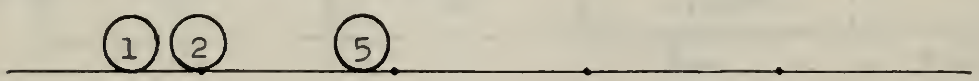
LIVE LOAD STRESSES

Members 3-4 and 5-6

The influence line shown is that for 3-4. The influence line for 5-6 has the same shape and loading but extends between points 3 and 6.



Position I: Wheel "2" at point 3



Position II: Wheel "3" at point 3

Gain = 5(0.00958)(90) = 4.31

Loss = 5(0.00958)(45) = 2.15

Position III: Wheel "4" at point 3

Gain = 5(0.00958)(60) + 3(.00958)(19.5) = 3.44

Loss = 5(0.00958)(60) + 4(0.00958)(15) = 3.46

Therefore use Position II

LIVE LOAD STRESSES

Member 3-4 and 5-6

Wheel	Weight	Influence Line Ordinate	Stress
1	15	0.0382	0.6
2	30	0.1148	3.4
3	30	0.1626	4.9
4	30	0.1148	3.4
5	30	0.0688	2.0
		Sum	= 14.3

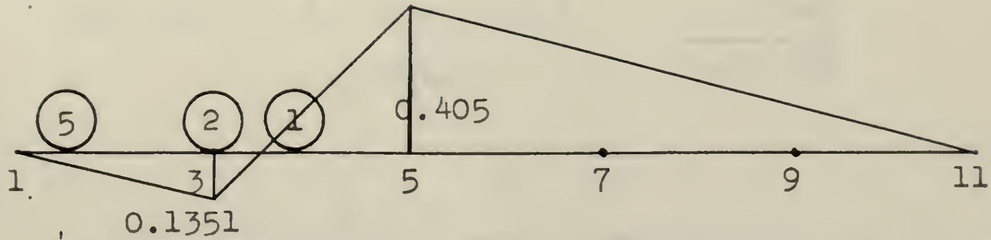
Stress = 2(14.3)(1.2) = 34.3 K.

Equivalent Uniform Load = 1052 lbs./ft. of track

Stress = 0.5(34)(0.1626)(10.52)(1.2) = 34.9 K.

REVERSAL OF STRESS

Member 3-14 and 4-14



Wheel	Weight	Influence Line Ordinate	Stress
1	15	(+)0.117	(+)1.8
2	30	(-)0.1351	(-)4.1
3	30	0.0953	2.9
4	30	0.0556	1.7
5	30	0.0159	0.5
		Sum	= (-)7.4

Stress = 2(7.4)(1.2) = (-)17.75 K.

Dead load stress = (+)20.3 K.

Therefore reversal cannot occur.

Member 5-14 and 6-14

Reversal cannot occur (Same as above; opposite sign)

Member 5-15 and 6-15

Reversal occurs (See Live Load Stress computations)

LIVE LOAD STRESSES

IMPACT

Vertical:	% = 100 - 0.6(85)	= 49.0%
Rolling	% = 2(10)5/6.5	= 15.4%
	Total	= 64.4%

DEAD LOAD STRESSES

Assume

Top Chord	= 675 lbs./ ft.	
Bottom Chord	= 350 "	"
Diagonals	= 175 "	"
Total	= 1200 "	"
Track	= 200 "	"
Ties	= 300 "	"
Guard Rail	= 60 "	"
Total	= 560 "	"

Total Assumed Dead Load = 1.76 K./ ft.

Members	Computations	Stress
1-3, 2-4	$-\left[\frac{1}{2}(0.34)(85)(1.76)\right]$	(-)25.4
3-5, 4-6	$-\left[\frac{1}{2}(0.595)(17)(1.76) + \frac{1}{2}(0.765)(51)(1.76) + \frac{1}{2}(1.76)(17)(1.36)\right]$	(-)63.5
5-7, 6-8	$-\left[\frac{1}{2}(2)(0.85)(34)(1.76) + 17(0.85)(1.76)\right]$	(-)76.1
13-14	$\frac{1}{2}(1.359)(85)(1.76)$	101.0
14-15	$\frac{1}{2}(2.04)(85)(1.76)$	153.0
1-13, 2-13	$\frac{1}{2}(0.541)(85)(1.76)$	40.5
3-13, 4-14	$-\frac{1}{2}(0.541)(85)(1.76)$	(-)40.5
3-14, 4-14	$-\frac{1}{2}(0.1351)(21.14)(1.76) + \frac{1}{2}(0.405)(63.9)(1.76)$	(-)20.3
5-14, 6-14	$\frac{1}{2}(0.1351)(21.14)(1.76) - \frac{1}{2}(0.405)(63.9)(1.76)$	(-)20.3
5-15, 6-15		
1-2	$-\frac{1}{2}(0.13)(85)(1.76)$	(-) 9.7
3-4	$\frac{1}{2}(0.1626)(34)(1.76)$	4.9
5-6	$\frac{1}{2}(0.1626)(34)(1.76)$	4.9

LATERAL STRESSES

WIND

Assumed windage area = 40% Of vertical projection

Windage area = 0.40(10)(76.5) = 306 sq.ft.

