

A RIGID FRAME FOOT BRIDGE
ACROSS THE RED CEDAR

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE
Bernard C. Steinbacher
1942

THESIS

C.1

**SUPPLEMENTARY
MATERIAL
IN BACK OF BOOK**

A RIGID FRAME FOOT BRIDGE
ACROSS THE RED CEDAR

A Thesis Submitted to

The Faculty of
MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

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Candidate for the Degree of

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INTRODUCTION

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Since the removal of the old footbridge across the Red Cedar at the dam on the Michigan State College Campus, there has been no means of getting across the river on foot between the Farm Lane Bridge and the Railroad bridge. The students occupying the new N.Y.A. barracks in back of the Forestry Cabin will have no means of getting to the campus unless by the Farm Lane Bridge or the Railroad bridge.

To solve this problem, I suggest a footbridge to be placed across the Red Cedar at a point near the Forestry Cabin and the N.Y.A. barracks. This is approximately 300 feet East of the dam.

I suggest that a rigid frame bridge be used, both because of its beauty and its economy. The design I suggest is a modern and fairly simple one with a single span of 80 feet and a width of 10 feet. This gives a slenderness ratio of 8 which is large for this type bridge but on the judgement of Mr. Weber, Chief draftsman of the Michigan State Highway Bridge Division,



1.5

it is quite safe. He pointed out that the Chicago Parks Commission have built footbridges of similar width and span over the Chicago Outer Drive. Greater economy could undoubtedly have been obtained with two or three spans but the beauty of one span, in my opinion, overbalances the greater economy of a multi-span bridge.

I would like here to thank Mr. C.L. Allen and Mr. C.A. Miller for there suggestions and help in the design of this structure.

THEORY OF DESIGN

As mentioned in the introduction, an 80 foot single span was chosen for beauty, rather than a more economical multi-span structure

The structure was designed as a rigid frame from footing to footing with "free end" conditions at each footing. The theory of the "free end" conditions is that due to the compressability of the soil, the footing will rotate about the center of its bottom when the structure bends.

Since the flood stage of the Red Cedar is quite high on the south bank and overflows the north bank, it was necessary to place a fill on the north side such that the walk across the structure had no grade. Due to the soil conditions the footings for both the north and south abutments were put at the same level. Thus the structure is symmetrical in every way.

The method of design used was that of Hayden's, described in his book, "The Rigid Frame Bridge," in the chapter on symmetrical structures with "free end" conditions. This method of design consists of assuming a structure

of certain dimensions and then checking to see if it is economical, uneconomical, or unsafe. After the first time over, you can choose dimensions that will be more economical and safe from the knowledge you gained from the original check.

The dimensions I chose for my original design were based on the empirical formulae found in "Analysis of Rigid Frame Concrete Bridges", by the Portland Cement Association. (pg. 7). These first dimensions proved too small after the first time over and were substantially changed for the second try.

I selected a parabolic curve for the intrados of the structure with a two foot rise in forty feet. The geometric axis of the profile was then determined and this axis divided into equal lengths. The length I chose was six feet with one of the points directly in the center of the span. It is then the idea to check the stresses at each of these points due to all the contributing factors which is done in the tables that follow.

Mr. Hayden, in his design, depended on the concrete to take all the compressive stresses, while I found that by using steel to take part of these stresses, I was able to cut the total area of steel at some of the points almost in half. This work may be found directly after the tables.

The earth pressure was determined according to Rankin's formula and the wingwalls were designed accordingly. These wingwalls are to retain the fill for the sidewalk on the approaches to the bridge. Allowances were made for this fill to be ten feet wide at the top with 2 to 1 side slopes. The wingwalls cut these side slopes at a 45 degree angle making a fall of 1 foot in every 2'/.707 or 1 to 2.83. The walls were designed for their maximum height and these dimensions and this steel carried on through the lower part.

