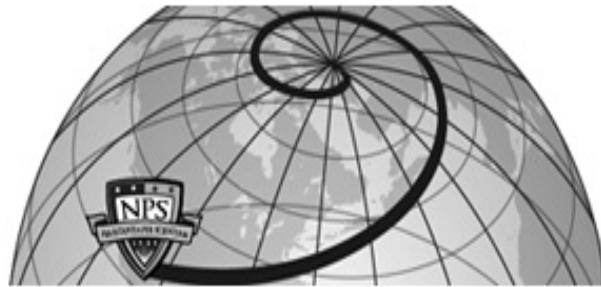




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Design of a deck type, three chord, space frame railway bridge

Williams, John Paxton

Rensselaer Polytechnic Institute

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

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

Thesis
W6

Thesis
W6

Postgraduate School.
U. S. Naval Academy,
Annapolis, Md.

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

Submitted to the faculty of
Rensselaer Polytechnic Institute in
partial fulfillment of the require-
ments for the degree of Master of
Civil Engineering.

By
John F. Williams

and

Foster M. Lalor Jr.

May, 1948

Troy, New York

ACKNOWLEDGEMENT

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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 separately 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 (A.R.A. 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.

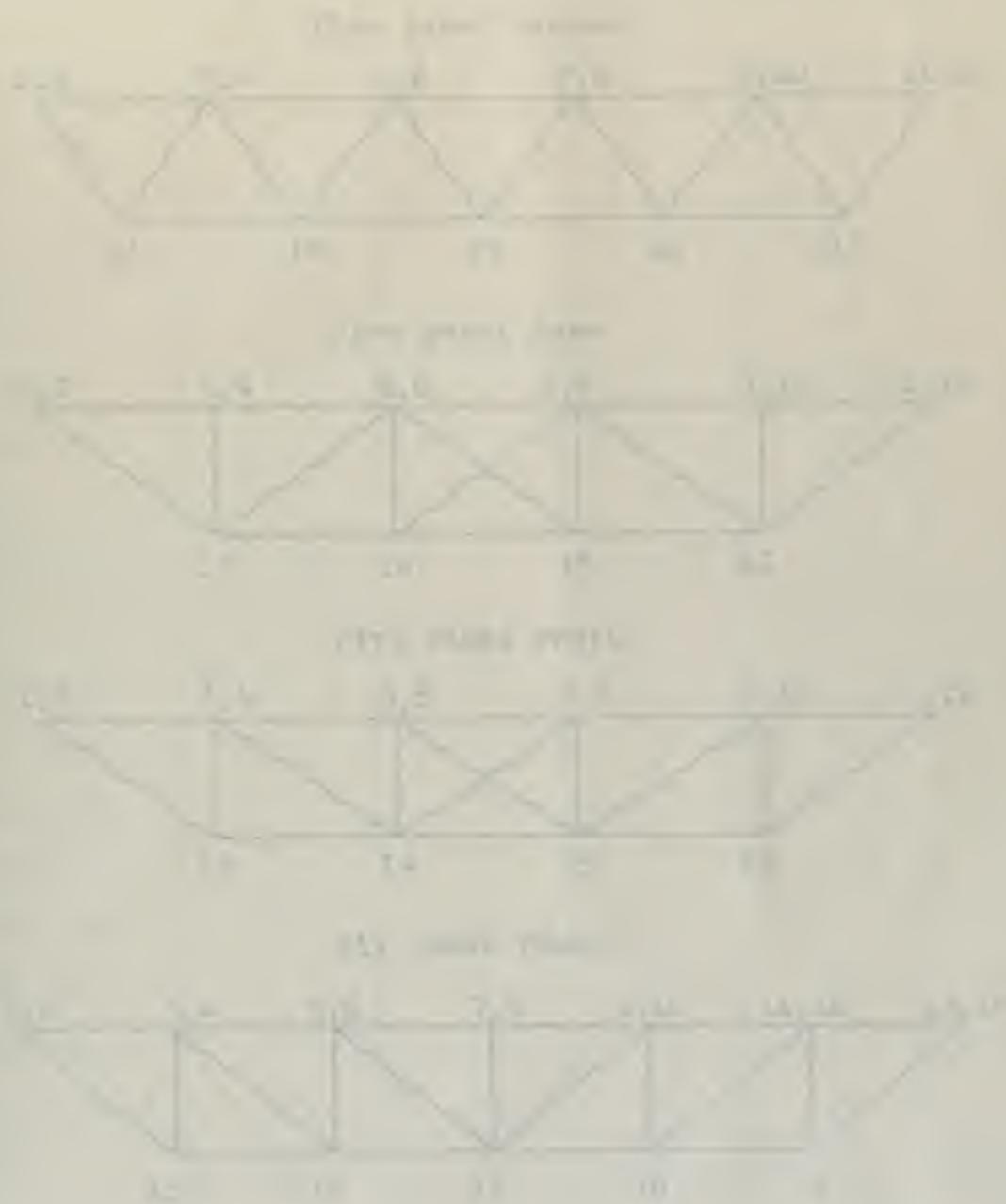
In order to realize all of the advantages of a space frame bridge mentioned in the introduction, it was necessary to select the dimensions of the truss to give the most compact cross section compatible with specifications, member size, and joint design. Both for economy and for ease of transportation 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

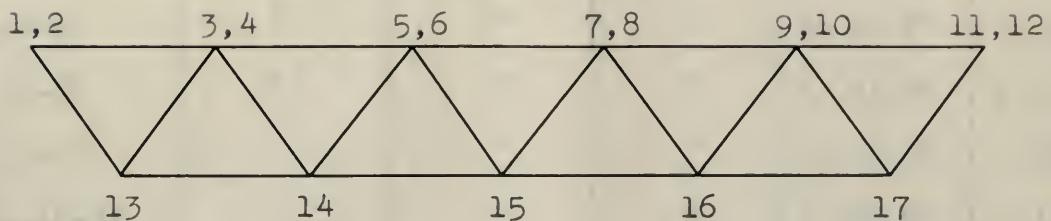
The selection of the truss to be used could best be made by a comparison of the influence lines of the members for each type. The method of solution is a variation of the tension

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

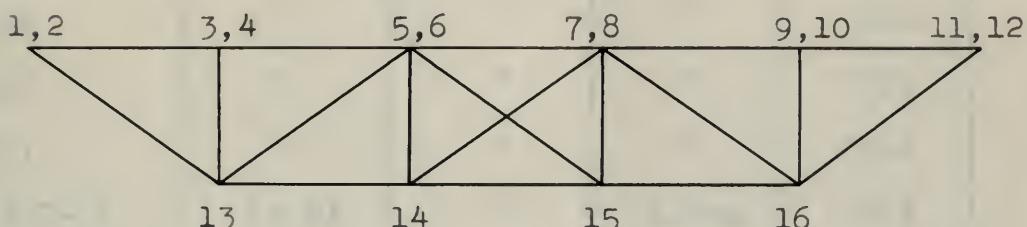


SKETCHES OF SIDE ELEVATIONS OF TRUSSES INVESTIGATED

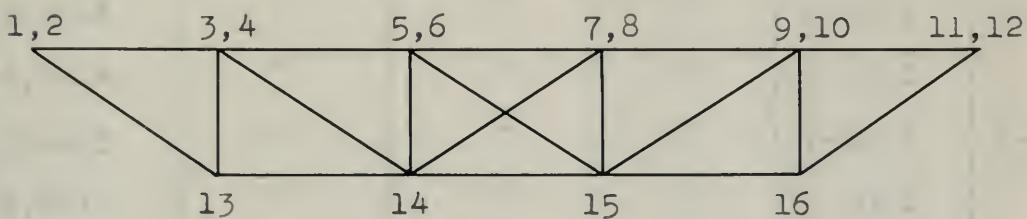
Five Panel Warren



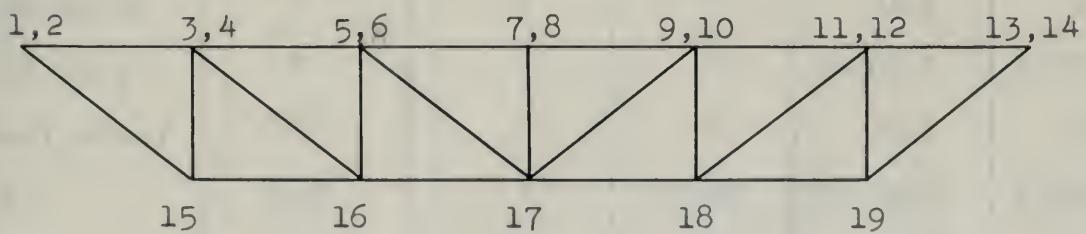
Five Panel Howe



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

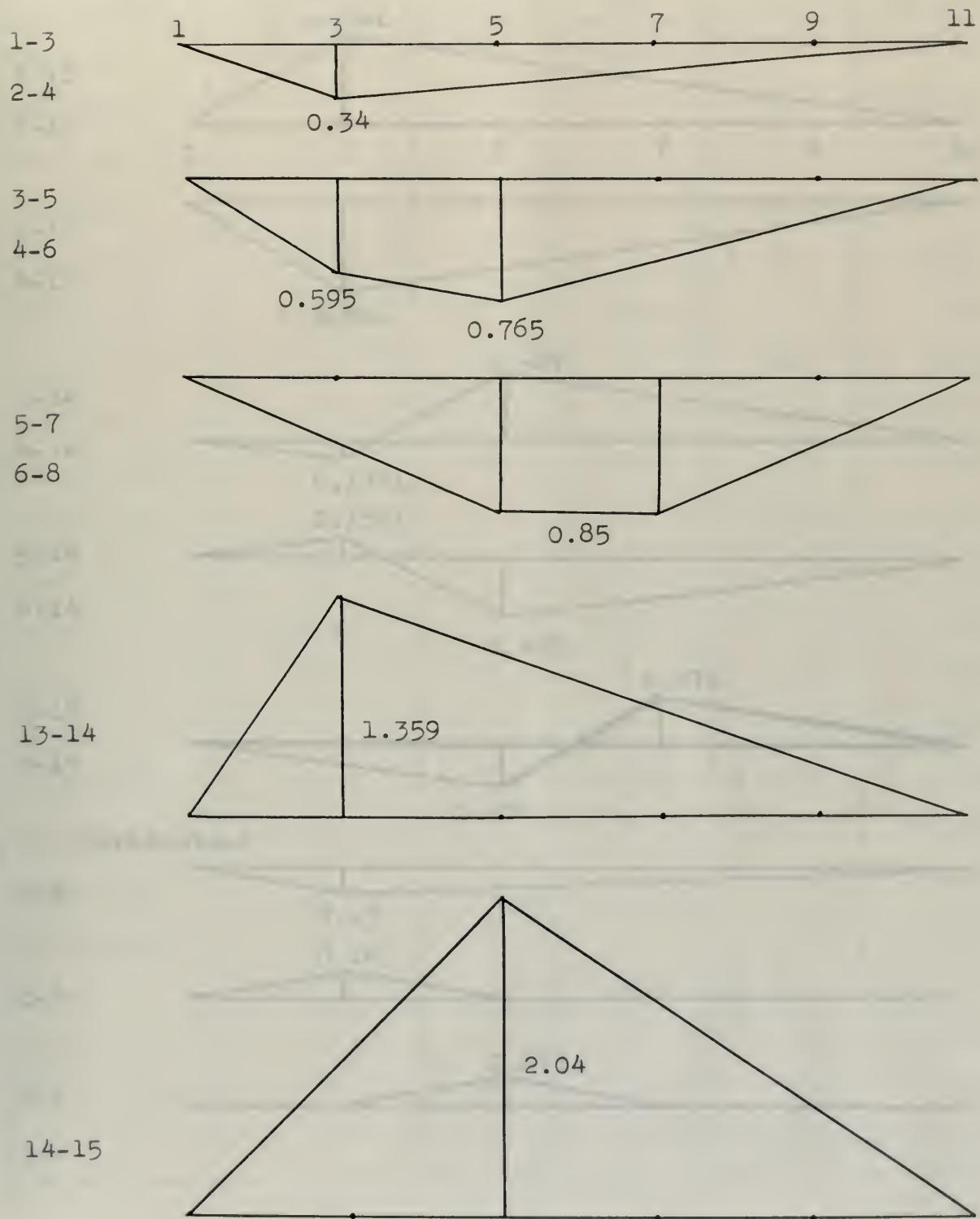
5 PANEL WARREN TYPE TRUSS Load at 5-6

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

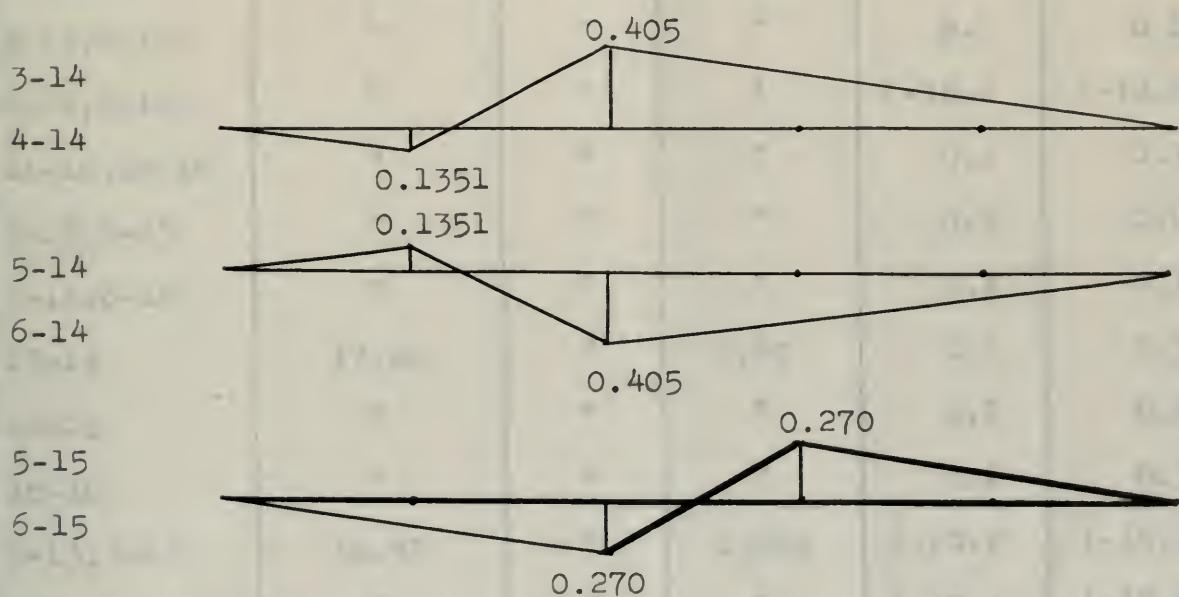
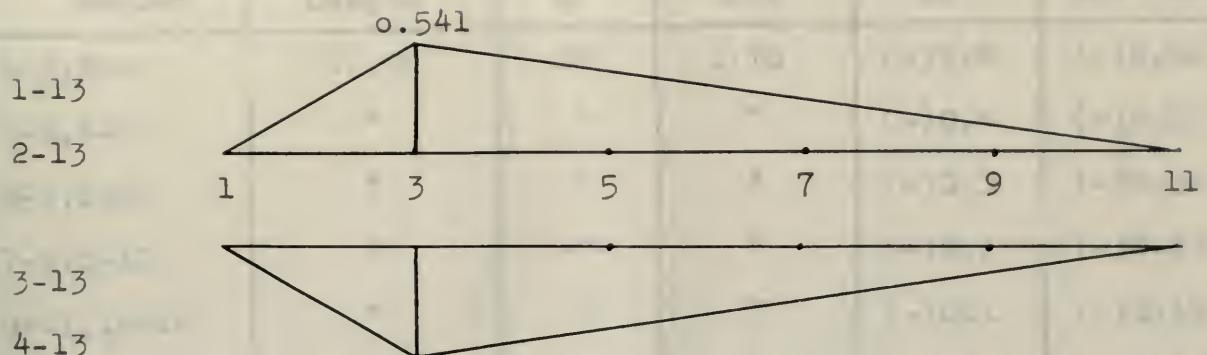
Chords