Area per foot = 306/85 = 3.6 sq.ft./ ft.

LOADED BRIDGE:

Assumed wind force = 3.6(30) = 108 lbs./ ft.

Wind on train	= 300	"	"
Wind on loaded bridge	= 408	"	"

UNLOADED BRIDGE:

Wind force = 50 lbs./ $ft.^2$

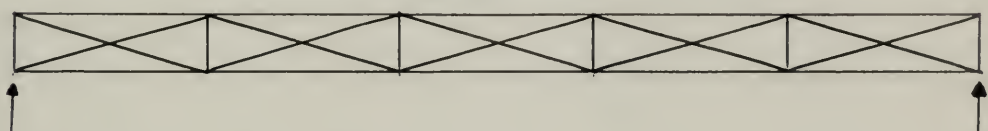
Wind load = 3.6(50) = 180 lbs./ ft.

Minimum load = 200 + 150 = 350 lbs./ ft.

Therefore Design Wind Load = 408 lbs./ ft.

PANEL CONCENTRATION = 17(0.408 K./ ft.) = 6.94 K.

NOSING = 20 K.



MEMBERS 1-2,1-3,1-4, and 3-4

$$R = 0.8(20) + \frac{4(6.9)17(2.5)}{85} = 16 + 13.8 = 29.8 \text{ K.}$$

Member 1-4 = 29.8(18.2)/6.5 = 83.4 K.

Member 3-4 = (-)29.8 K.

Member 1-3 = 29.8(17)/6.5 = (-)78 K.

Member 1-2 = 29.8/2 = (-)14.9 K.

LATERAL STRESSES

MEMBERS 3-6 AND 5-6

$$R = \frac{6.9(17)2(3)}{85} + 3(20)/5 = 8.3 + 12 = 20.3 \text{ K.}$$

Member 3-6 = 20.3(18.2)/6.5 = 56.8 K.

Member 5-6 = (-)26.9 K.

MEMBER 5-8 AND 7-8

$$R = \frac{6.9(17)1.5(2)}{85} + 2(20)/5 = 4.1 + 8 = 12.1 \text{ K.}$$

Member 5-8 = 12.1(18.2)/6.5 = 34.0 K.

Member 7-8 = (-)26.9 K.

MEMBER 3-5

$$R = \frac{4(6.9)42.5}{85} + 3(20)/5 = 13.8 + 12 = 25.8 \text{ K.}$$

$$\text{Member 3-5} = \frac{25.8(34) - 17(6.9)}{6.5} = (-)117 \text{ K.}$$

MEMBER 5-7

R = 13.8 + 8 = 21.8 K.

$$\text{Member 5-7} = \frac{21.8(51) - 6.9(34) - 6.9(17)}{6.5} = (-)117 \text{ K.}$$

LONGITUDINAL STRESSES

Members 1-3 and 2-4

Braking: 0.15(348) = 52.1 K.

Traction: 0.25(240) = 60.0 K. (use)

Moment = 1.2(60)(9) = 648 ft. K.

R = 648/85 = 7.62 K.

Stress = $\frac{7.62(0.2)(17)}{(0.4)(10)}$ = (-)6.5 K. (vertical)

Stress = 1.2(60) = (-)72.0 K. (horizontal)

Total stress = (-)78.5 K.

Members 3-5 and 4-6

Braking: 0.15(321) = 48.1 K. (use)

Traction: o.25(150) = 45 K.

Moment = 1.2(48.1)(9) = 518 ft.K.

R = 518/85 = 6.1 K.

Stress = $\frac{6.1(0.45)(17)}{(0.30)(10)}$ = (-) 15.6 K. (vertical)

Stress = 1.2(48.1) = (-)57.6 K. (horizontal)

Total stress = (-)73.2 K.

Members 5-7 and 6-8

Braking: 0.15(291) = 43.6 K. (use)

Traction: 0.25(120) = 30 K.

Moment = 1.2(43.6)(9) = 471 ft.K.

R = 471/85 = 5.54 K.

Stress = $\frac{5.54(0.5)(17)}{(0.2)(10)}$ = (-)23.6 K. (vertical)

Stress = 1.2(43.6) = (-)52.2 K. (horizontal)

Total stress = (-)75.8 K.

LONGITUDINAL STRESSES (continued)

Member 13-14

Braking = 0.15(348) = 52.2 K.