Due to the fact that the wingwalls were at a 45 degree angle with the face of the abutment, a difficulty arose in that a part of the heel of the wingwall footing must coincide with a part of the toe of the abutment footing. This could

not be since the structure was originally designed assuming "free end" conditions and interposing these two footings would cause the abutment footing to be restrained.

Thus, it was deemed best to place a construction joint between these two footings and use a counterfort to support the part of the wingwall that was without footing support in the heel. The details of this counterfort may be found on the sheet with the wall details.

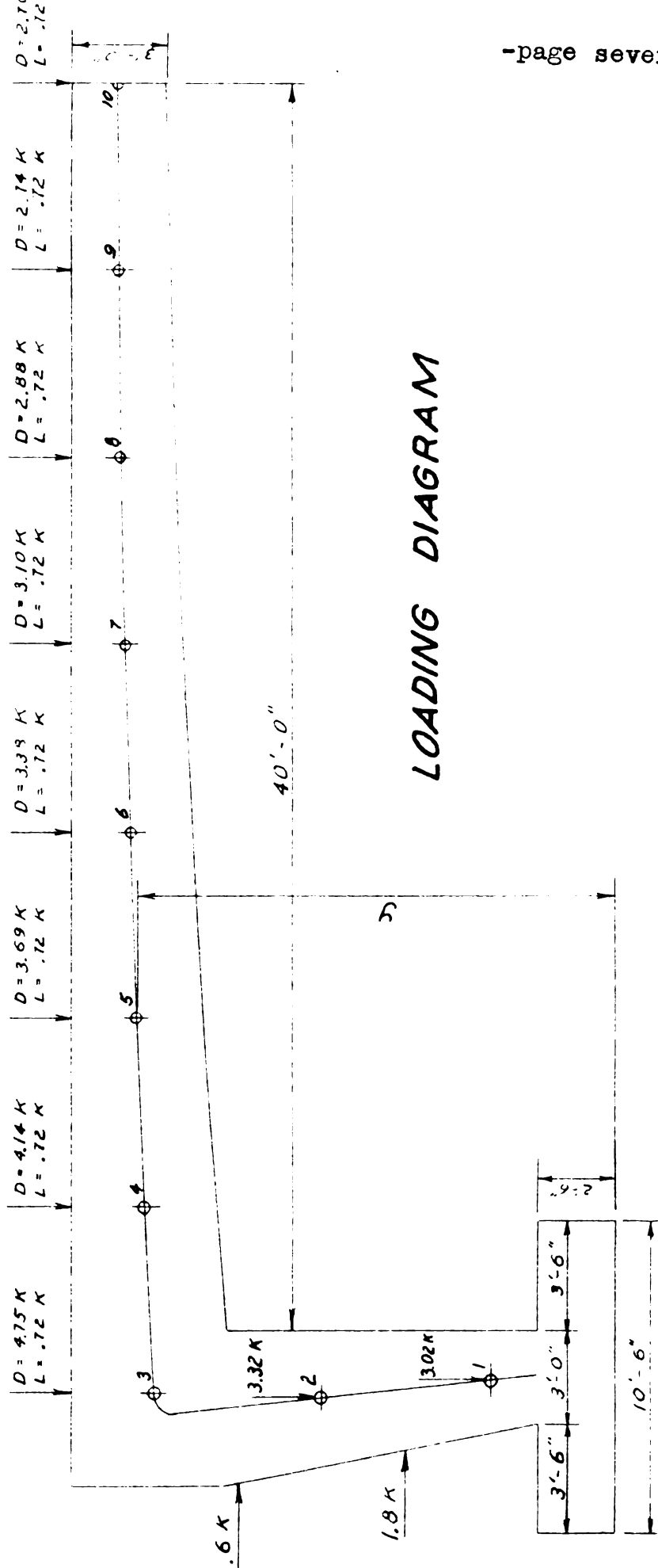


TABLE I
FRAME CONSTANTS

[illegible]