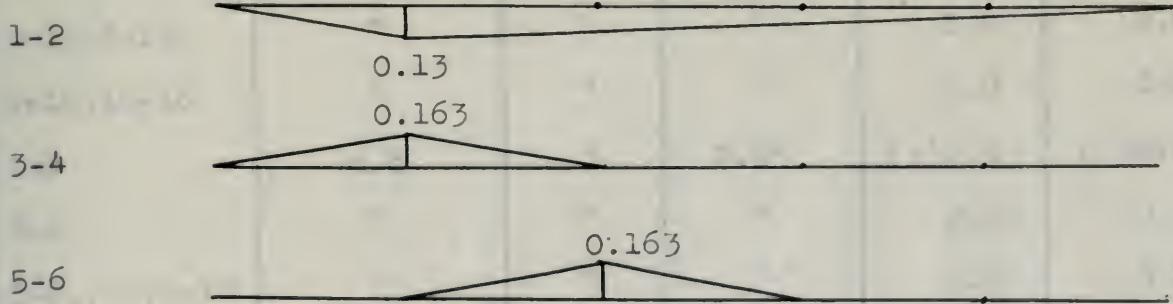
INFLUENCE LINES

5 PANEL WARREN TYPE TRUSS

Web Members



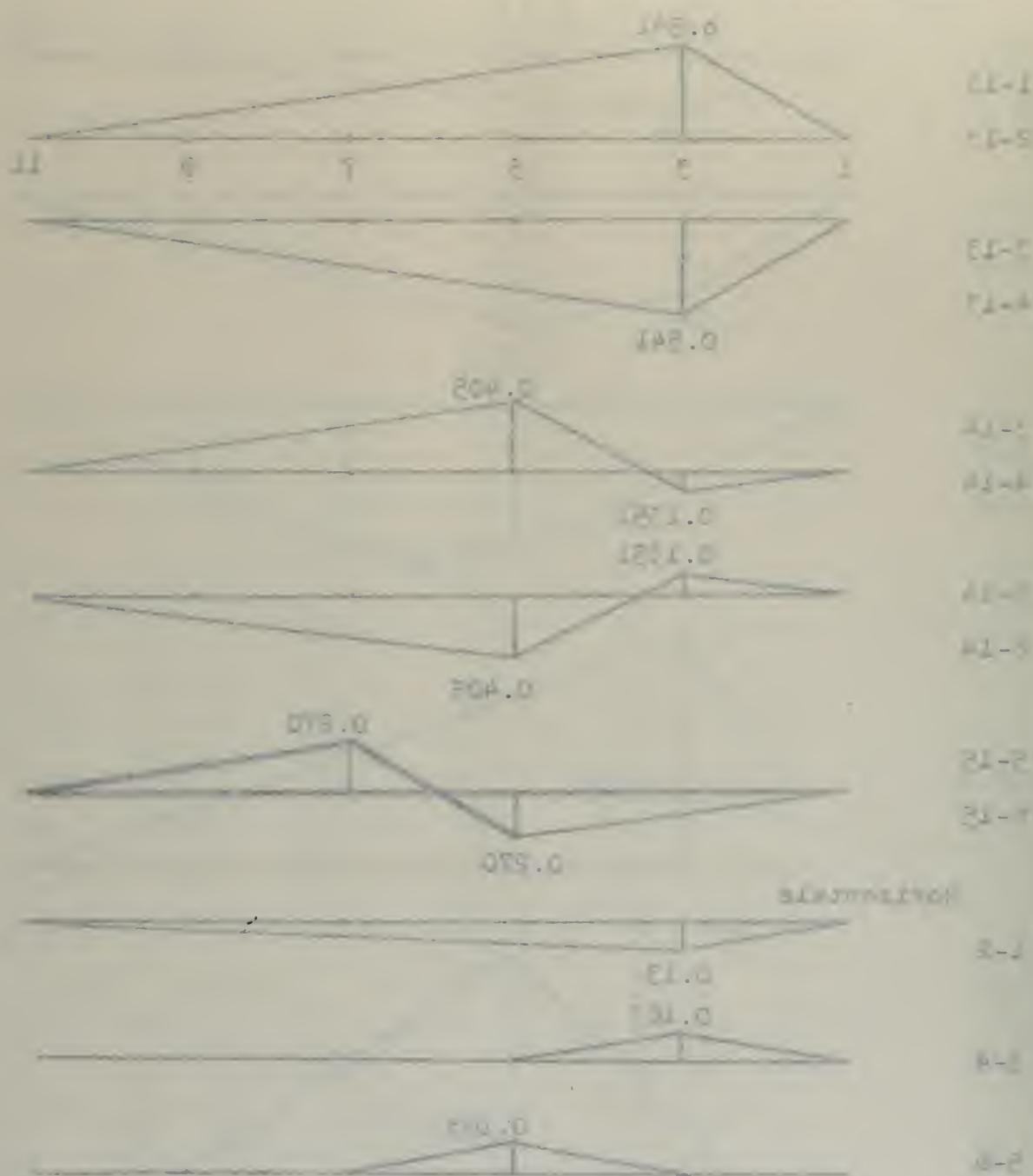
Horizontals



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STATION 2000



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

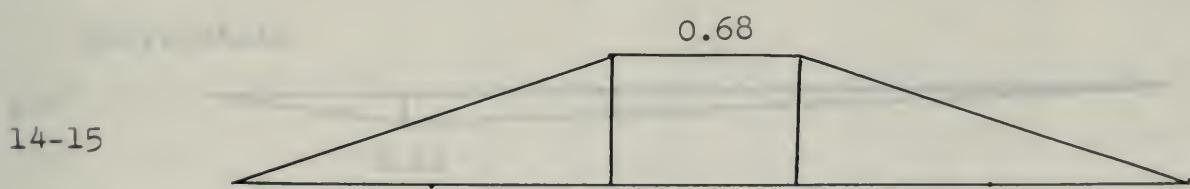
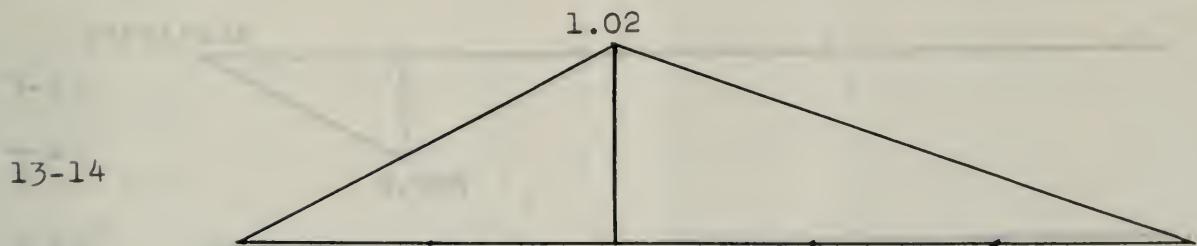
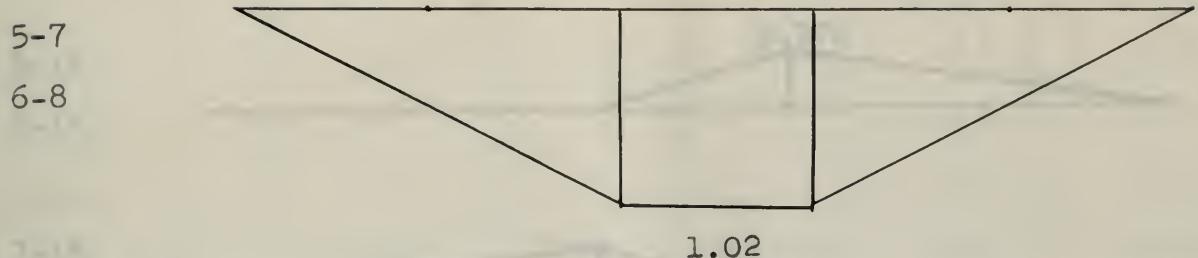
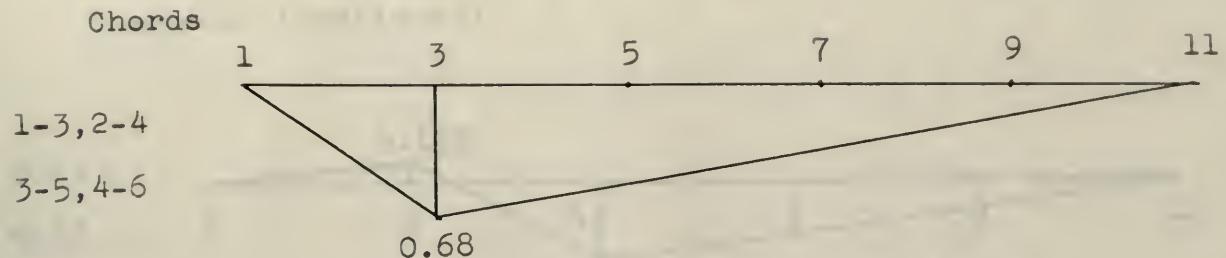
INFLUENCE LINES

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

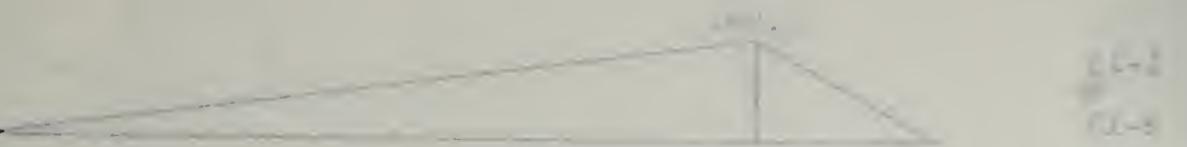
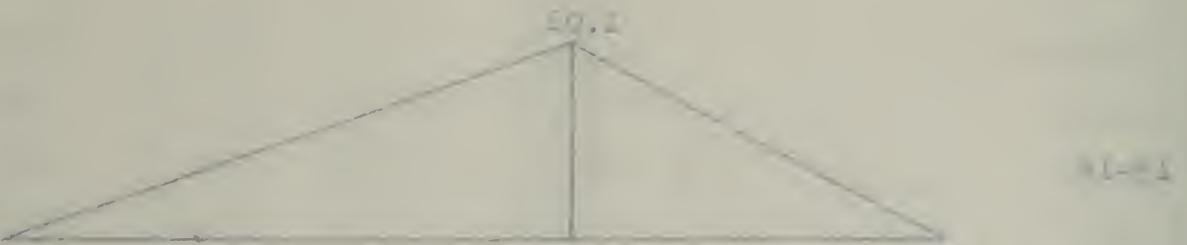
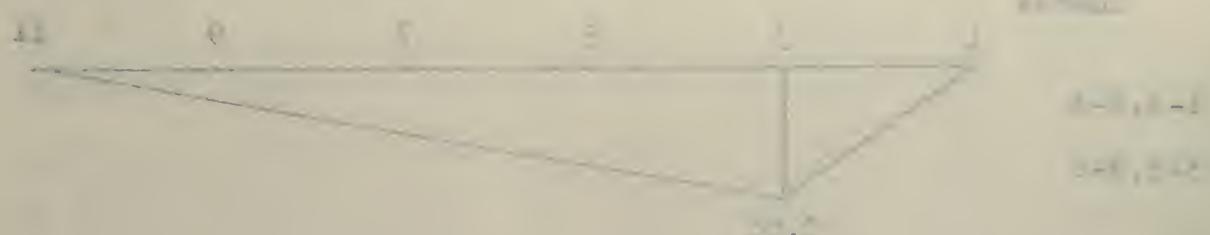
INFLUENCE LINES

5 PANEL HOWE TYPE TRUSS



Diagonals

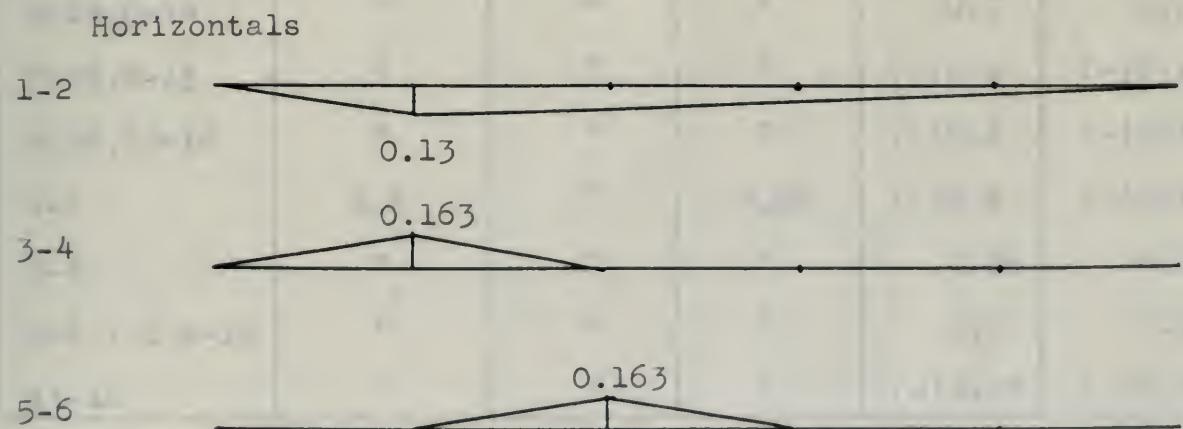
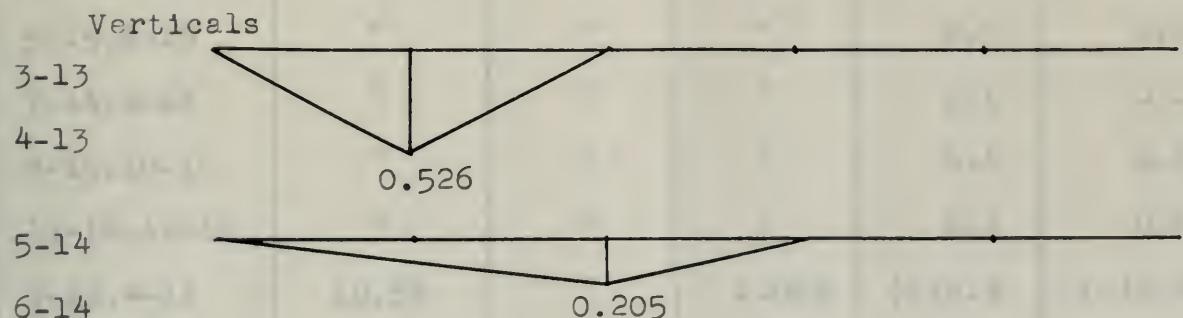
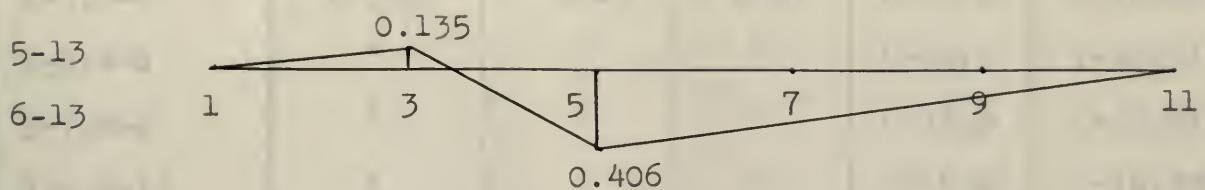