Traction = 0.25(240) = 60 K.

Moment = 1.2(60)9 = 648 ft. K.

R = 648/85 = 7.62 K.

$$\text{Stress} = \frac{7.62(0.6)17}{0.3(10)} = 25.9 \text{ K.}$$

Member 14-15

Braking = 0.15(321) = 48.1 K.

Traction = 0.25(150) = 37.5 K.

Moment = 1.2(48.1)9 = 518 ft. K.

R = 518/85 = 6.10 K.

$$\text{Stress} = \frac{6.10(0.8)17}{0.2(10)} = 41 \text{ K.}$$

Member 1-13 and 2-13

Moment = 648 ft. K. (Same as 1-3 and 2-4)

R = 7.62 K.

Stress = 7.62(13.51)/10 = 10.3 K.

Members 3-13 and 4-13

Stress = (-)10.3 K. (Same magnitude; opposite sign as 1-13 and 2-13)

Members 3-14 and 4-14

Braking = 0.15(258) = 38.7 K

Traction = 0.25(150) = 37.5 K.

Moment = 1.2(9)38.7 = 418 ft. K.

R = 418/85 = 4.92 K.

Stress = 4.92(13.51)/10 = 6.65 K.

LONGITUDINAL STRESSES (continued)

Member 5-14 and 6-14

Stress = (-)6.65 K. (Same magnitude as 3-14; opposite sign)

Members 5-15 and 6-15

Braking: 0.15(174) = 26.1 K.

Traction: o.25(120) = 30 K. (use)

Moment = 1.2(30)9 = 324 ft. K.

R = 324/85 = 3.82 K.

Stress = 3.82(13.51)/10 = (±)5.16 K.

Member 1-2

R = 7.62 K. (Same as 1-3 and 2-4)

$$\text{Stress} = \frac{7.62(0.15)6.5}{(0.3)(10)} = (-)2.5 \text{ K.}$$

Member	Dead Load Stress	Live Load Stress	Impact Stress	Lateral Load Stress	Longitudinal Force Stresses	Dead Load Live Load & Impact	Summation of all Stresses	4/5 of all Stresses	Design Stress	Connection Design Stress
1-13,2-13	40.5	231.0	149.0	-----	10.3	420.5	430.8	345.0	420.5	
3-13,4-13	(-)40.5	(-)231.0	(-)149.0	-----	(-)10.3	(-)420.5	(-)430.8	(-)345.0	(-)420.5	
3-14,4-14	20.3	134.6	86.7	-----	6.6	241.6	248.2	199.0	241.6	
5-14,6-14	(-)20.3	(-)134.6	(-) 86.7	-----	(-) 6.6	(-)241.6	(-)248.2	(-)199.0	(-)241.6	
5-15,6-15	0.0	(±) 66.7	(±) 43.0	-----	(±) 5.2	(±)109.7	(±)114.9	(±) 92.0	(±)164.2	(±)219.4
1-2	(-) 9.7	(-) 55.0	(-) 35.4	-----	(-) 2.5	(-)100.1	(-)117.5	(-) 94.0	(-)100.1	
3-4	4.9	34.3	22.1	(-)29.8	-----	61.3	61.2	49.1	61.3	
5-6	4.9	34.3	22.1	(-)29.8	-----	61.3	61.2	49.1	61.3	
13-14	101.0	576.0	371.0	-----	25.9	1048.0	1074.0	861.0	1048.0	
14-15	153.0	831.0	535.0	-----	41.0	1519.0	1560.0	1250.0	1519.0	
1-3,2-4	(-)25.4	(-)144.5	(-) 93.2	(-) 78.0	(-)78.5	(-)263.1	(-)419.6	(-)336.0	(-)336.0	
3-5,4-6	(-)63.5	(-)341.0	(-)220.0	(-)117.0	(-)73.2	(-)624.5	(-)814.7	(-)652.0	(-)652.0	
5-7,6-8	(-)76.1	(-)403.0	(-)263.0	(-)117.0	(-)75.8	(-)747.1	(-)939.9	(-)751.0	(-)751.0	
1-4,2-3	-----	-----	-----	83.4	-----	-----	83.4	66.7	66.7	
3-6,4-5	-----	-----	-----	56.8	-----	-----	56.8	45.4	45.4	
5-8,6-7	-----	-----	-----	34.0	-----	-----	34.0	27.2	27.2	

DESIGN OF TOP CHORD

BEAM ACTION

Maximum Stringer Reaction

Dead Load

Track $w = 200/2 = 100$ lbs./ft.

$R = 8.5(100) = 850$ lbs.

Ties $w = \frac{8(10)5(60)12}{144(14)} = 142.7$ lbs./ft.

$R = 142.7(8.5) = 1212$ lbs.