EXPLANATION AND SAMPLE
COMPUTATIONS FOR TABLE 1

This is a table of the frame constants or non-variables which are assumed and then checked.

t = Measured from the diagram

$$I_c = 1/12 t^3 = 1/12 \times 3.1^3 = 2.53$$

$A_s = .014$ (Assumed according to expected B.M.)

$$D = t/2 - .17 = 3.1/2 - .17 = 1.38$$

$$I_s = n A_s D^2 = 15 \times .014 \times 1.38^2 = .40$$

$$I = I_c + I_s = 2.53 + .40 = 2.93$$

y = Measured from the diagram

y^2 = Self explanatory

$$y^2/I = 7.82/2.93 = 2.67$$

TABLE 2
INFLUENCE LOAD = $\frac{14}{6}$

I	Y	Influence Load at 4R				Influence Load at 5R				Influence Load at 6R				Influence Load at 7R				POINT
		Mom	$\frac{My}{I}$	-Hy	Total M	Mom.	$\frac{My}{I}$	-Hy	Total M	Mom	$\frac{My}{I}$	-Hy	Total M	Mom	$\frac{My}{I}$	-Hy	Total M	
293	2.8	0	0	2.2	2.2	0	0	4.2	4.2	0	0	5.9	5.9	0	0	7.6	7.6	1 L
719	8.8	0	0	6.8	6.8	0	0	13.2	13.2	0	0	18.6	18.6	0	0	23.8	23.8	2 L
2348	14.8	0	0	11.5	11.5	0	0	22.2	22.2	0	0	31.4	31.4	0	0	40.1	40.1	3 L
1009	15.1	1	1.5	11.7	10.7	2	3.0	22.6	20.6	3	4.5	32.0	29.0	4	6.0	40.9	36.9	4 L
648	15.5	2	4.8	12.0	10.0	4	9.6	23.2	19.2	6	14.4	32.9	26.9	8	19.2	42.0	34.0	5 L
503	15.7	3	9.3	12.2	9.2	6	18.6	23.5	17.5	9	27.9	33.3	24.3	12	37.2	42.6	30.6	6 L
363	15.8	4	17.4	12.2	8.2	8	34.8	23.1	15.7	12	32.2	33.5	21.5	16	69.7	42.8	26.8	7 L
316	15.9	5	25.2	12.3	7.3	10	50.3	23.8	13.8	15	75.5	33.7	18.7	20	100.7	43.1	23.1	8 L
278	16.0	6	34.5	12.4	6.4	12	68.1	24.0	12.0	18	103.6	33.9	15.9	24	138.2	43.3	19.3	9 L
299	16.0	7	37.5	12.4	5.4	14	75.0	24.0	10.0	21	112.4	33.9	12.9	28	150.0	43.3	15.3	10
278	16.0	8	46.0	12.4	4.4	16	92.1	24.0	8.0	24	138.2	33.9	9.9	32	184.2	43.3	11.3	9 R
316	15.9	9	45.3	12.3	3.3	18	90.6	23.8	5.8	27	135.8	33.7	5.7	36	181.0	43.1	7.1	8 R
363	15.8	10	43.6	12.2	2.2	20	95.8	23.7	3.7	30	130.7	33.5	3.5	40	174.3	42.8	2.8	7 R
503	15.7	11	39.3	12.2	1.2	22	68.7	23.5	1.5	33	103.0	33.3	.3	30	92.7	42.6	12.6	6 R
648	15.5	12	28.7	12.0	0.0	24	57.5	23.2	1.8	22	52.7	32.4	10.9	20	57.9	42.0	22.0	5 R
1009	15.1	13	19.6	11.7	1.3	12	18.1	22.6	10.6	11	16.6	32.0	21.0	10	15.1	40.4	30.9	4 R
2348	14.8	0	0	11.5	11.5	0	0	22.2	22.2	0	0	31.4	31.4	0	0	40.1	40.1	3 R
719	8.8	0	0	6.8	6.8	0	0	13.2	13.2	0	0	18.6	18.6	0	0	23.8	23.8	2 R
293	2.8	0	0	2.2	2.2	0	0	4.2	4.2	0	0	5.9	5.9	0	0	7.6	7.6	1 R
Σ			352.7				682.2				967.5				1236.2			
			H = .774				H = 1.5				H = 2.12				H = 2.71			