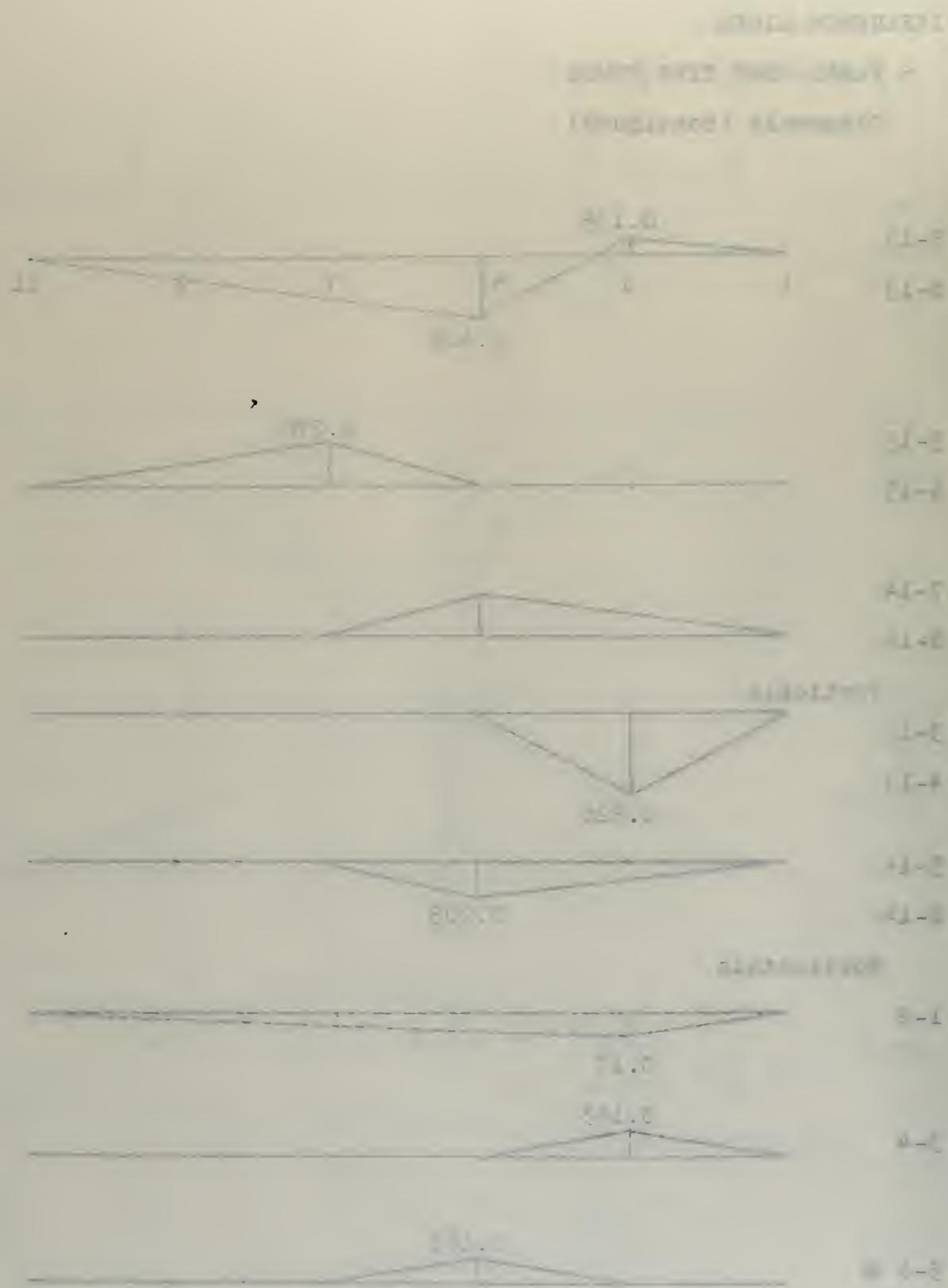


INFLUENCE LINES

5 PANEL HOWE TYPE TRUSS

Diagonals (continued)





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

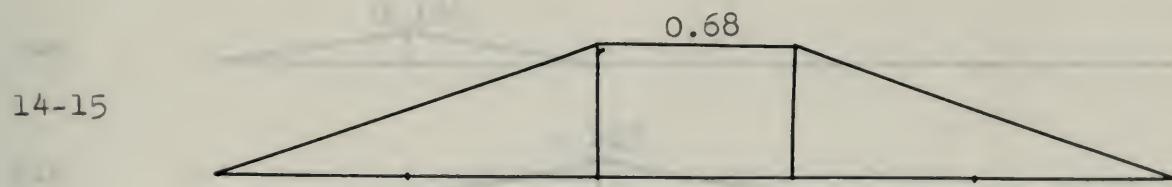
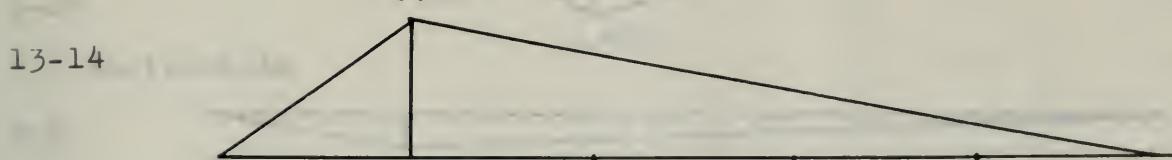
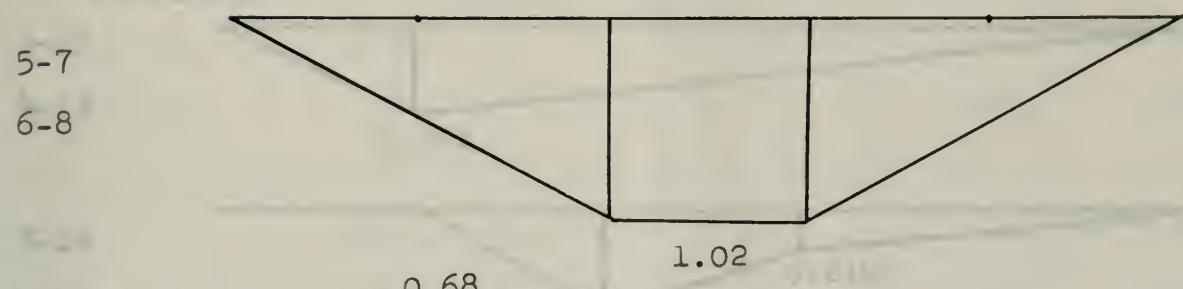
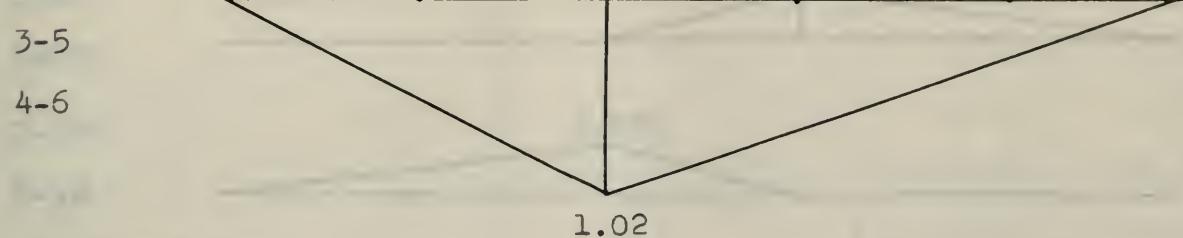
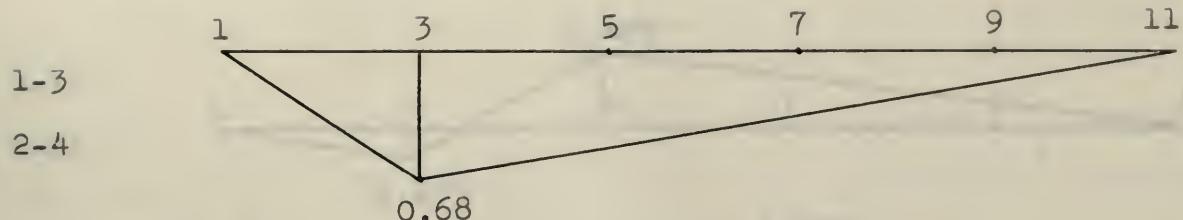
5 PANEL PRATT TYPE TRUSS Load at 5-6

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



Diagonals 0.59



SMALL CONCENTRIC

WATER LEVEL TOWER DRAWS 2'

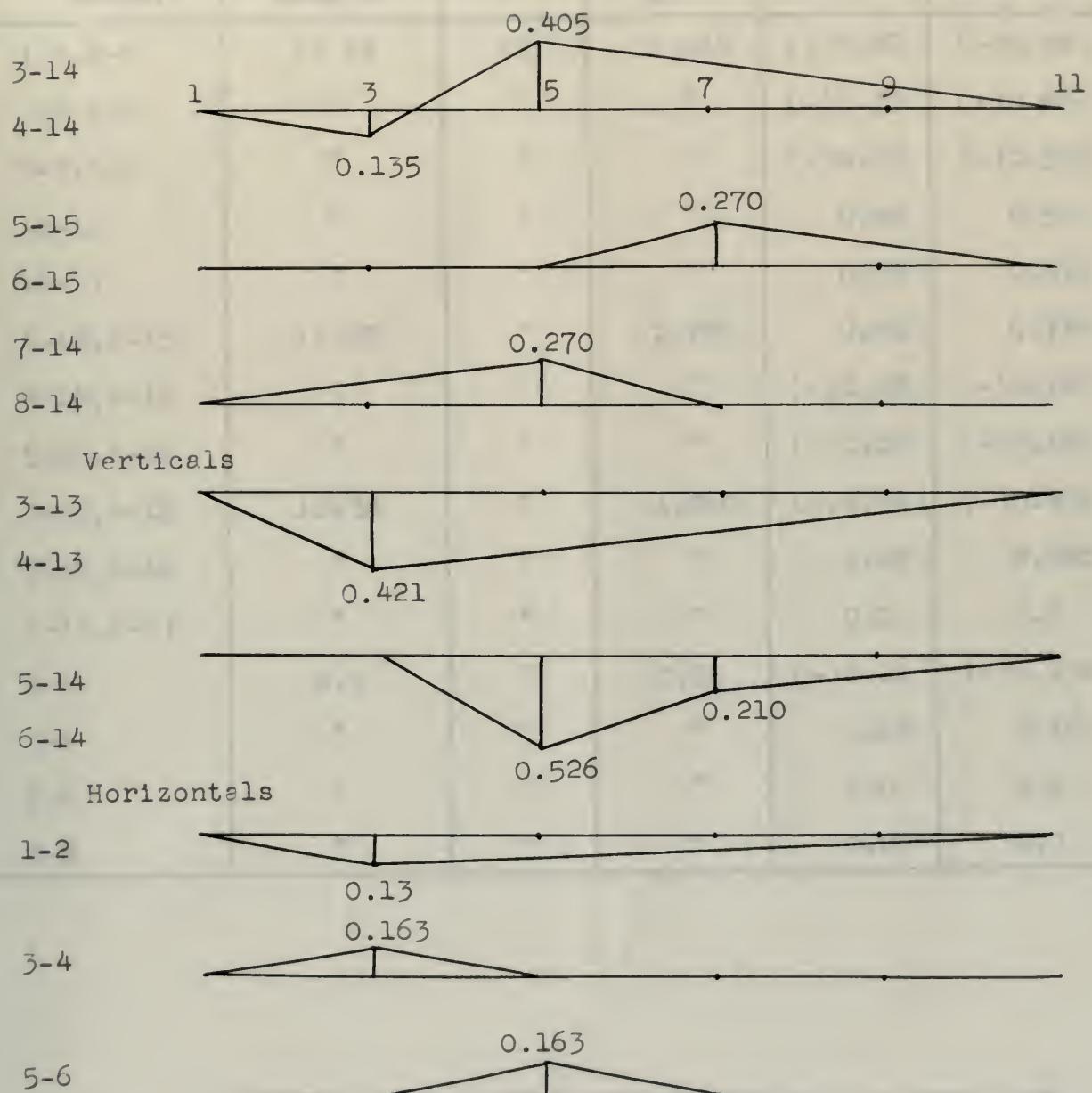
ENTRANCE

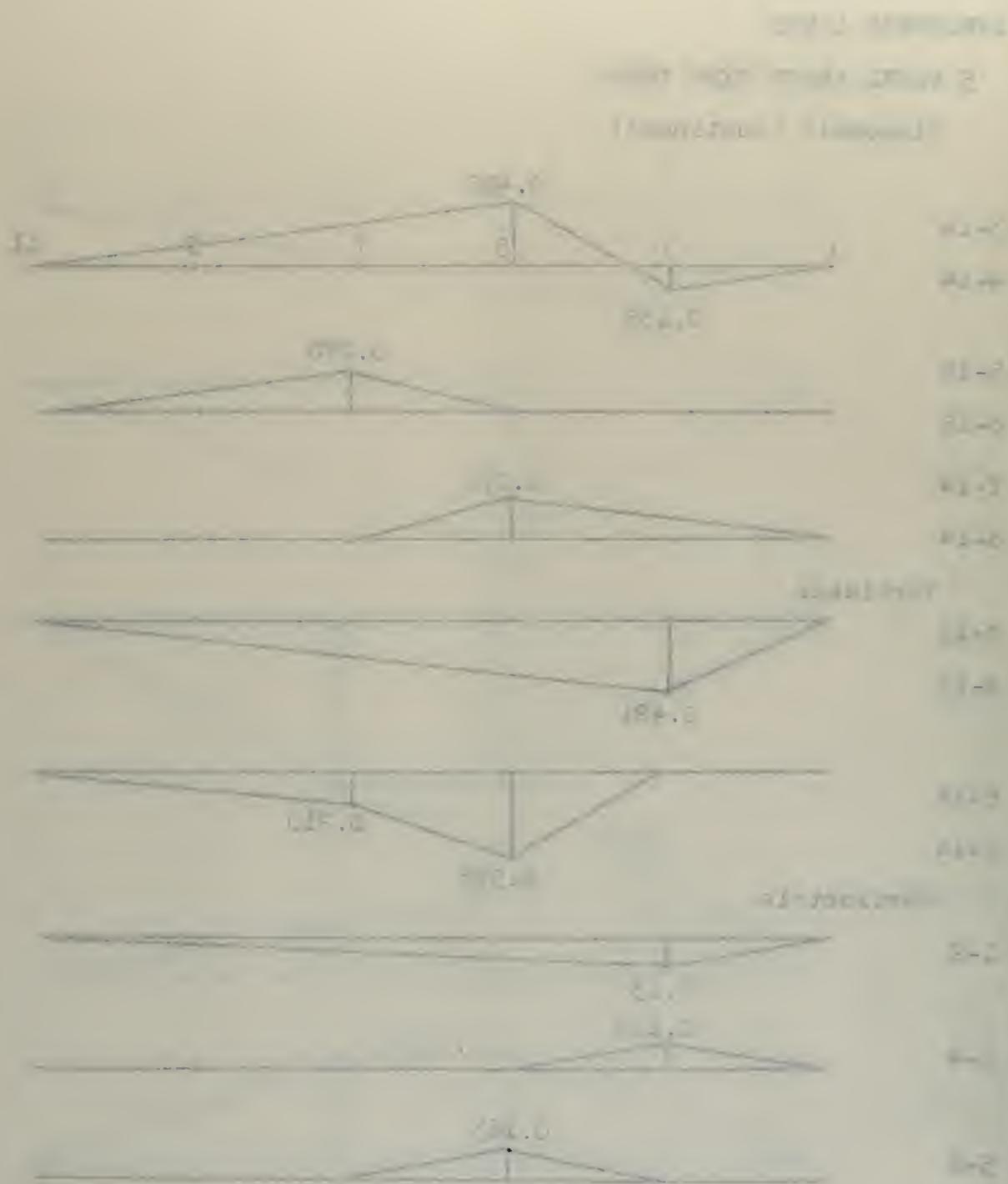


INFLUENCE LINES

5 PANEL PRATT TYPE TRUSS

Diagonals (continued)