Guard Rail $w = 8(8)60/144 = 26.7$ lbs./ft.

$R = 26.7(8.5) = 221$ lbs.

Assumed Stringer $w = 300$ lbs./ft.

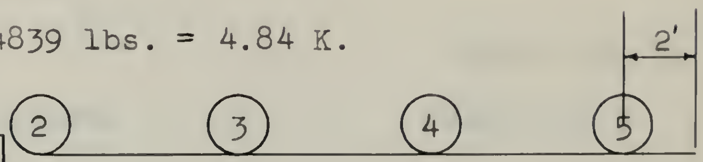
$R = 300(8.5) = 2550$ lbs.

Total Dead Load Reaction

$R(total) = 4839$ lbs. $= 4.84$ K.

Live Load

$R = \frac{1.2}{17}\left[900 + 2(120)\right]$



$R = 80.5$ K.

Impact

Direct Vertical and Rolling

$\% = 100 - 0.6\,L = 89.8$

$\% = 2(10)5/6.5 = 15.4$

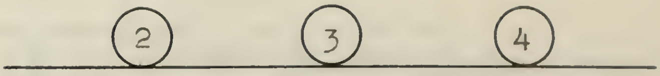
Total Impact $= 105.2\%$

$R(Impact) = 1.052(80.5) = 84.5$ K.

TOTAL MAXIMUM STRINGER REACTION $= 84.5 + 4.84 + 80.5 = 169.8$ K.

DESIGN OF TOP CHORD

BEAM ACTION



Maximum Stringer Moment

Live Load

$$R = \frac{450 + 90(3.5)}{17} = 45 \text{ K.}$$

$$\text{Moment at Center Line} = 1.2\left[45(8.5) - 150\right] = 3350 \text{ in.K.}$$

Dead Load

Track $M = \frac{wl^2}{8} = \frac{100(17)^2\,12}{8(1000)} = 43.3 \text{ in. K.}$

Ties $M = \frac{142.7(17)^2\,(12)}{8(1000)} = 61.7 \text{ in. K.}$

Guard Rails $M = \frac{26.7(17)^2\,(12)}{8(1000)} = 11.6 \text{ in. K.}$

Stringer Weight $= \frac{300(17)^2\,(12)}{8(1000)} = 130.0 \text{ in. K.}$

Total Dead Load Moment $= 246.6$ in. K.

Impact

$M = 1.052(3350) = 3530.0$ in. K.

Total Maximum Stringer Moment $= 7127$ in. K.

MOMENT OF INERTIA OF RIVET HOLES IN CHORDS

Rivets	Area		y	y	I
1	11/16	.688	1.5	2.3	1.5
2	"	"	4.5	20.2	13.9
3	"	"	7.5	56.2	38.7
4	"	"	10.5	110.2	76.0
5	"	"	13.5	182.2	125.5
				Sum	= 255.6
Flange 6	4(1.125)		17.5	306.0	1378.0

Total I of Rivets = 4(255.6) + 4(1378) = 6534 in.

MOMENT OF INERTIA OF MEMBER 3-5 AND 4-6 AT SPLICE

Net I = 2(10470) - 6534 = 14406 in.

MOMENT OF INERTIA OF MEMBER AT SECTION OF MAXIMUM STRESS

Net I = 2(10470) - 2(1378) = 18184 in.

DESIGN OF TOP CHORD

Maximum stress occurs in 5-7, member being designed for combined axial and flexural stresses.

M(max.) = 7127 in. K.

Maximum axial stress = (-)751.0 K.

TRY 2-36 WF 170 SECTIONS

Flexure

$l/b = 17(12)/12 = 17$

f(all.) = 16.56 Ksi.

$f(act.) = \frac{3(7127)18}{4(18184)} = 5.31$ Ksi.

Axial Load

$l/r = 17(12)/2.45 = 83.3$

f(all.) = 13.24 Ksi.

$f(act.) = \frac{(-)751}{2(49.98)} = 7.52$ Ksi.

Total f(act.) = 5.31 + 7.52 = 12.83 Ksi. Satisfactory

After a study of possible savings in steel against greater simplicity of joints and splices, we decided to use the same section for all of the members in the top chord.

CHECK IN MEMBERS 1-3 and 2-4

Flexure

f(all.) = 16.56 Ksi. M = 7127 in. K.

f(act.) = 7127(18)/18184 = (-)7.07 Ksi.

Axial Load f(all.) = (-)13.24 Ksi.

f(act.) = 336/2(49.98) = (-)3.37 Ksi.

Combined Stresses

f(act.) = (-)(7.07 + 3.37) = (-)10.44 Ksi.

f(all.) = (-)13.24 Ksi.