TABLE 2 (cont'd)

P o i n t	I	Y	Influence Load at BR			Influence Load at 9 R			Influence Load at 10		
			Mom	My I	-Hy	Total M	Mom	My I	-Hy	Total M	Total M
1L	2.93	2.8	0	0	0.6	8.6	0	0	7.1	9.4	9.9
2L	7.79	8.8	0	0	27.1	27.1	0	0	29.4	29.4	30.1
3L	23.48	14.8	0	0	45.6	45.6	0	0	44.4	44.4	50.6
4L	10.09	15.1	5	7.5	46.5	41.5	6	9.0	50.4	44.4	44.6
5L	6.48	15.5	10	24.0	47.7	37.7	12	28.8	51.8	34.8	39.0
6L	5.03	15.7	15	46.5	48.3	33.3	18	55.8	52.4	39.4	32.7
7L	3.63	15.8	20	87.1	48.6	28.6	24	104.5	52.8	28.8	26.0
8L	3.16	15.9	25	125.8	49.0	29.0	30	150.9	53.1	23.1	19.4
9L	2.78	16.0	30	172.6	49.3	19.3	36	207.0	53.4	17.4	12.7
10	2.99	16.0	35	187.4	49.3	14.3	42	225.0	53.4	11.4	5.7
9R	2.78	16.0	40	230.0	49.3	9.3	48	276.4	53.4	5.4	12.7
8R	3.16	15.9	45	226.5	49.0	4.0	40	201.2	53.1	13.1	19.4
7R	3.63	15.8	36	156.9	48.6	12.6	32	139.3	52.8	20.8	26.0
6R	5.03	15.7	27	84.3	48.3	21.3	24	75.0	52.4	28.4	32.7
5R	6.48	15.5	18	43.1	47.7	29.7	16	38.3	51.8	35.8	39.0
4R	10.09	15.1	9	13.6	46.5	37.5	8	12.1	50.4	42.4	44.6
3R	23.48	14.8	0	0	27.1	45.6	0	0	49.4	49.4	50.6
2R	7.79	8.8	0	0	27.1	27.1	0	0	29.4	29.4	30.1
1R	2.93	2.8	0	0	8.6	8.6	0	0	9.4	9.4	9.9
Σ			1405.3			1523.3			1560.8		
			H = 3.08			H = 3.34			H = 3.42		

EXPLANATION AND SAMPLE
COMPUTATIONS FOR TABLE 2

This is a table of the moments and thrusts developed at all points due to placing an influence load of 14/6 at each point individually. 14/6 was chosen as the influence load as there are 14 spaces, each 6' in length across the top of the span; thus the moments are simplified.

I = Taken from Table 1

y = Taken from Table 1

Mom. = $14/6 \times 6/14 = 1$ (for point 4)(inf. load at 4)

$H = (E M_y / I) / (E y^2 / I) = 352.7 / 455.6 = .774$
(inf. load at pt. 4)

$H_y = .774 \times 2.8 = 2.2$ (for pt. 1)(inf. load at pt. 4)

Note:- All H_y moments are negative unless otherwise noted.