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

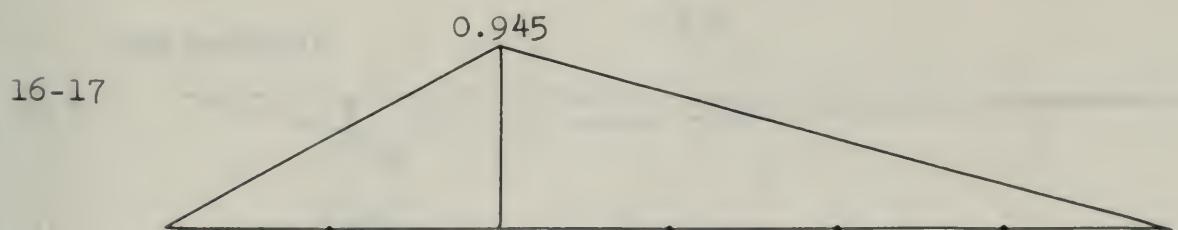
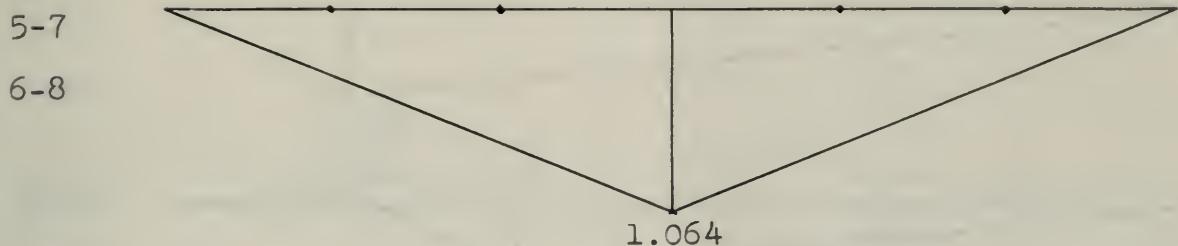
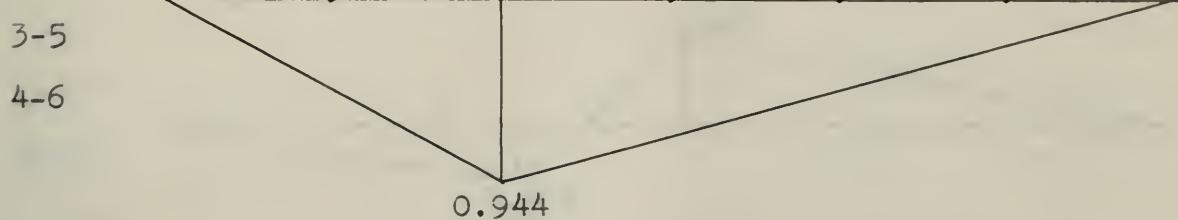
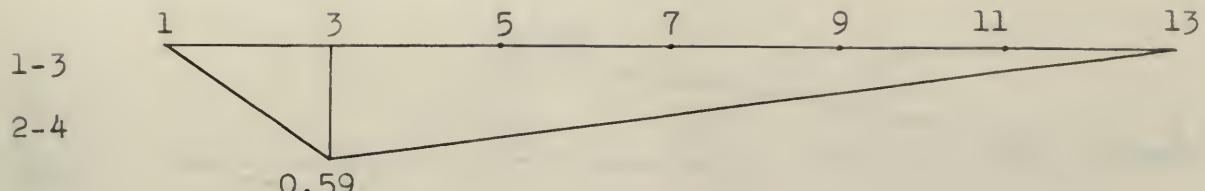
6 Panel Pratt Type Truss Load at 7-8

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



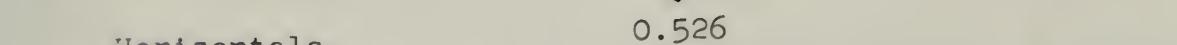
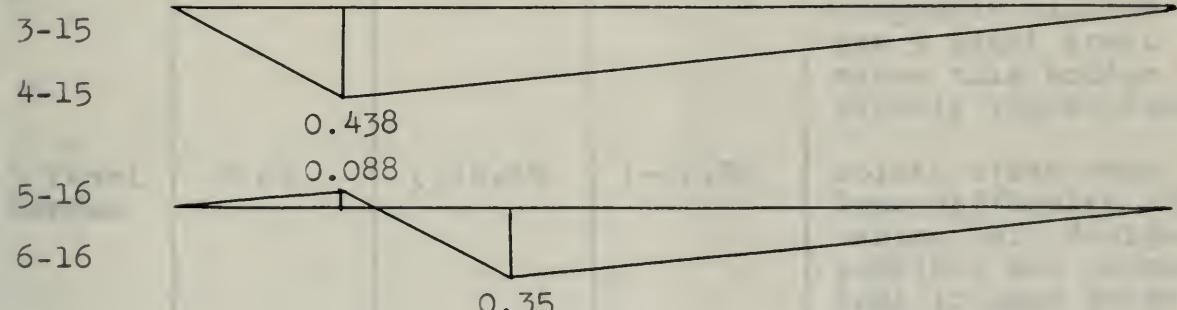
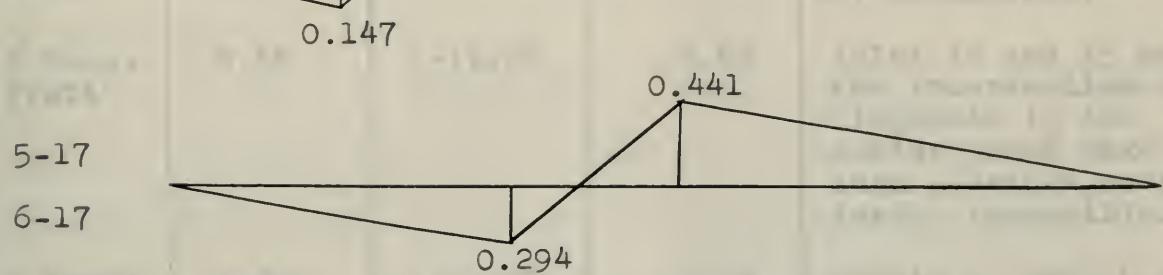
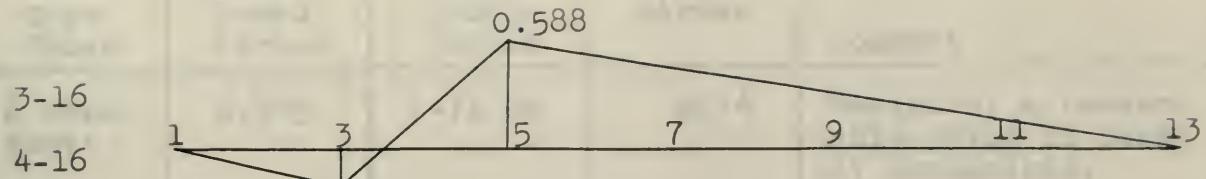
Diagonals



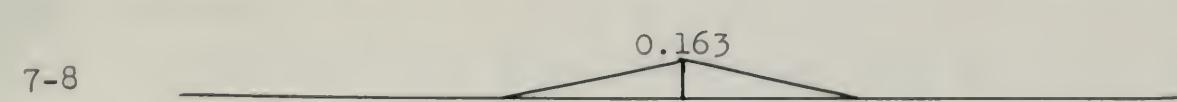
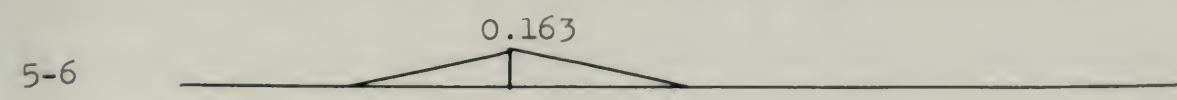
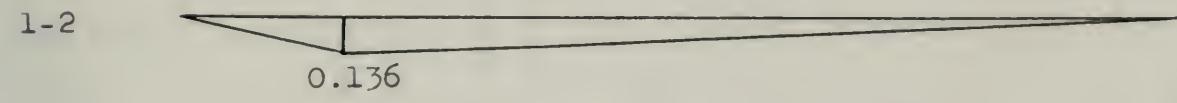
INFLUENCE LINES

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 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 $2\frac{1}{2}$ times that of the single bottom chord, the final analysis shows the Warren only 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

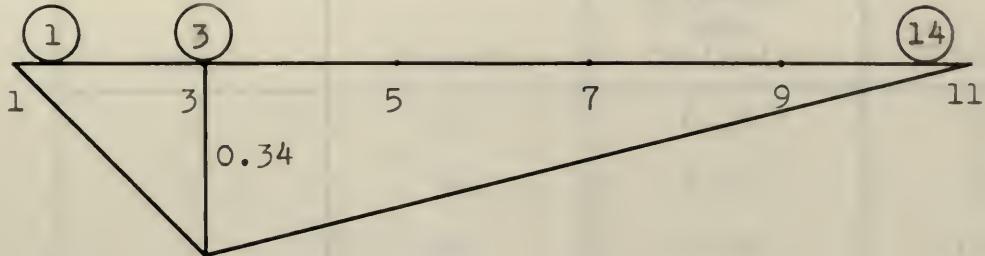
The economical if not structural impossibility of developing joints for the Howe and Pratt bridges, however, made the Warren bridge the only possible type to use. The fact that its apparent lack of economy is only slight has been stressed only to indicate that if the Howe or Pratt bridges were used,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

Following are the computations for the stresses in all members:

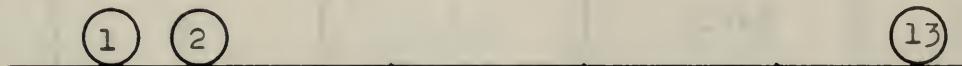
WARREN TYPE SPACE FRAME RAILWAY BRIDGE

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



$$\text{Loss} = 45(5)(0.02) = 4.5$$

$$\text{Gain} = 303(5)(0.0005) = 7.58$$

Position III: Wheel "4" at point 3



$$\text{Loss} = 75(5)(0.02) = 7.5$$

$$\text{Gain} = 273(5)(0.005) = 6.82$$

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

$$\text{Stress} = 2(60.2)(1.2) = (-)144.5 \text{ K.}$$

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

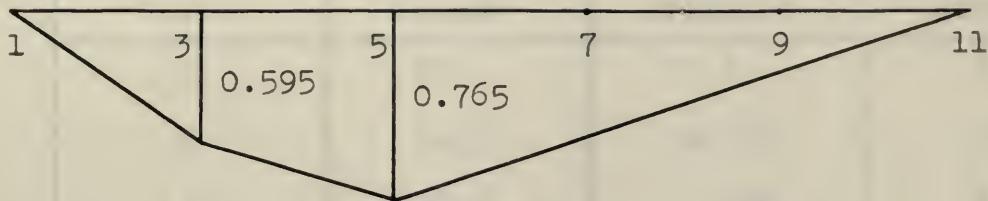
$$\text{Stress} = 0.5(0.34)(85)(8.34)(1.2) = (-)144.5 \text{ K.}$$

Species	Mean size (mm)	Mean age (years)	Mean sex ratio
<i>S. macrourus</i>	100.5	1.6	0.6
<i>S. canaliculatus</i>	90.0	1.6	0.6
<i>S. squalidus</i>	77.0	1.6	0.6
<i>S. aculeatus</i>	62.0	1.6	0.6
<i>S. leucostomus</i>	57.0	1.6	0.6
<i>S. punctifer</i>	50.0	1.6	0.6
<i>S. squalidus</i>	46.0	1.6	0.6
<i>S. canaliculatus</i>	44.0	1.6	0.6
<i>S. aculeatus</i>	43.0	1.6	0.6
<i>S. leucostomus</i>	42.0	1.6	0.6
<i>S. punctifer</i>	41.0	1.6	0.6
<i>S. macrourus</i>	39.0	1.6	0.6
<i>S. canaliculatus</i>	38.0	1.6	0.6
<i>S. squalidus</i>	37.0	1.6	0.6
<i>S. aculeatus</i>	36.0	1.6	0.6
<i>S. leucostomus</i>	35.0	1.6	0.6
<i>S. punctifer</i>	34.0	1.6	0.6
<i>S. macrourus</i>	33.0	1.6	0.6
<i>S. canaliculatus</i>	32.0	1.6	0.6
<i>S. squalidus</i>	31.0	1.6	0.6
<i>S. aculeatus</i>	30.0	1.6	0.6
<i>S. leucostomus</i>	29.0	1.6	0.6
<i>S. punctifer</i>	28.0	1.6	0.6
<i>S. macrourus</i>	27.0	1.6	0.6
<i>S. canaliculatus</i>	26.0	1.6	0.6
<i>S. squalidus</i>	25.0	1.6	0.6
<i>S. aculeatus</i>	24.0	1.6	0.6
<i>S. leucostomus</i>	23.0	1.6	0.6
<i>S. punctifer</i>	22.0	1.6	0.6
<i>S. macrourus</i>	21.0	1.6	0.6
<i>S. canaliculatus</i>	20.0	1.6	0.6
<i>S. squalidus</i>	19.0	1.6	0.6
<i>S. aculeatus</i>	18.0	1.6	0.6
<i>S. leucostomus</i>	17.0	1.6	0.6
<i>S. punctifer</i>	16.0	1.6	0.6
<i>S. macrourus</i>	15.0	1.6	0.6
<i>S. canaliculatus</i>	14.0	1.6	0.6
<i>S. squalidus</i>	13.0	1.6	0.6
<i>S. aculeatus</i>	12.0	1.6	0.6
<i>S. leucostomus</i>	11.0	1.6	0.6
<i>S. punctifer</i>	10.0	1.6	0.6
<i>S. macrourus</i>	9.0	1.6	0.6
<i>S. canaliculatus</i>	8.0	1.6	0.6
<i>S. squalidus</i>	7.0	1.6	0.6
<i>S. aculeatus</i>	6.0	1.6	0.6
<i>S. leucostomus</i>	5.0	1.6	0.6
<i>S. punctifer</i>	4.0	1.6	0.6
<i>S. macrourus</i>	3.0	1.6	0.6
<i>S. canaliculatus</i>	2.0	1.6	0.6
<i>S. squalidus</i>	1.0	1.6	0.6
<i>S. aculeatus</i>	0.5	1.6	0.6
<i>S. leucostomus</i>	0.2	1.6	0.6
<i>S. punctifer</i>	0.1	1.6	0.6