DESIGN OF TOP CHORD

TOP CHORD SPLICE

Web Splice

Moment of Inertia of One Row of Rivets in the Web

Rivets	Area	y	y	I
1	11/16(7/8)	1.5	2.3	1.4
2	"	4.5	20.2	12.2
3	"	7.5	56.3	33.9
4	"	10.5	110.4	66.5
5	"	13.5	182.3	109.8

Sum = 223.8

I of one row = 2(223.8) = 447.6

I of one row in both WF sections = 2(447.6) = 895.2

Axial Stress

Net area of one web = 11/16(34) - 10(0.688) = 16.47

Total net area = 2(49.98 - 6.88 - 9.00) = 68.20

Axial stress in web = $\frac{(-)652(2)16.47}{68.20}$ = (-)315 K.

Bending Stress

I of web = 2(2251.8) = 4503.6

f(all.) = 18 Ksi.   c = 17

M - $\frac{18(4503.6)}{17}$ = 4770 in. K.

Shear Stress

Value of web = 11/16(34)(11) = 257 K.

DESIGN OF TOP CHORD

TOP CHORD SPLICE

Web Splice (continued)

Try 6 Rows of Rivets

Area of one row of rivets = 11/16(7/8)10 = 6.02

$f(\text{shear}) = \frac{257}{6.02(6)} = 7.11 \text{ K./ in.}^2$

$f(\text{moment}) = \frac{4770(17)}{6(895.2)} = 15.1 \text{ K./ in.}^2$

$f(\text{axial}) = \frac{315}{6(6.02)} = 8.72 \text{ K./ in.}^2$

$f(\text{total}) = \sqrt{(7.11)^2 + (15.1 + 8.72)^2} = 24.8 \text{ K./ in.}^2$

$f(\text{all.}) = 27 \text{ K./ in.}^2$

Flange Splice

Axial Stress

Axial load = (-)652 K. = 326 K./ member

Net area of one flange = 12(1.125) - 4.5 = 9.0

Total net area/ member = 34.10

Stress in one flange = $\frac{9(326)}{34.10}$ = (-)86.0 K.

Bending Stress

I of section = 2(10470) = 20940

f(act.) at edge of flange = $\frac{3(7127)18}{4(18184)}$ = (-)5.30 Ksi.

Bending stress in flange = 5.30(8)1.125 = (-)47.7 K.

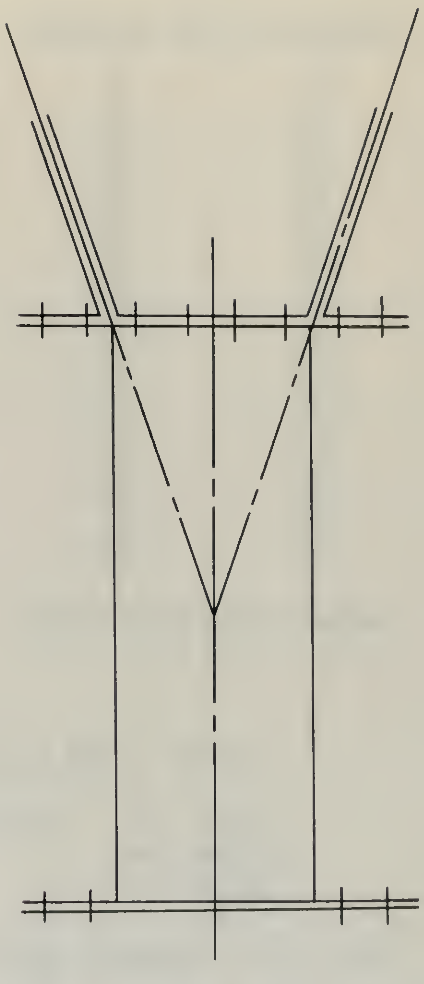
Total Stress

Axial stress	= (-)86.0 K.
Bending Stress	= (-)47.7 K.
Total	= (-)133.7 K.

Number of Rivets = $\frac{133.7}{8.12}$ = 16.45

Rows = $\frac{16.45}{4}$ = 4.11 Use five

DESIGN OF BOTTOM CHORD



Stress = 1519.0 K.