Total M = Mom. + $H_y = 0 + 2.2 = 2.2$

TABLE 3
SUMMARY OF TABLE 2
 $\frac{6}{14} \times \text{Total Mom.}$

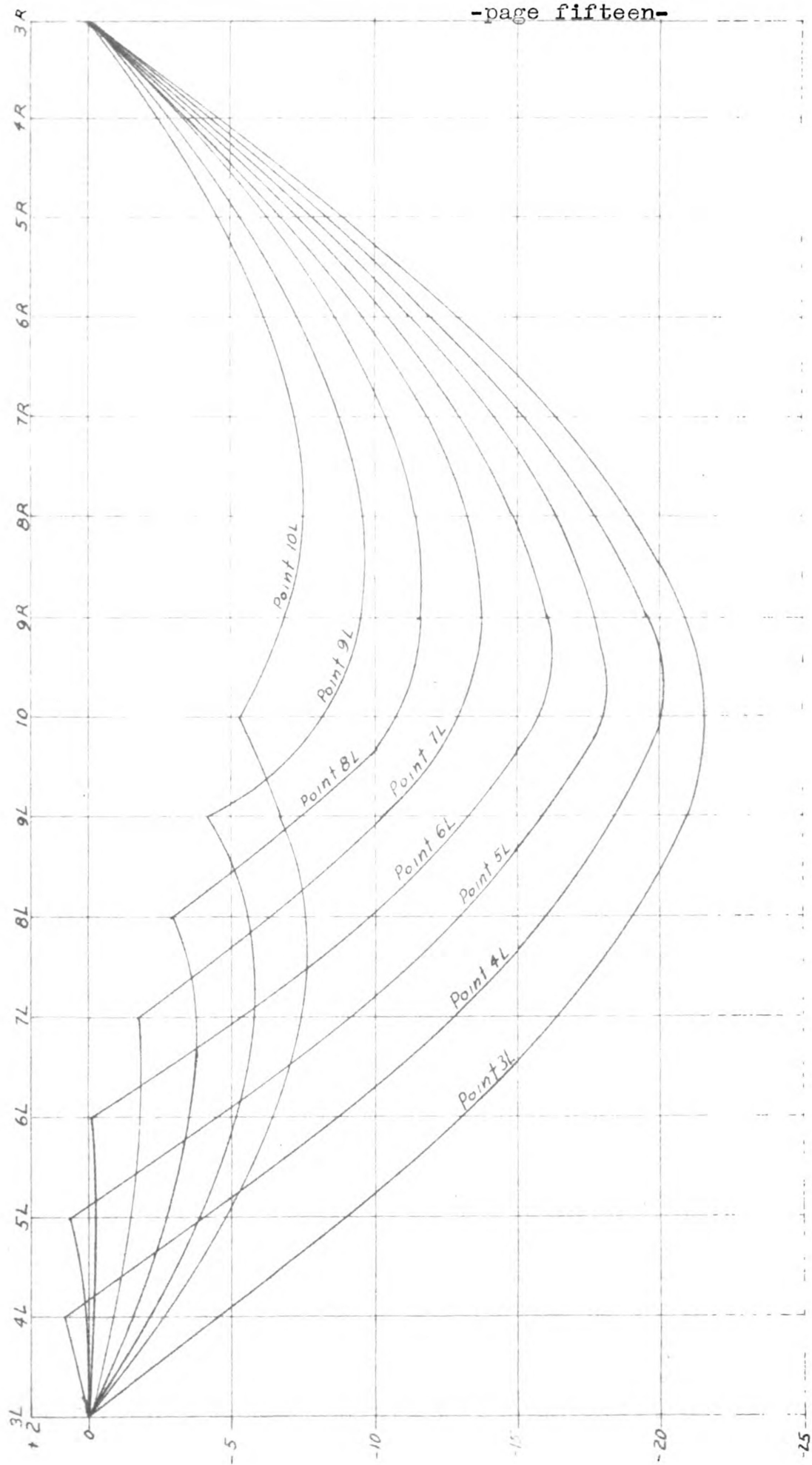
Point	4R	5R	6R	7R	8R	9R	10
1L	.9	1.8	2.5	3.2	3.7	4.0	4.2
2L	2.9	5.7	8.0	10.2	11.6	12.6	13.9
3L	4.9	9.5	13.5	17.2	19.5	21.2	21.7
4L	4.6	8.8	12.4	14.8	17.8	19.0	9.1
5L	4.3	8.2	11.5	14.0	16.2	2.1	16.7
6L	3.9	7.5	10.4	13.1	14.3	14.7	14.0
7L	3.5	6.7	9.2	11.5	12.3	12.3	11.1
8L	3.1	5.9	8.0	9.9	10.3	9.9	8.3
9L	2.7	5.1	6.8	8.3	8.3	7.5	5.4
10	2.3	4.3	5.5	6.6	6.1	4.9	3.4
9R	1.9	3.4	4.2	4.8	4.0	2.3	5.4
8R	1.4	2.5	2.4	3.0	1.7	5.6	8.3
7R	.9	1.6	1.5	1.2	5.4	8.9	11.1
6R	.5	.6	.1	5.4	9.1	12.2	14.0
5R	0	+.3	4.7	9.4	12.7	15.3	16.7
4R	+.6	4.5	9.0	13.2	16.1	18.2	19.1
3R	4.7	9.5	13.5	17.2	19.5	21.2	21.7
2R	2.9	5.7	8.0	10.2	11.6	12.6	12.9
1R	.9	1.8	2.5	3.2	3.7	4.0	4.2