Table 2 Mean size, mean age at first maturity, and the mean sex ratio for each species. The mean sex ratio was calculated from the proportion of males in each age class. The mean age at first maturity was calculated from the age at which 50% of the population had reached sexual maturity (i.e., 50% of the population was female)

LIVE LOAD STRESSES

Members 3-5 and 4-6



Position I: Wheel "12" reversed at 5



Position II: Wheel "13" reversed at 5



$$\text{Gain} = 58.5(0.035)(5) + 19.5(0.01)(5) = 14.2$$

$$\text{Loss} = 213(0.015)(5) = 16.0$$

Position III: Wheel "14" reversed at 5



$$\text{Gain} = 39(0.035)(5) + 69(0.01)(5) = 10.3$$

$$\text{Loss} = 213(0.015)(5) = 16.0$$

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

$$\text{Stress} = (-)2(142.0)(1.2) = (-)341 \text{ K.}$$

Uniform Load = 8030 lbs./ft. of track

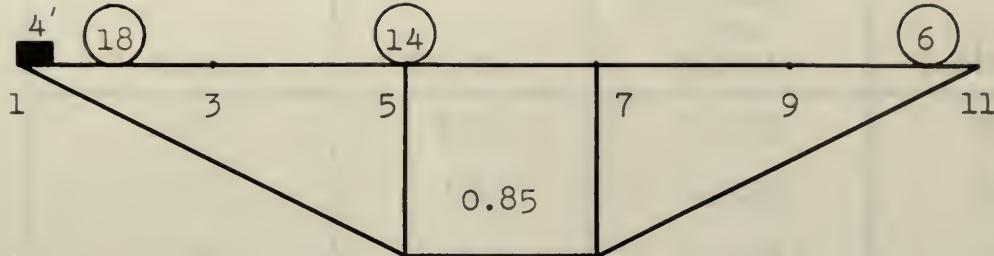
$$\begin{aligned} \text{Stress} = 1.2 & \left[(-)0.5(17)(8.03)(0.595) - 0.5(51)(0.765)(8.03) \right. \\ & \left. - 0.5(0.595 + 0.765)(8.03)(17) \right] = (-)347 \text{ K.} \end{aligned}$$

Number	Specimen No. & Date	Length mm.	Width mm.
1	1000 2	78	8
2	1002 2	69.2	8
3	1003 2	77.8	7
4	1004 2	74.2	6
5	1005 2	74.8	6
6	1006 2	70	6
7	1007 2	70	7
8	1008 2	70	6
9	1009 2	70	6
10	1010 2	70	6
11	1011 2	70	6
12	1012 2	70	6
13	1013 2	70	6
14	1014 2	70	6

Specimens 1000-1014 were collected from the same area as specimens 1001-1009, but were taken at a slightly lower elevation (1000-1014, 1000-1009 = 1000-1009). The mean length of 1000-1014 = 70 mm., and the mean width = 6.1 mm.

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

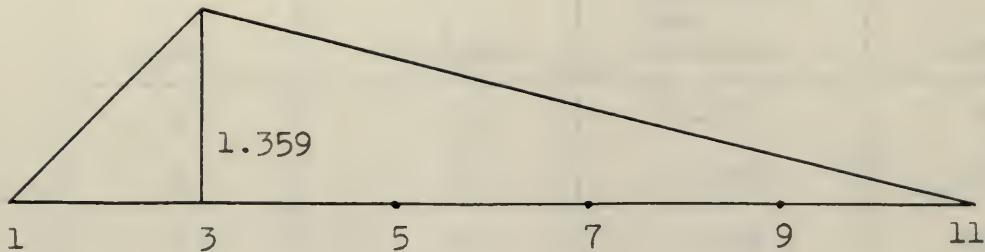
$$\text{Stress} = 2(170.0)(1.2) = (-)408 \text{ K.}$$

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

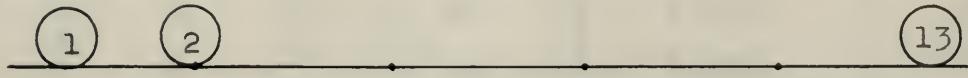
$$\begin{aligned} \text{Stress} &= [1.2] [(-)(0.5)(2)(34)(0.85)(8.0) - (0.85)(17)(8.0)] \\ &= (-)416 \text{ K.} \end{aligned}$$

LIVE LOAD STRESSES

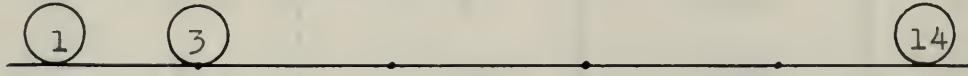
Member 13-14



Position I: Wheel "2" at point 3



Position II: Wheel "3" at 3



$$\text{Gain} = 5(0.01995)(303) = 30.2$$

$$\text{Loss} = 5(0.0799)(45) = 18.0$$

Position III: Wheel "4" at 3



$$\text{Gain} = 5(0.01995)(273) = 27.2$$

$$\text{Loss} = 5(0.0799)(75) = 30.0$$

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

$$\text{Stress} = 2(240.2)(1.2) = 576 \text{ K.}$$

$$\text{Equivalent Uniform Load} = 8.33 \text{ K./ft. of track}$$

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

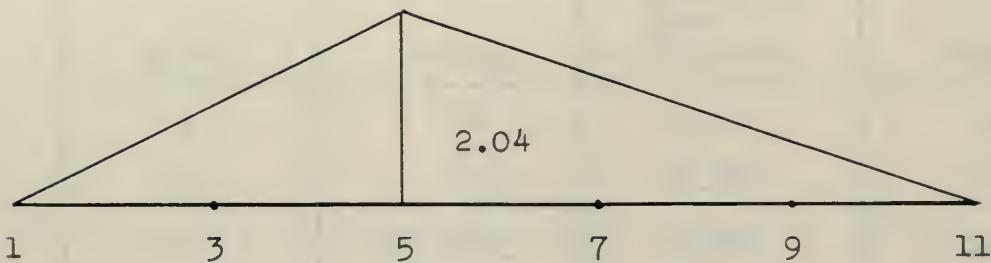
Year	Population	Area	Density
1850	50,000	24	2,083
1860	60,000	28	2,143
1870	60,000	30	2,000
1880	65,000	31	2,097
1890	70,000	32	2,188
1900	75,000	33	2,273
1910	80,000	34	2,353
1920	85,000	35	2,429
1930	90,000	36	2,500
1940	95,000	37	2,568
1950	100,000	38	2,632
1960	105,000	39	2,718
1970	110,000	40	2,750
1980	115,000	41	2,805
1990	120,000	42	2,857
2000	125,000	43	2,884

Source: U.S. Census Bureau

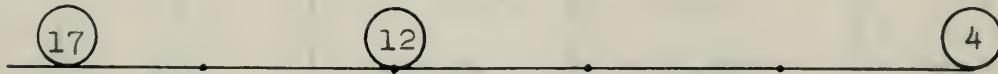
Population (in thousands) = annual
mean; area (in square miles) = annual
mean; density = population/area.

LIVE LOAD STRESSES

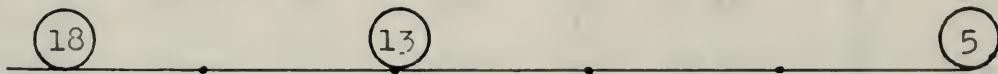
Member 14-15



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

$$\text{Stress} = 2(346.4)(1.2) = 831 \text{ K.}$$

$$\text{Equivalent Uniform Load} = 8.04 \text{ K./ft. of track}$$

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

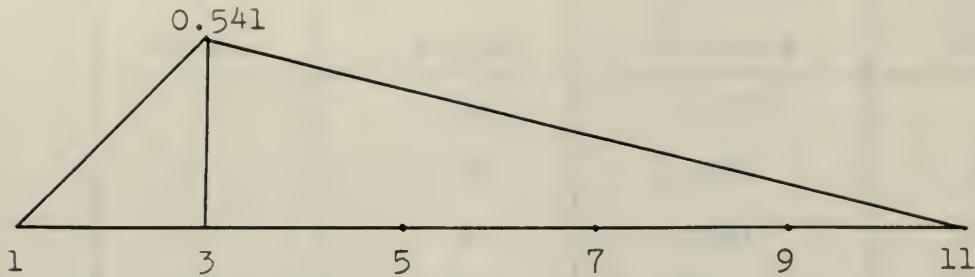
Location	Geographical coordinates	Depth (m)	Latitude
1.0	9.3	10	9
2.0	0.0	2.01	6
3.04	002.0	2.61	7
3.05	007.0	2.61	7
3.06	07.0	2.61	8
3.07	00.0	2.61	81
3.08	00.0	26	8
3.09	A6.0	26	81
3.10	00.0	26	81
3.11	00.0	26	81
3.12	A1.0	26	81
3.13	00.0	26	81
3.14	00.0	2.61	81
3.15	00.0	2.61	81
3.16	00.0	2.61	81
3.17	00.0	2.61	81
3.18	00.0	2.61	81

Depth = m

$\pm 2.08 \times 10^{-10} \text{ s}^{-1} \text{ m}^{-1}$ (2000–2005) ± 0.0001
 $\pm 0.0002 \text{ s}^{-1} \text{ m}^{-1}$ (2006–2007) ± 0.0001
 $\pm 0.0002 \text{ s}^{-1} \text{ m}^{-1}$ (2008–2009)

LIVE LOAD STRESSES

Members 1-13 and 2-13



Position I: Wheel "2" at point 3



Position II: Wheel "3" at point 3



$$\text{Gain} = 5(0.000796)(303) = 9.9$$

$$\text{Loss} = 5(0.0319)(45) = 5.9$$

Position III: Wheel "4" at point 3



$$\text{Gain} = 5(0.00796)(273) = 9.0$$

$$\text{Loss} = 5(0.0319)(75) = 9.9$$

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

$$\text{Stress} = 2(96.4)(1.2) = 231 \text{ K.}$$

$$\text{Equivalent Uniform Load} = 8.33 \text{ K./ft. of track}$$

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

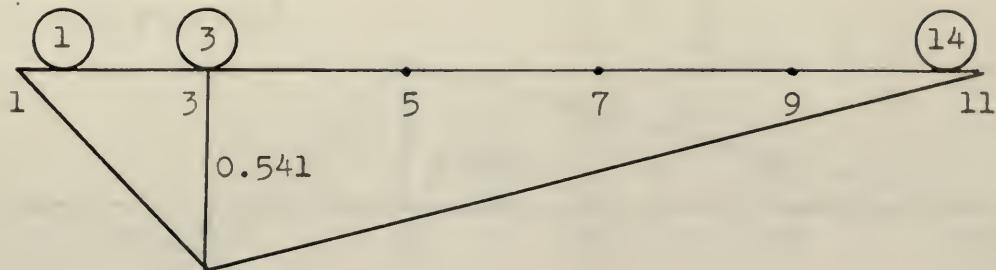
Author	Author's number	Page	Page
C. C.	2122	23	L
C. C.	28110	26	2
C. C.	14011	28	11
C. C.	20910	29	4
C. C.	20912	30	2
C. C.	21	291	18
C. C.	20914	291	7
C. C.	20915	292	1
C. C.	20916	293	19
C. C.	20917	294	34
C. C.	20918	295	12
C. C.	20919	296	11
C. C.	20920	297	11

20918-20920

10. J. S. C. (1972) 16, 76-78 & plate 7
 (cont'd. from p. 76) - *Calostoma* (Lamouroux)
 (2) (2) = *C. ciliatum* (Lamouroux) + *C. fimbriatum*

LIVE LOAD STRESSES

Members 3-13 and 4-13

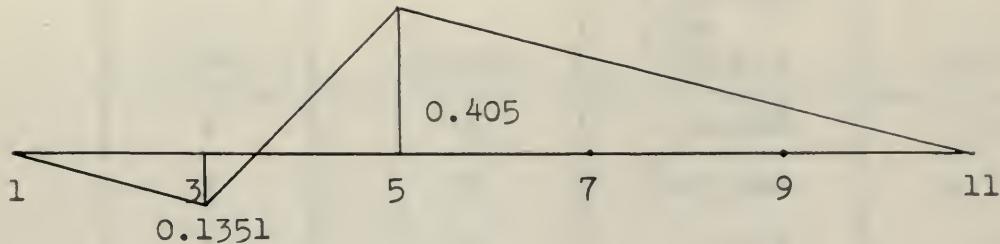


Place wheel "3" at point 3 (Same as loading for 1-13 and 2-13)

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

LIVE LOAD STRESSES

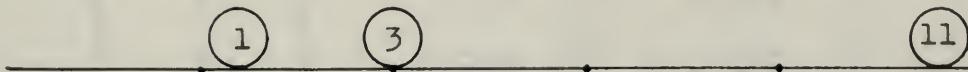
Member 3-14 and 4-14



Position I: Wheel "2" at point 5



Position II: Wheel "3" at point 5



$$\text{Gain: } = 5(183)(0.00795) = 6.02$$

$$\text{Loss: } = 5(45)(0.0315) = 5.91$$

Position III: Wheel "4" at point 5



$$\text{Gain: } = 183(5)(0.00795) = 6.02$$

$$\text{Loss: } = 75(5)(0.0315) = 11.8$$

Therefore use Position II

W. C. GANNON

1000' from the mouth



Distance to 100' mouth of stream



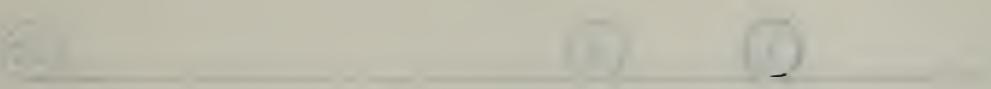
Distance to 100' mouth of stream



Distance to 100' mouth of stream

Distance to 100' mouth of stream

Distance to 100' mouth of stream



Distance to 100' mouth of stream

Distance to 100' mouth of stream

Distance to 100' mouth of stream

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

$$\text{Stress} = 2(56.1)(1.2) = 134.6 \text{ K.}$$

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

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

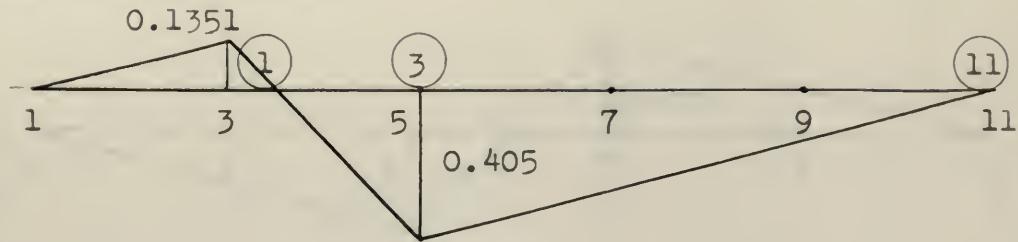
Year	Estimated yield tonnes	Actual	Length
1970(-)	250,000(+)	241	4
1971(+)	380,000(+)	371	3
1972	360,000	361	3
1973	360,000	361	3
1974	360,000	361	3
1975	420,000	418.4	3
1976	420,000	420.2	3
1977	420,000	420.1	3
1978	420,000	420.1	3
1979	420,000	420.1	3
1980	420,000	420.1	3

1,000 = 100

Estimated yield for 1970-71 was 250,000 tonnes.
 Actual yield for 1971 was 371,000 tonnes. This is based
 on Total = (250,000)(21.2)(1.12)(1.02)(1.01) = 371,000.