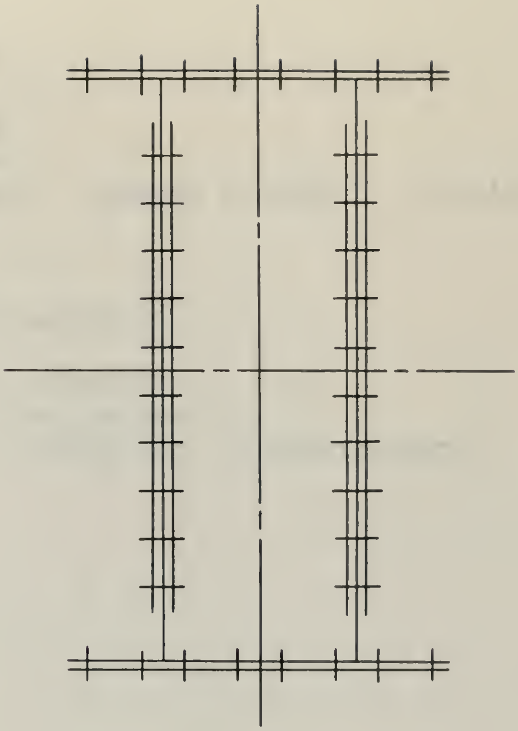
Try 2-36 WF 182

Net Area Required = 1519.0/18		= 84.30
Deduct 12, 7/8 inch rivets = 12(1.188)		= 14.25
Cut $1\frac{1}{2}$ inches off each bottom chord for backing up rivets		= 3.56
Gross area required		= 102.11
Gross area available		= 107.08

DESIGN OF BOTTOM CHORD

BOTTOM CHORD SPLICE

Splice in Member 13-14



Stress = 1048 K.

Flange Splice

Gross area of one flange = 12(1.187) = 14.24

Deduct area of 4, 7/8 inch rivets = 4.75

Effective area of the flange = 9.49

Strength of effective area of one flange = 18(9.49) = 171 K.

Rivets required = 171/8.12 = 21.0 Use 5 rows of rivets.

Web Splice

Gross area of one web = 0.75(33.96) = 25.45

Deduct 10, 7/8 inch rivets = 7.50

Effective area of web = 17.95 sq. in.

Strength of effective area of one web = 18(17.95) = 319 K.

Number of rivets = 319/16.23 = 19.65

Use 2 rows of ten rivets in each web splice plate.

DESIGN OF BOTTOM CHORD

BOTTOM CHORD SPLICE

Value of Net Section

Webs: 2(319) = 638 K.

Flanges: 4(171) = 684 K.

Total: = 1322 K. Stress = 1048 K. Satisfactory

Value of Splice Rivets

Flange rivets: 24(4)8.12 = 780 K.

Web rivets: 10(4)16.23 = 650 K.

Total = 1430 K. Satisfactory.

DESIGN OF DIAGONALS

Members 1-13, 2-13 Stress = 420.5 K.

Try 30 WF 116

l=13.51 ft. r = 2.12 inches

A = 34.13

t(web) = 9/16 inches

l/r = 13.51(12)/2.12 = 76.5

Net area required = 420.5/18	= 23.40	
Deduct 6 web rivets = 6(.563)	= 3.38	
Deduct 8 flange rivets = 8(.875)	= 7.00	
Gross area required	= 33.78	Satisfactory

Members 3-13, 4-13 Stress = (-)420.5 K.

Try 30 WF 108

r = 2.06 inches

l/r = 13.51(12)/ 2.06 = 78.8

f(all.) = 13.44 Ksi.

Area required = 420.5/13.44 = 31.3 sq. in.

Area available = 31.77 Satisfactory

Rivets for 1-13, 2-13, 3-13, and 4-13 Stress = 420.5 K.

Single Shear Rivets

Minimum = 420.5/8.12 = 51.8

Use: 44 rivets in web (4 rows of eleven)

8 rivets in flange

Double Shear Rivets

Minimum = 420.5/13.28 = 31.7 Use 33

DESIGN OF DIAGONALS

Members 3-14, 4-14 Stress = 241.6 K.

Try 21 WF 62

r = 1.71 inches

l/r = 13.51(12)/1.71 = 94.8

Net area required = 241.6/18 = 13.41

Deduct 6 rivet holes = 6(0.375) = 2.25

Gross area required = 15.66

Gross area available = 18.23 Satisfactory.

Members 5-14, 6-14 Stress = (-)241.6 K.

Try 21 WF 68

r = 1.74 inches

l/r = 13.51(12)/1.74 = 93.2

f(all.) = 12.79 Ksi.

Area required = 241.6/12.79 = 18.90

Area available = 20.02 Satisfactory.

Rivets for 3-14, 4-14, 5-14, and 6-14

Single Shear Rivets

Minimum = 241.6/8.12 = 29.8 Use 32 (4 rows of 8)

Double Shear Rivets for 5-14 and 6-14

Minimum = 241.6/10.34 = 23.4 Use 27

Double Shear Rivets for 3-14 and 4-14

Minimum = 241.6/8.80 = 27.3 Use 33

DESIGN OF DIAGONALS

Members 5-15, 6-15 Stress = (±)164.2 K.

Tension Design

Try 18 WF 60

Net area required = 164.2/18 = 9.13

Deduct 5,7/8 inch rivets = 5(0.438) = 2.19

Gross area required = 11.32

Gross area available = 17.64

l/r = 13.51(12)/1.63 = 99.5 Satisfactory

Compression Design

Try 18 WF 60

f(all.) = 12.50 Ksi.