EXPLANATION AND SAMPLE
COMPUTATIONS FOR TABLE 3

This is a summary table of Table 2. The results were obtained by multiplying the total moments obtained in Table 2 by $6/14$ to convert the influence load back to a load of unity.

This table might be used to draw influence lines such as the following sheet which were obtained in the first design with slightly different arch thicknesses and y distances. It is not necessary to draw these lines, however, since all values needed may be taken directly from this table. Thus, due to limited time, I did not draw influence lines for this table. These lines are valuable in that they tell you pictorially just how the bending moment at each point varies as a live load moves over the structure.

INFLUENCE LINES



INFLUENCE LINES

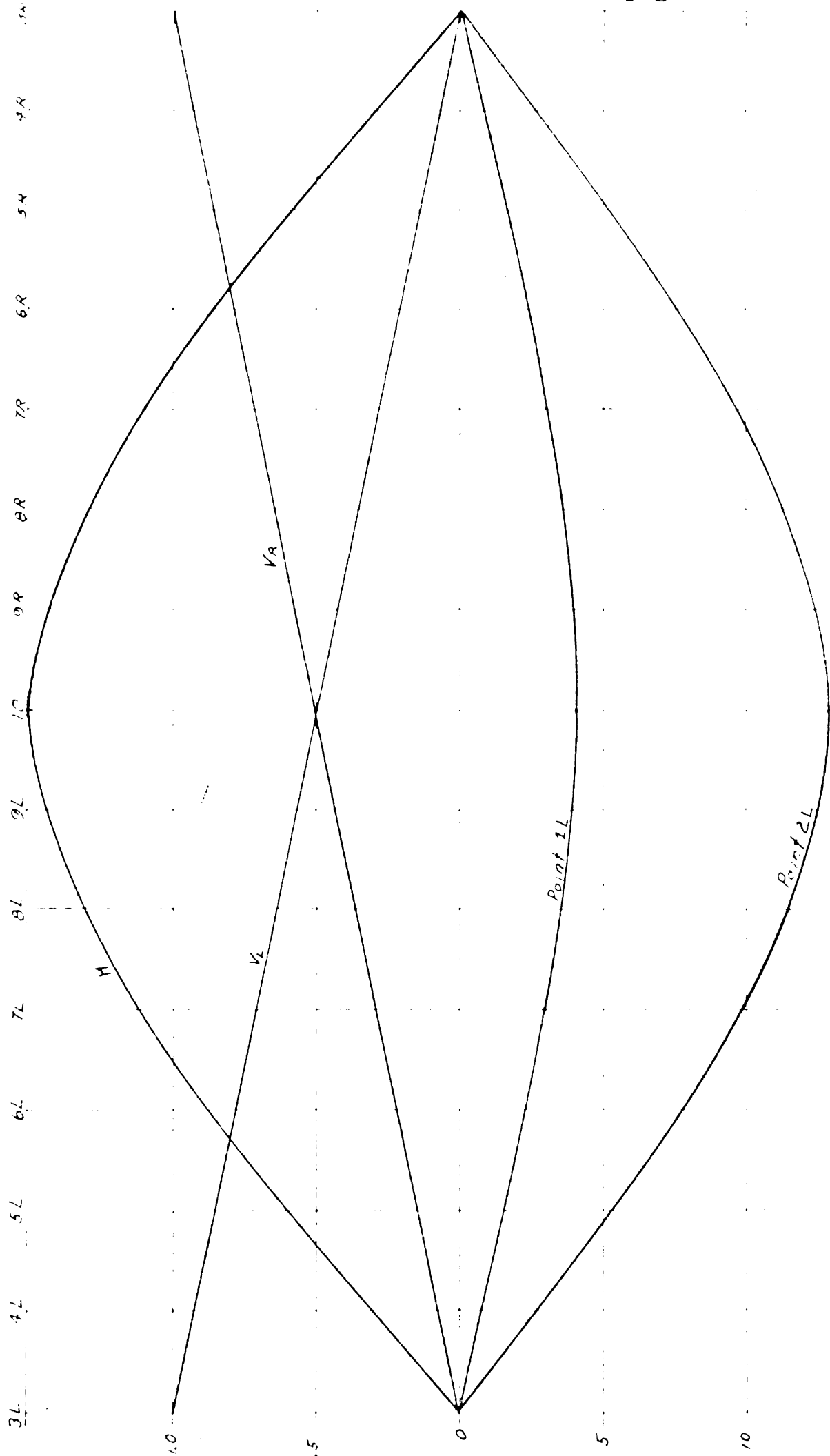


TABLE 4
DEAD & LIVE LOADS

Point	Volume	KIPS Dead Load	KIPS Live Load	KIPS Total
1	20.1	3.02	0	3.02
2	25.5	3.82	0	3.82
3	33.0	4.95	.72	5.67
4	27.6	4.14	.72	4.86
5	24.6	3.69	.72	4.41
6	22.5	3.38	.72	4.10
7	20.7	3.10	.72	3.82
8	19.2	2.88	.72	3.60