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.

WATERFALLS AND RIVERS



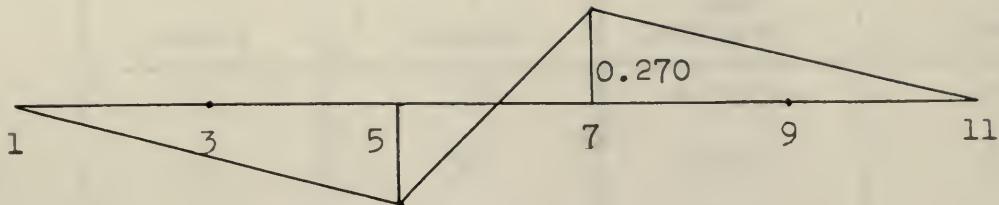
WATERFALLS AND RIVERS

WATERFALLS AND RIVERS

WATERFALLS AND RIVERS

LIVE LOAD STRESSES

Members 5-15 and 6-15

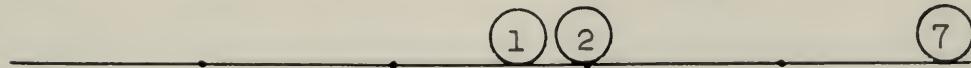


For Tension

Position I: Wheel "1" at point 7



Position II: Wheel "2" at point 7



$$\text{Gain} = 8(0.00795)(159) = 10.1$$

$$\text{Loss} = 8(0.0318)(15) = 3.8$$

Position III: Wheel "3" at point 7



$$\text{Gain} = 5(0.00795)(148.5) = 5.9$$

$$\text{Loss} = 5(0.0318)(45) = 7.15$$

Therefore use Position II

For Compression

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

1990-1991

1990-1991



1990-1991

1990-1991



1990-1991



1990-1991

1990-1991

1990-1991



1990-1991

1990-1991

1990-1991



1990-1991

1990-1991

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

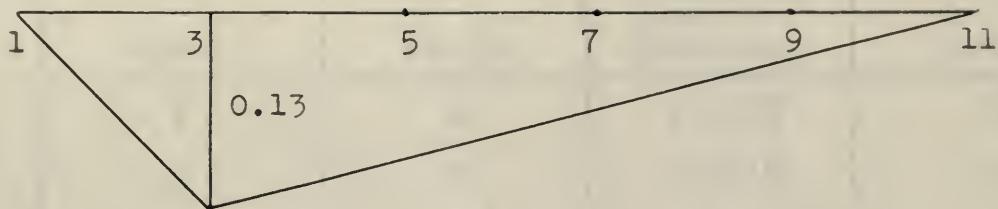
$$\text{Stress} = 2(27.8)(1.2) = (\pm)66.7 \text{ K.}$$

$$\text{Equivalent Uniform Load} = 9880 \text{ lbs./ft. of track}$$

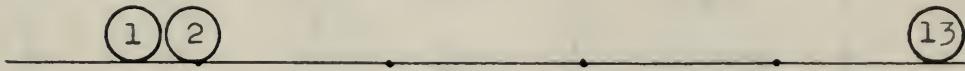
$$\text{Stress} = 0.5(0.270)(42.5)(9.88)(1.2) = (\pm)68 \text{ K.}$$

LIVE LOAD STRESSES

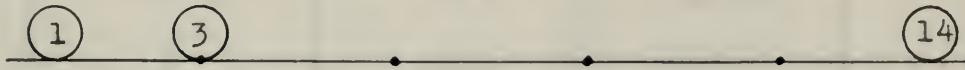
Member 1-2



Position I: Wheel "2" at point 3



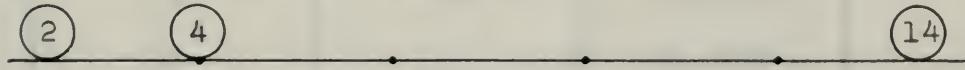
Position II: Wheel "3" at point 3



$$\text{Gain} = 45(5)(0.00765) = 1.72$$

$$\text{Loss} = 303(5)(0.001911) = 2.89$$

Position III: Wheel "4" at point 3



$$\text{Gain} = 273(5)(0.001911) = 2.62$$

$$\text{Loss} = 75(5)(0.00765) = 2.87$$

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

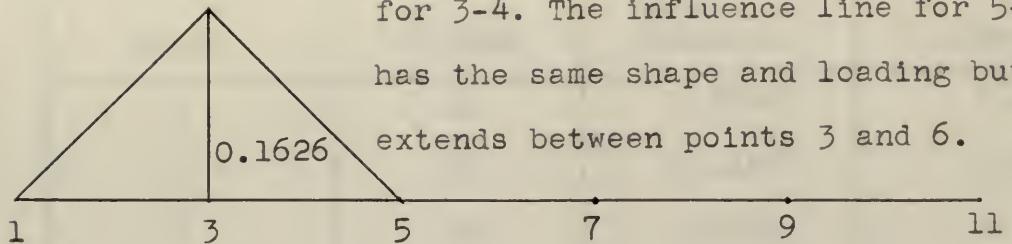
$$\text{Stress} = 2(22.9)(1.2) = (-)55.0 \text{ K.}$$

$$\text{Equivalent Uniform Load} = 8330 \text{ lbs./ft. of track}$$

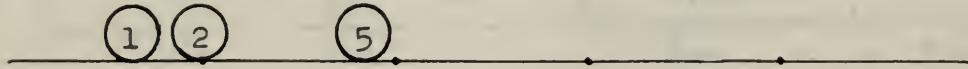
$$\text{Stress} = 0.5(85)(0.13)(8.33)(1.2) = (-)55.3 \text{ K.}$$

LIVE LOAD STRESSES

Members 3-4 and 5-6



Position I: Wheel "2" at point 3



Position II: Wheel "3" at point 3



$$\text{Gain} = 5(0.00958)(90) = 4.31$$

$$\text{Loss} = 5(0.00958)(45) = 2.15$$

Position III: Wheel "4" at point 3



$$\text{Gain} = 5(0.00958)(60) + 3(0.00958)(19.5) = 3.44$$

$$\text{Loss} = 5(0.00958)(60) + 4(0.00958)(15) = 3.46$$

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

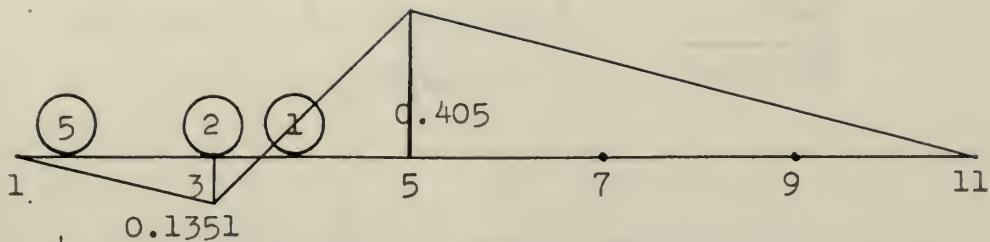
$$\text{Stress} = 2(14.3)(1.2) = 34.3 \text{ K.}$$

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

$$\text{Stress} = 0.5(34)(0.1626)(10.52)(1.2) = 34.9 \text{ K.}$$

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

$$\text{Stress} = 2(7.4)(1.2) = (-)17.75 \text{ K.}$$

$$\text{Dead load stress} = (+)20.3 \text{ K.}$$

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

$$\begin{array}{lll} \text{Vertical: } \% = 100 - 0.6(85) & = 49.0\% \\ \text{Rolling } \% = 2(10)5/6.5 & = 15.4\% \\ \hline \text{Total} & & = 64.4\% \end{array}$$

DEAD LOAD STRESSES

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

$$\text{Total Assumed Dead Load} = 1.76 \text{ K./ ft.}$$

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

ITEM	UNIT	AMOUNT	PERCENT	ITEM	UNIT	AMOUNT	PERCENT
6	%	27.0	100.0000%	10	MMB	1000000	1000000000
7	%	27.0	100.0000%	11	MMB	1000000	1000000000
8	%	27.0	100.0000%	12	MMB	1000000	1000000000
9	%	27.0	100.0000%	13	MMB	1000000	1000000000
10	%	27.0	100.0000%	14	MMB	1000000	1000000000
11	%	27.0	100.0000%	15	MMB	1000000	1000000000
12	%	27.0	100.0000%	16	MMB	1000000	1000000000
13	%	27.0	100.0000%	17	MMB	1000000	1000000000
14	%	27.0	100.0000%	18	MMB	1000000	1000000000
15	%	27.0	100.0000%	19	MMB	1000000	1000000000
20	%	27.0	100.0000%	21	MMB	1000000	1000000000
21	%	27.0	100.0000%	22	MMB	1000000	1000000000
23	%	27.0	100.0000%	24	MMB	1000000	1000000000
25	%	27.0	100.0000%	26	MMB	1000000	1000000000
27	%	27.0	100.0000%	28	MMB	1000000	1000000000
29	%	27.0	100.0000%	30	MMB	1000000	1000000000
31	%	27.0	100.0000%	32	MMB	1000000	1000000000
33	%	27.0	100.0000%	34	MMB	1000000	1000000000
35	%	27.0	100.0000%	36	MMB	1000000	1000000000
37	%	27.0	100.0000%	38	MMB	1000000	1000000000
39	%	27.0	100.0000%	40	MMB	1000000	1000000000
41	%	27.0	100.0000%	42	MMB	1000000	1000000000
43	%	27.0	100.0000%	44	MMB	1000000	1000000000
45	%	27.0	100.0000%	46	MMB	1000000	1000000000
47	%	27.0	100.0000%	48	MMB	1000000	1000000000
49	%	27.0	100.0000%	50	MMB	1000000	1000000000
51	%	27.0	100.0000%	52	MMB	1000000	1000000000
53	%	27.0	100.0000%	54	MMB	1000000	1000000000
55	%	27.0	100.0000%	56	MMB	1000000	1000000000
57	%	27.0	100.0000%	58	MMB	1000000	1000000000
59	%	27.0	100.0000%	60	MMB	1000000	1000000000
61	%	27.0	100.0000%	62	MMB	1000000	1000000000
63	%	27.0	100.0000%	64	MMB	1000000	1000000000
65	%	27.0	100.0000%	66	MMB	1000000	1000000000
67	%	27.0	100.0000%	68	MMB	1000000	1000000000
69	%	27.0	100.0000%	70	MMB	1000000	1000000000
71	%	27.0	100.0000%	72	MMB	1000000	1000000000
73	%	27.0	100.0000%	74	MMB	1000000	1000000000
75	%	27.0	100.0000%	76	MMB	1000000	1000000000
77	%	27.0	100.0000%	78	MMB	1000000	1000000000
79	%	27.0	100.0000%	80	MMB	1000000	1000000000
81	%	27.0	100.0000%	82	MMB	1000000	1000000000
83	%	27.0	100.0000%	84	MMB	1000000	1000000000
85	%	27.0	100.0000%	86	MMB	1000000	1000000000
87	%	27.0	100.0000%	88	MMB	1000000	1000000000
89	%	27.0	100.0000%	90	MMB	1000000	1000000000
91	%	27.0	100.0000%	92	MMB	1000000	1000000000
93	%	27.0	100.0000%	94	MMB	1000000	1000000000
95	%	27.0	100.0000%	96	MMB	1000000	1000000000
97	%	27.0	100.0000%	98	MMB	1000000	1000000000
99	%	27.0	100.0000%	100	MMB	1000000	1000000000

TABLE 2. RIVER SYSTEMS IN THE UNITED STATES

STATE	STATE NUMBER	STATE NAME	STATE NUMBER	STATE NAME	STATE NUMBER	STATE NAME	STATE NUMBER	STATE NAME
1	1	ALASKA	2	AK	3	ALABAMA	4	AL
5	5	ARIZONA	6	AZ	7	ARKANSAS	8	AR
9	9	CALIFORNIA	10	CA	11	CONNECTICUT	12	CT
13	13	DELAWARE	14	DE	15	FLORIDA	16	FL
17	17	GEORGIA	18	GA	19	HAWAII	20	HI
21	21	IDAHO	22	ID	23	ILLINOIS	24	IL
25	25	KANSAS	26	KS	27	KENTUCKY	28	KY
29	29	LAWRENCE	30	LA	31	MAINE	32	ME
33	33	MARYLAND	34	MD	35	MASSACHUSETTS	36	MA
37	37	MISSOURI	38	MO	39	MISSISSIPPI	40	MS
41	41	NEVADA	42	NV	43	NEW HAMPSHIRE	44	NH
45	45	NEW JERSEY	46	NJ	47	NEW MEXICO	48	NM
49	49	NEW YORK	50	NY	51	NEW HAMPSHIRE	52	NH
53	53	PENNSYLVANIA	54	PA	55	PENNSYLVANIA	56	PA
57	57	PENNSYLVANIA	58	PA	59	PENNSYLVANIA	60	PA
61	61	PENNSYLVANIA	62	PA	63	PENNSYLVANIA	64	PA
65	65	PENNSYLVANIA	66	PA	67	PENNSYLVANIA	68	PA
69	69	PENNSYLVANIA	70	PA	71	PENNSYLVANIA	72	PA
73	73	PENNSYLVANIA	74	PA	75	PENNSYLVANIA	76	PA
77	77	PENNSYLVANIA	78	PA	79	PENNSYLVANIA	80	PA
81	81	PENNSYLVANIA	82	PA	83	PENNSYLVANIA	84	PA
85	85	PENNSYLVANIA	86	PA	87	PENNSYLVANIA	88	PA
89	89	PENNSYLVANIA	90	PA	91	PENNSYLVANIA	92	PA
93	93	PENNSYLVANIA	94	PA	95	PENNSYLVANIA	96	PA
97	97	PENNSYLVANIA	98	PA	99	PENNSYLVANIA	100	PA