Area required = 164.2/12/50 = 13.14 sq. in.

Area available = 17.64 Satisfactory

Rivets for 5-16, 6-15

Connection design stress = (±)219.4 K.

Single Shear Rivets

Minimum = 219.4/8.12 = 27.0 Use 28 (4 rows of 7)

Double Shear Rivets

Minimum = 219.4/10.34 = 21.2 Use 25

DESIGN OF LATERALS

Member 1-2

Stress = (-)100.1 K.

Try 16 WF 45

1/r = 6.5(12)/1.52 = 51.3

f(all.) = 14.32 Ksi.

Area required = 100.1/14.32 = 6.98 sq. in.

Area available = 13.24 sq. in.

Single Shear Rivets

Minimum = 100.1/8.12 = 12.3 Use 16

Double Shear Rivets

Minimum = 100.1/8.86 = 11.3 Use 12

Members 3-4, 5-6

Stress = 61.3 K.

Try 16 WF 45

Net area required = 61.3/18	= 3.41
Deduct 4, 7/8 in. rivets in web	= 1.50
Deduct 2, 7/8 in. rivets in flange	= 1.13
Gross area required	= 6.04
Gross area available	= 13.25
1/r = 6.5(12)/ 1.52 = 51.3	Satisfactory

Single Shear Rivets

Minimum = 61.3/ 8.12 = 7.56 Use 8

Double Shear Rivets

Minimum = 61.3/ 8.86 = 6.94 Use 8

DESIGN OF LATERAL BRACING

FIRST PANEL Members 1-4, 2-3

Stress = 66.7 K.

Try 8 × 6 × 7/16 Angles

r = 1.31 inches

l/r = 18.22(12)/1.31 = 167

Effective Area = 7/16(8) + $\frac{1}{2}$(5.563)7/16 - 2(0.438)

= 3.85 sq. in.

Required Area = 66.7/ 18 = 3.71 sq. in. Satisfactory

Rivets

Strength of member = 18(3.85) = 69.4 K.

Minimum number = 69.4/ 8.12 = 8.55 Use 9, 7/8 rivets

SECOND PANEL Members 3-6, 4-5

Stress = 45.4 K.

Try 6 × 6 × 3/8 Angles

r = 1.19

l/r = 18.22(12)/ 1.19 = 184

Effective Area = 3/8(6) + $\frac{1}{2}$(5.625)3/8 - 2(0.375)

= 2.55 sq. in.

Required Area = 45.4/18 = 2.52 Satisfactory

Rivets

Strength of member = 2.55(18) = 45.9 K.

Minimum number = 45.9/ 8.12 = 5.65 Use 6, 7/8 rivets

DESIGN OF LATERAL BRACING (continued)

CENTER PANEL Members 5-8, 6-7

Stress = 27.2 K.

Try 6 × 6 × 3/8 Angles

r = 1.19

l/r = 18.22(12)/ 1.19 = 184

Effective Area = 2.55 sq. in.(Same as for second panel)

Required Area = 27.2/ 18 = 1.51 Satisfactory

Rivets

Strength of member = 2.55(18) = 45.9 K.

Minimum rivets = 45.9/ 8.12 = 5.6 Use 6, 7/8 rivets

LACING DESIGN

$$r = \sqrt{\frac{I}{A}} = \frac{h}{\sqrt{12}} = 0.289\ h$$

h = t(flange of 36 WF 170) = 1.125 in.

r = 0.289(1.125) = 0.325 in.

$$2/3 \text{ of } l/r \text{ Of flange} = \frac{2(12)(17)}{3(2.42)} = 56.2$$

Maximum distance between rivets in one flange =

0.325(40) = 13 inches

Normal Shearing Force

r = 2.42 in   l/r = 17(12)/ 2.42 = 84.2

$$V = \frac{47.09(13.19)}{100}\left[\frac{100}{82.8 \quad 10} + \frac{82.8}{100}\right] = 11.82\ \text{K.}$$

Use Double Lacing, Flat Bars

t(minimum) = 20/60 = 0.333 in.   Use 3/8 inch bars

l/r = 20/(0.375)0.70 = 130

f(all.) = 10.78 Ksi.

$$\text{Area required} = \frac{11.82(20)}{4(15.25)10.78} = 0.361\ \text{sq. in.}$$

Width = 0.361/0.375 = 0.962 in.

Minimum width = 3(7/8) = 2.625 in.

Use 3/8 by 3 inch bars.

MAXIMUM END REACTION

Dead Load

Ties, rails, fittings, and guard rails

$R = \frac{1}{2}(42.5)(0.560) = 11.9$ K

Top Chord

$R = 42.5(0.3) = 12.75$

Bottom chord

$R = 34(0.3) = 10.2$

Diagonals

$R = 5(13.51)0.3 = 20.3$

Live Load

$R = 1.2(16670)/85 = 235.0$ K.

Impact

Rolling Effect

$5/18(2)10 = 5.56\%$

Direct Vertical Effect

$100 - 0.6(85) = 49\%$

Total Impact = 54.6%

Impact Reaction = 128.1 K.

Total Maximum Reaction = 418.1 K.

STIFFENER DESIGN AT END BEARING PLATES

Use 6 × 4 Angles 8 angles total

Bearing

$$t(req.) = \frac{418.1}{8(27)5.625)} = 0.344 \text{ in.}$$

Axial Compression

$$\text{Area required} = \frac{418.1}{8(18)} = 2.9 \text{ sq. in.}$$

Use 6 × 4 × 3/8 Angles

Rivets to Web

$$\text{Minimum rivets} = \frac{418.1}{4(16.24)} = 6.44$$

T distance = 32.25 in. ⟶ Use 11 rivets at 3 in. spacing

Required Area of End Bearing Plate

Assume Concrete Foundation

p(all.) = 600 psi.

$$\text{Area required} = \frac{418.1}{0.6} = 697 \text{ sq. in.}$$

CHECK OF ACTUAL BRIDGE WEIGHT AGAINST ASSUMED WEIGHT

ACTUAL WEIGHT OF BRIDGE PER FOOT

Top Chord	
170(2)170	57,800 lbs.
Bottom Chord	
68(2)182	24,800 lbs.
Horizontals	
6.0(6)45	1,620 lbs.
Diagonals	
4(9.5)116	4,440 lbs.
4(9.5)108	4,100 lbs.
4(9.5)62	2,360 lbs.
4(9.5)68	2,590 lbs.
4(9.5)60	2,280 lbs.
Bracing	
4(18.2)20.2	1,470 lbs.
6(18.2)14.9	1,620 lbs.
Total Weight of Bridge	103,080 lbs.

Weight in Kips/ ft. $= \frac{103,800}{85(1000)} = 1.22$ K./ ft.

ASSUMED DEAD WEIGHT OF BRIDGE = 1.20 K./ ft. Satisfactory.

CONCLUSIONS

The preceding investigation demonstrates conclusively the similarity between the influence lines of space frames and conventional trusses. After gaining familiarity with the subject, it is possible, merely by inspection, to determine the shape of the influence line of any member of a space frame bridge. This enables the influence line to be determined by a solution at only one or possibly two points. The amount of work necessary for solution is thus reduced by one-half in the case of this five panel bridge. The use of influence lines is just as benificial in space frame trusses as in ordinary ones. The equivalent uniform loads as proposed by Steinman in Transactions A.S.C.E., Vol. LXXXVI for planar trusses have been shown to be equally applicable to space frames. By making use of this fact considerable time can be saved with no appreciable loss in accuracy.

The difficulty in obtaining practical joints in the Pratt and Howe bridges forced the authors to forego their lesser stresses and potential weight savings for the simpler joints and somewhat greater stresses of the Warren type. If a satisfactory joint detail can be devised for the Howe or Pratt trusses, it is thought that a somewhat lighter bridge might be developed.

The total weight of this Warren type space frame bridge was computed to be 103,000 pounds. The weight of a plate girder bridge of equal span and designed for the same loading and specifications was computed to be 115,000 pounds. The

resulting saving of 12,000 pounds is appreciable---amounting to 12% of the total weight of the space frame bridge. Undoubtedly this saving in steel would not pay for the increased fabrication costs on the first bridge constructed. The fabrication costs, however, would be materially reduced as special techniques were developed through experience. This trend, coupled with the considerable savings in the number of rivets required and the decrease in erection costs as no field riveting is required, would, in the opinion of the authors, reduce the cost to only slightly greater than that of the plate girder bridge.

To summarize, it is believed that as greater experience in the construction of space frame bridges results in decreased fabrication costs, a space frame bridge of this span can compete favorably with a plate girder bridge where rapid transportion and erection are important.

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2-36WF170
30WF116
2-36WF182
30WF108
16WF45
Joint 3-4


86'-9"
85'-0"
5'-5¾"
6×6×⅜ L
21WF62
21WF68
ALL DIMENSIONS ON THE
DIAGONALS ARE IN THE
PLANE OF THE MEMBER

WARREN TYPE SPACE FRAME RAILWAY BRIDGE

THREE CHORD DECK TYPE

DEPARTMENT OF CIVIL ENGINEERING
RENSSELAER POLYTECHNIC INSTITUTE
TROY, NEW YORK

DESIGNED BY:

DRAWN BY:

CHECKED BY:

SCALE: 1" = 1'0"

APRIL 7, 1948