LATERAL STRESSES

WIND

Assumed windage area = 40% Of vertical projection

$$\text{Windage area} = 0.40(10)(76.5) = 306 \text{ sq.ft.}$$

$$\text{Area per foot} = 306/85 = 3.6 \text{ sq.ft./ ft.}$$

LOADED BRIDGE:

$$\text{Assumed wind force} = 3.6(30) = 108 \text{ lbs./ ft.}$$

$$\text{Wind on train} = 300 \quad " \quad "$$

$$\text{Wind on loaded bridge} = 408 \quad " \quad "$$

UNLOADED BRIDGE:

$$\text{Wind force} = 50 \text{ lbs./ ft.}^2$$

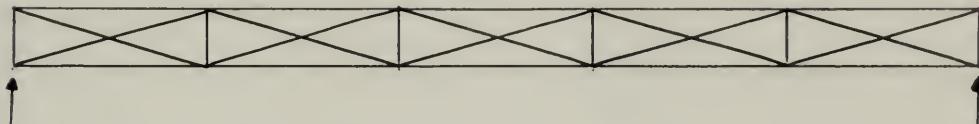
$$\text{Wind load} = 3.6(50) = 180 \text{ lbs./ ft.}$$

$$\text{Minimum load} = 200 + 150 = 350 \text{ lbs./ ft.}$$

$$\text{Therefore Design Wind Load} = 408 \text{ lbs./ ft.}$$

$$\text{PANEL CONCENTRATION} = 17(0.408 \text{ K./ ft.}) = 6.94 \text{ K.}$$

$$\text{NOSING} = 20 \text{ K.}$$



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

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

$$\text{Member 1-4} = 29.8(18.2)/6.5 = 83.4 \text{ K.}$$

$$\text{Member 3-4} = (-)29.8 \text{ K.}$$

$$\text{Member 1-3} = 29.8(17)/6.5 = (-)78 \text{ K.}$$

$$\text{Member 1-2} = 29.8/2 = (-)14.9 \text{ K.}$$

LATERAL STRESSES

MEMBERS 3-6 AND 5-6

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

$$\text{Member } 3-6 = 20.3(18.2)/6.5 = 56.8 \text{ K.}$$

$$\text{Member } 5-6 = (-)26.9 \text{ K.}$$

MEMBER 5-8 AND 7-8

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

$$\text{Member } 5-8 = 12.1(18.2)/6.5 = 34.0 \text{ K.}$$

$$\text{Member } 7-8 = (-)26.9 \text{ K.}$$

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

$$R = 13.8 + 8 = 21.8 \text{ K.}$$

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

the first time, and I am sure that it will be a long time before we have another such opportunity. It is a great privilege to be invited to speak at such a meeting, and I hope that my remarks will be of interest to all those present.

The first point I would like to make is that the situation in which we find ourselves is one of great uncertainty. We do not know exactly what the future holds, and we must therefore be prepared for whatever may happen. This is a difficult position to be in, but it is one that we must face if we are to survive.

The second point I would like to make is that we must be prepared to change our ways of working. We cannot rely on the same methods that we have used in the past, because they no longer work. We must be willing to experiment and to try new things, if we are to succeed.

The third point I would like to make is that we must be prepared to take risks. We cannot afford to be afraid of failure, because failure is a part of life. We must be willing to take risks, if we are to achieve our goals.

The fourth point I would like to make is that we must be prepared to work together. We cannot achieve our goals alone, we must work together as a team. This means that we must be willing to compromise, and to listen to the opinions of others.

The fifth point I would like to make is that we must be prepared to face the challenges of the future. We cannot afford to be complacent, because the world is changing rapidly. We must be prepared to face the challenges of the future, if we are to survive.

In conclusion, I would like to say that the situation in which we find ourselves is one of great uncertainty, but it is also one of great opportunity. We must be prepared to change our ways of working, to take risks, to work together, and to face the challenges of the future. If we do this, we will be successful.

LONGITUDINAL STRESSES

Members 1-3 and 2-4

$$\text{Braking: } 0.15(348) = 52.1 \text{ K.}$$

$$\text{Traction: } 0.25(240) = 60.0 \text{ K. (use)}$$

$$\text{Moment} = 1.2(60)(9) = 648 \text{ ft. K.}$$

$$R = 648/85 = 7.62 \text{ K.}$$

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

$$\text{Stress} = 1.2(60) = (-)72.0 \text{ K. (horizontal)}$$

$$\text{Total stress} = (-)78.5 \text{ K.}$$

Members 3-5 and 4-6

$$\text{Braking: } 0.15(321) = 48.1 \text{ K. (use)}$$

$$\text{Traction: } 0.25(150) = 45 \text{ K.}$$

$$\text{Moment} = 1.2(48.1)(9) = 518 \text{ ft.K.}$$

$$R = 518/85 = 6.1 \text{ K.}$$

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

$$\text{Stress} = 1.2(48.1) = (-)57.6 \text{ K. (horizontal)}$$

$$\text{Total stress} = (-)73.2 \text{ K.}$$

Members 5-7 and 6-8

$$\text{Braking: } 0.15(291) = 43.6 \text{ K. (use)}$$

$$\text{Traction: } 0.25(120) = 30 \text{ K.}$$

$$\text{Moment} = 1.2(43.6)(9) = 471 \text{ ft.K.}$$

$$R = 471/85 = 5.54 \text{ K.}$$

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

$$\text{Stress} = 1.2(43.6) = (-)52.2 \text{ K. (horizontal)}$$

$$\text{Total stress} = (-)75.8 \text{ K.}$$

LONGITUDINAL STRESSES (continued)

Member 13-14

$$\text{Braking} = 0.15(348) = 52.2 \text{ K.}$$

$$\text{Traction} = 0.25(240) = 60 \text{ K.}$$

$$\text{Moment} = 1.2(60)9 = 648 \text{ ft. K.}$$

$$R = 648/85 = 7.62 \text{ K.}$$

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

Member 14-15

$$\text{Braking} = 0.15(321) = 48.1 \text{ K.}$$

$$\text{Traction} = 0.25(150) = 37.5 \text{ K.}$$

$$\text{Moment} = 1.2(48.1)9 = 518 \text{ ft. K.}$$

$$R = 518/85 = 6.10 \text{ K.}$$

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

Member 1-13 and 2-13

$$\text{Moment} = 648 \text{ ft. K. (Same as 1-3 and 2-4)}$$

$$R = 7.62 \text{ K.}$$

$$\text{Stress} = 7.62(13.51)/10 = 10.3 \text{ K.}$$

Members 3-13 and 4-13

$$\text{Stress} = (-)10.3 \text{ K. (Same magnitude; opposite sign as 1-13 and 2-13)}$$

Members 3-14 and 4-14

$$\text{Braking} = 0.15(258) = 38.7 \text{ K}$$

$$\text{Traction} = 0.25(150) = 37.5 \text{ K.}$$

$$\text{Moment} = 1.2(9)38.7 = 418 \text{ ft. K.}$$

$$R = 418/85 = 4.92 \text{ K.}$$

$$\text{Stress} = 4.92(13.51)/10 = 6.65 \text{ K.}$$

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: $0.15(174) = 26.1 \text{ K.}$ Traction: $0.25(120) = 30 \text{ K. (use)}$ Moment = $1.2(30)9 = 324 \text{ ft. K.}$ $R = 324/85 = 3.82 \text{ K.}$ Stress = $3.82(13.51)/10 = (\pm)5.16 \text{ K.}$

Member 1-2

 $R = 7.62 \text{ K. (Same as 1-3 and 2-4)}$ 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 & 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	
5-14, 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	1043.0	
14-15	155.0	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	(-)75.2	(-)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

$$\text{Track } w = 200/2 = 100 \text{ lbs./ft.}$$

$$R = 8.5(100) = 850 \text{ lbs.}$$

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

$$R = 142.7(8.5) = 1212 \text{ lbs.}$$

$$\text{Guard Rail } w = 8(8)60/144 = 26.7 \text{ lbs./ft.}$$

$$R = 26.7(8.5) = 221 \text{ lbs.}$$

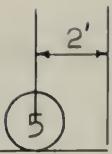
$$\text{Assumed Stringer } w = 300 \text{ lbs./ft.}$$

$$R = 300(8.5) = 2550 \text{ lbs.}$$

Total Dead Load Reaction

$$R(\text{total}) = 4839 \text{ lbs.} = 4.84 \text{ K.}$$

Live Load

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


$$R = 80.5 \text{ K.}$$

Impact

Direct Vertical and Rolling

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

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

$$\text{Total Impact} = 105.2\%$$

$$R(\text{Impact}) = 1.052(80.5) = 84.5 \text{ K.}$$

$$\text{TOTAL MAXIMUM STRINGER REACTION} = 84.5 + 4.84 + 80.5 = 169.8 \text{ K.}$$

DESIGN OF TOP CHORD

BEAM ACTION



Maximum Stringer Moment

Live Load

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

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

Dead Load

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

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

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

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

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

$$\text{Total Dead Load Moment} = 246.6 \text{ in. K.}$$

Impact

$$M = 1.052(3350) = 3530.0 \text{ in. K.}$$

$$\text{Total Maximum Stringer Moment} = 7127 \text{ in. K.}$$

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

$$\text{Sum} = 255.6$$

Flange 6	4(1.125)	17.5	306.0	1378.0
-------------	----------	------	-------	--------

$$\text{Total I of Rivets} = 4(255.6) + 4(1378) = 6534 \text{ in.}$$

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

$$\text{Net I} = 2(10470) - 6534 = 14406 \text{ in.}$$

MOMENT OF INERTIA OF MEMBER AT SECTION OF MAXIMUM STRESS

$$\text{Net I} = 2(10470) - 2(1378) = 18184 \text{ in.}$$

DESIGN OF TOP CHORD

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

$$M(\text{max.}) = 7127 \text{ in. K.}$$

$$\text{Maximum axial stress} = (-)751.0 \text{ Ksi.}$$

TRY 2-36 WF 170 SECTIONS

Flexure

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

$$f(\text{all.}) = 16.56 \text{ Ksi.}$$

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

Axial Load

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

$$f(\text{all.}) = 13.24 \text{ Ksi.}$$

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

$$\text{Total } f(\text{act.}) = 5.31 + 7.52 = 12.83 \text{ Ksi. Satisfactory}$$

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

$$f(\text{all.}) = 16.56 \text{ Ksi.} \quad M = 7127 \text{ in. K.}$$

$$f(\text{act.}) = 7127(18)/18184 = (-)7.07 \text{ Ksi.}$$

$$\text{Axial Load} \quad f(\text{all.}) = (-)13.24 \text{ Ksi.}$$

$$f(\text{act.}) = 336/2(49.98) = (-)3.37 \text{ Ksi.}$$

Combined Stresses

$$f(\text{act.}) = (-)(7.07 + 3.37) = (-)10.44 \text{ Ksi.}$$

$$f(\text{all.}) = (-)13.24 \text{ Ksi.}$$

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

$$\text{Sum} = 223.8$$

$$I \text{ of one row} = 2(223.8) = 447.6$$

$$I \text{ of one row in both WF sections} = 2(447.6) = 895.2$$

Axial Stress

$$\text{Net area of one web} = 11/16(34) - 10(0.688) = 16.47$$

$$\text{Total net area} = 2(49.98 - 6.88 - 9.00) = 68.20$$

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

Bending Stress

$$I \text{ of web} = 2(2251.8) = 4503.6$$

$$f(\text{all.}) = 18 \text{ ksi.} \quad c = 17$$

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

Shear Stress

$$\text{Value of web} = 11/16(34)(11) = 257 \text{ K.}$$

and 1.000 ml. 10% DMSO

100 µl. 10% DMSO

100 µl. 10% DMSO

and 100 µl. 10% DMSO and 75 µl. complete medium.

Time	µM	µM	µM
0	0	0	0
10 min	0.05	0	0
20 min	0.10	0	0
30 min	0.15	0	0
40 min	0.20	0	0
50 min	0.25	0	0
60 min	0.30	0	0
70 min	0.35	0	0
80 min	0.40	0	0
90 min	0.45	0	0
100 min	0.50	0	0
110 min	0.55	0	0
120 min	0.60	0	0
130 min	0.65	0	0
140 min	0.70	0	0
150 min	0.75	0	0
160 min	0.80	0	0
170 min	0.85	0	0
180 min	0.90	0	0
190 min	0.95	0	0
200 min	1.00	0	0
210 min	1.05	0	0
220 min	1.10	0	0
230 min	1.15	0	0
240 min	1.20	0	0
250 min	1.25	0	0
260 min	1.30	0	0
270 min	1.35	0	0
280 min	1.40	0	0
290 min	1.45	0	0
300 min	1.50	0	0
310 min	1.55	0	0
320 min	1.60	0	0
330 min	1.65	0	0
340 min	1.70	0	0
350 min	1.75	0	0
360 min	1.80	0	0
370 min	1.85	0	0
380 min	1.90	0	0
390 min	1.95	0	0
400 min	2.00	0	0
410 min	2.05	0	0
420 min	2.10	0	0
430 min	2.15	0	0
440 min	2.20	0	0
450 min	2.25	0	0
460 min	2.30	0	0
470 min	2.35	0	0
480 min	2.40	0	0
490 min	2.45	0	0
500 min	2.50	0	0
510 min	2.55	0	0
520 min	2.60	0	0
530 min	2.65	0	0
540 min	2.70	0	0
550 min	2.75	0	0
560 min	2.80	0	0
570 min	2.85	0	0
580 min	2.90	0	0
590 min	2.95	0	0
600 min	3.00	0	0
610 min	3.05	0	0
620 min	3.10	0	0
630 min	3.15	0	0
640 min	3.20	0	0
650 min	3.25	0	0
660 min	3.30	0	0
670 min	3.35	0	0
680 min	3.40	0	0
690 min	3.45	0	0
700 min	3.50	0	0
710 min	3.55	0	0
720 min	3.60	0	0
730 min	3.65	0	0
740 min	3.70	0	0
750 min	3.75	0	0
760 min	3.80	0	0
770 min	3.85	0	0
780 min	3.90	0	0
790 min	3.95	0	0
800 min	4.00	0	0
810 min	4.05	0	0
820 min	4.10	0	0
830 min	4.15	0	0
840 min	4.20	0	0
850 min	4.25	0	0
860 min	4.30	0	0
870 min	4.35	0	0
880 min	4.40	0	0
890 min	4.45	0	0
900 min	4.50	0	0
910 min	4.55	0	0
920 min	4.60	0	0
930 min	4.65	0	0
940 min	4.70	0	0
950 min	4.75	0	0
960 min	4.80	0	0
970 min	4.85	0	0
980 min	4.90	0	0
990 min	4.95	0	0
1000 min	5.00	0	0

Conc. = 0.05 mM DMSO and 5.0 µM

100 µl. 10% DMSO + complete DMEM + 5% FBS + 100 µl.

DATA ANALYSIS

GraphPad Prism 5.01 was used to fit dose-response curves to the data. The sigmoidal curve was fitted with the equation:

$$Y = \frac{1}{1 + (\frac{X - X_0}{K})^n}$$

where Y = response, X = concentration, X_0 = EC₅₀, K = EC₅₀ half-maximal effect, and n = Hill coefficient.

The sigmoidal curve was fitted with the equation:

$Y = \frac{1}{1 + (\frac{X - X_0}{K})^n}$

where Y = response, X = concentration,

X_0 = EC₅₀, K = EC₅₀ half-maximal effect,

n = Hill coefficient, and R = response.

DESIGN OF TOP CHORD

TOP CHORD SPLICE

Web Splice (continued)

Try 6 Rows of Rivets

$$\text{Area of one row of rivets} = 11/16(7/8)10 = 6.02$$

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

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

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

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

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

Flange Splice

Axial Stress

$$\text{Axial load} = (-)652 \text{ K.} = 326 \text{ K./ member}$$

$$\text{Net area of one flange} = 12(1.125) - 4.5 = 9.0$$

$$\text{Total net area/ member} = 34.10$$

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

Bending Stress

$$I \text{ of section} = 2(10470) = 20940$$

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

$$\text{Bending stress in flange} = 5.30(8)1.125 = (-)47.7 \text{ K.}$$

Total Stress

$$\text{Axial stress} = (-)86.0 \text{ K.}$$

$$\text{Bending Stress} = (-)47.7 \text{ K.}$$

$$\text{Total} = (-)133.7 \text{ K.}$$

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

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

and the 1990s, the U.S. market

is now the largest in the world.

Constitutionalism and Democracy

Democracy is often seen as

the "good life" or "ideal" of democracy. This concept

of "ideal" has been used by many

to describe the political system of

the United States, Canada, Australia,

and other countries that have adopted

representative government.

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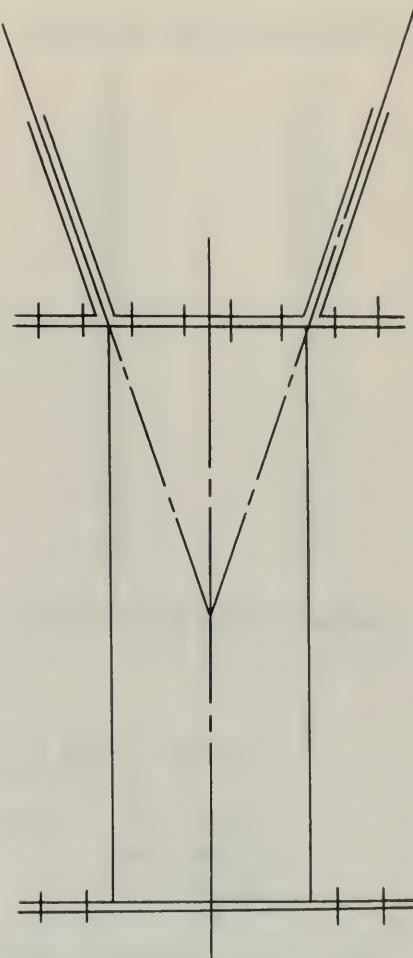
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DESIGN OF BOTTOM CHORD



Stress = 1519.0 K.

Try 2-36 WF 182

$$\text{Net Area Required} = 1519.0 / 18 = 84.30$$

$$\text{Deduct } 12, \frac{7}{8} \text{ inch rivets} = 12(1.188) = 14.25$$

Cut $1\frac{1}{2}$ inches off each bottom chord

$$\text{for backing up rivets} \quad = 3.56$$

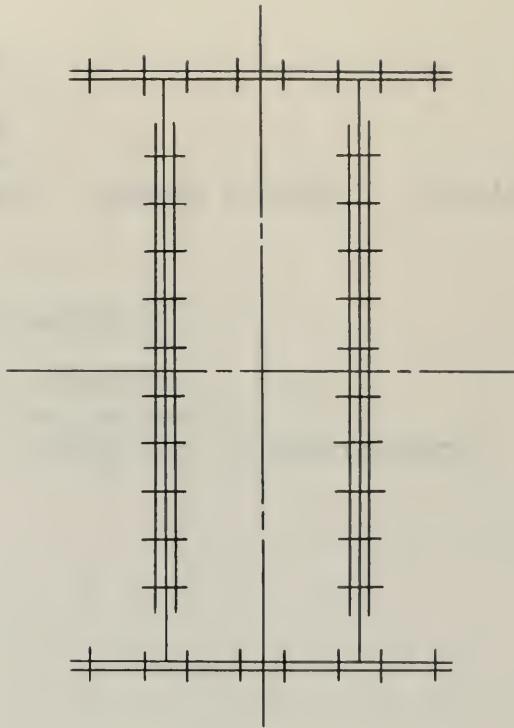
$$\text{Gross area required} \quad = 102.11$$

$$\text{Gross area available} \quad = 107.08$$

DESIGN OF BOTTOM CHORD

BOTTOM CHORD SPLICE

Splice in Member 13-14



Stress = 1048 K.

Flange Splice

$$\text{Gross area of one flange} = 12(1.187) = 14.24$$

$$\text{Deduct area of 4, } \frac{7}{8} \text{ inch rivets} = 4.75$$

$$\text{Effective area of the flange} = 9.49$$

$$\text{Strength of effective area of one flange} = 18(9.49) = 171 \text{ K.}$$

$$\text{Rivets required} = 171/8.12 = 21.0 \quad \text{Use 5 rows of rivets.}$$

Web Splice

$$\text{Gross area of one web} = 0.75(33.96) = 25.45$$

$$\text{Deduct 10, } \frac{7}{8} \text{ inch rivets} = 7.50$$

$$\text{Effective area of web} = 17.95 \text{ sq. in.}$$

$$\text{Strength of effective area of one web} = 18(17.95) = 319 \text{ K.}$$

$$\text{Number of rivets} = 319/16.23 = 19.65$$

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

DESIGN OF BOTTOM CHORD

BOTTOM CHORD SPLICE

Value of Net Section

$$\text{Webs: } 2(319) = 638 \text{ K.}$$

$$\text{Flanges: } 4(171) = 684 \text{ K.}$$

$$\text{Total: } \underline{\quad = 1322 \text{ K.}} \quad \text{Stress} = 1048 \text{ K. Satisfactory}$$

Value of Splice Rivets

$$\text{Flange rivets: } 24(4)8.12 = 780 \text{ K.}$$

$$\text{Web rivets: } 10(4)16.23 = 650 \text{ K.}$$

$$\text{Total } \underline{\quad = 1430 \text{ K.}} \quad \text{Satisfactory.}$$

DESIGN OF DIAGONALS

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

Try 30 WF 116

$$l=13.51 \text{ ft. } r = 2.12 \text{ inches}$$

$$A = 34.13$$

$$t(\text{web}) = 9/16 \text{ inches}$$

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

$$\text{Net area required} = 420.5/18 = 23.40$$

$$\text{Deduct 6 web rivets} = 6(.563) = \underline{3.38}$$

$$\text{Deduct 8 flange rivets} = 8(.875) = 7.00$$

$$\text{Gross area required} = 33.78 \text{ Satisfactory}$$

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

Try 30 WF 108

$$r = 2.06 \text{ inches}$$

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

$$f(\text{all.}) = 13.44 \text{ ksi.}$$

$$\text{Area required} = 420.5/13.44 = 31.3 \text{ sq. in.}$$

$$\text{Area available} = 31.77 \text{ Satisfactory}$$

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

Single Shear Rivets

$$\text{Minimum} = 420.5/8.12 = 51.8$$

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

8 rivets in flange

Double Shear Rivets

$$\text{Minimum} = 420.5/13.28 = 31.7 \quad \text{Use 33}$$

DESIGN OF DIAGONALS

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

Try 21 WF 62

$$r = 1.71 \text{ inches}$$

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

$$\text{Net area required} = 241.6/18 = 13.41$$

$$\text{Deduct 6 rivet holes} = 6(0.375) = \underline{\underline{2.25}}$$

$$\text{Gross area required} = 15.66$$

$$\text{Gross area available} = 18.23 \quad \text{Satisfactory.}$$

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

Try 21 WF 68

$$r = 1.74 \text{ inches}$$

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

$$f(\text{all.}) = 12.79 \text{ ksi.}$$

$$\text{Area required} = 241.6/12.79 = 18.90$$

$$\text{Area available} = 20.02 \quad \text{Satisfactory.}$$

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

Single Shear Rivets

$$\text{Minimum} = 241.6/3.12 = 29.8 \quad \text{Use 32 (4 rows of 8)}$$

Double Shear Rivets for 5-14 and 6-14

$$\text{Minimum} = 241.6/10.34 = 23.4 \quad \text{Use 27}$$

Double Shear Rivets for 3-14 and 4-14

$$\text{Minimum} = 241.6/8.80 = 27.3 \quad \text{Use 33}$$

DESIGN OF DIAGONALS

Members 5-15, 6-15 Stress = (\pm)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

$f(\text{all.}) = 12.50 \text{ ksi.}$

Area required = $164.2/12.50 = 13.14 \text{ sq. in.}$

Area available = 17.64 Satisfactory

Rivets for 5-16, 6-15

Connection design stress = (\pm)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

$$\text{Stress} = (-)100.1 \text{ K.}$$

Try 16 WF 45

$$l/r = 6.5(12)/1.52 = 51.3$$

$$f(\text{all.}) = 14.32 \text{ ksi.}$$

$$\text{Area required} = 100.1/14.32 = 6.98 \text{ sq. in.}$$

$$\text{Area available} = 13.24 \text{ sq. in.}$$

Single Shear Rivets

$$\text{Minimum} = 100.1/8.12 = 12.3 \quad \text{Use 16}$$

Double Shear Rivets

$$\text{Minimum} = 100.1/8.86 = 11.3 \quad \text{Use 12}$$

Members 3-4, 5-6

$$\text{Stress} = 61.3 \text{ K.}$$

Try 16 WF 45

$$\text{Net area required} = 61.3/18 = 3.41$$

$$\text{Deduct } 4, 7/8 \text{ in. rivets in web} = 1.50$$

$$\text{Deduct } 2, 7/8 \text{ in. rivets in flange} = 1.13$$

$$\text{Gross area required} = 6.04$$

$$\text{Gross area available} = 13.25$$

$$l/r = 6.5(12)/1.52 = 51.3 \quad \text{Satisfactory}$$

Single Shear Rivets

$$\text{Minimum} = 61.3/8.12 = 7.56 \quad \text{Use 8}$$

Double Shear Rivets

$$\text{Minimum} = 61.3/8.86 = 6.94 \quad \text{Use 8}$$

DESIGN OF LATERAL BRACING

FIRST PANEL Members 1-4, 2-3

Stress = 66.7 K.

Try 8 x 6 x 7/16 Angles

$$r = 1.31 \text{ inches}$$

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

$$\begin{aligned} \text{Effective Area} &= 7/16(8) + \frac{1}{2}(5.563)7/16 - 2(0.438) \\ &= 3.85 \text{ sq. in.} \end{aligned}$$

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

Rivets

$$\text{Strength of member} = 18(3.85) = 69.4 \text{ K.}$$

$$\text{Minimum number} = 69.4 / 8.12 = 8.55 \quad \text{Use 9, } 7/8 \text{ rivets}$$

SECOND PANEL Members 3-6, 4-5

Stress = 45.4 K.

Try 6 x 6 x 3/8 Angles

$$r = 1.19$$

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

$$\begin{aligned} \text{Effective Area} &= 3/8(6) + \frac{1}{2}(5.625)3/8 - 2(0.375) \\ &= 2.55 \text{ sq. in.} \end{aligned}$$

Required Area = 45.4 / 18 = 2.52 Satisfactory

Rivets

$$\text{Strength of member} = 2.55(18) = 45.9 \text{ K.}$$

$$\text{Minimum number} = 45.9 / 8.12 = 5.65 \quad \text{Use 6, } 7/8 \text{ rivets}$$

the first time in 1971
and again in 1973.
The first time I
had a very good
experience with it, but
I think I'm not
able to do it again.
I think it's a good
idea to have a
lot of people work
on it, because it's
not something
that you can do
alone. It's a team
effort.

DESIGN OF LATERAL BRACING (continued)

CENTER PANEL Members 5-8, 6-7

Stress = 27.2 K.

Try 6 x 6 x 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

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

$$h = t(\text{flange of 36 WF 170}) = 1.125 \text{ in.}$$

$$r = 0.289(1.125) = 0.325 \text{ in.}$$

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

Maximum distance between rivets in one flange =

$$0.325(40) = 13 \text{ inches}$$

Normal Shearing Force

$$r = 2.42 \text{ in} \quad l/r = 17(12)/2.42 = 84.2$$

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

Use Double Lacing, Flat Bars

$$t(\text{minimum}) = 20/60 = 0.333 \text{ in.} \quad \text{Use } 3/8 \text{ inch bars}$$

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

$$f(\text{all.}) = 10.78 \text{ ksi.}$$

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

$$\text{Width} = 0.361/0.375 = 0.962 \text{ in.}$$

$$\text{Minimum width} = 3(7/8) = 2.625 \text{ in.}$$

Use 3/8 by 3 inch bars.

MAXIMUM END REACTION

Dead Load

Ties, rails, fittings, and guard rails

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

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

$$R = 1.2(16670)/85 = 235.0 \text{ K.}$$

Impact

Rolling Effect

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

Direct Vertical Effect

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

$$\text{Total Impact} = 54.6\%$$

$$\text{Impact Reaction} = 128.1 \text{ K.}$$

$$\text{Total Maximum Reaction} = 418.1 \text{ K.}$$

DEPARTMENT OF THE ARMY

REF ID: A2287

ARMED FORCES AND CIVILIAN PERSONNEL

IN THE FIELD OF MILITARY POLICIES

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IN THE FIELD OF MILITARY POLICIES

REF ID: A2290

STIFFENER DESIGN AT END BEARING PLATES

Use 6 x 4 Angles 8 angles total

Bearing

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

Axial Compression

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

Use 6 x 4 x 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

$$p(\text{all.}) = 600 \text{ psi.}$$

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

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	<u>103,080 lbs.</u>
------------------------	---------------------

Weight in Kips/ ft. = <u>103,800</u> / 85(1000)	= 1.22 K./ ft.
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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 beneficial 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 transportation and erection are important.

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1935



DESIGNED BY

WILLIAM H.
FLEMING

DRAWN BY

J. P. WILSON
CIVIL ENGINEER

CHECKED BY

WARREN TYPE SPACE FRAME
RAILWAY BRIDGE
THREE CHORD DECK TYPE
DEPARTMENT OF CIVIL ENGINEERING
RENSSELAER POLYTECHNIC INSTITUTE
TROY, NEW YORK

SCALE: 1" = 1' 0"

APRIL 7, 1948

DATE DUE

Thesis

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

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