9	18.3	2.74	.72	3.46
10	18.0	2.70	.72	3.42
Σ				40.20

EXPLANATION AND SAMPLE
COMPUTATIONS FOR TABLE 4

This is a summary of the dead and live loads at each point. The live load was considered to be 120[#]/sq.' with no impact because of the use to which the bridge is to be put with no high speed heavy vehicles passing over it. Thus the live load was added to the dead and the total used as dead load.

The volumes were computed by the average end area method. The end areas were measured from the diagram at the half-way points between the points over the span.

The load was computed at 150[#]/cu. ft. for reinforced concrete.

The live load equaled 6' x 120 [#]/sq.'
= 720 # or .72 kips at each point.

TABLE 5
DEAD & LIVE LOAD MOMENTS

Load Point	Load Kips	Point 2		Point 3		Point 4		Point 5		Point 6	
		M F	M	M F	M	M F	M	M F	M	M F	M
4 L	4.9	2.9	14.2	4.7	23.0	4.6	42.9	0	0	.5	2.4
5 L	4.4	5.7	25.1	9.5	41.8	4.5	19.8	4.3	1.3	.6	2.6
6 L	4.1	8.0	32.8	13.5	55.4	9.0	36.9	4.7	19.3	.1	.4
7 L	3.8	10.2	38.8	17.2	65.4	13.2	50.1	9.4	35.7	5.4	20.5
8 L	3.6	11.6	41.8	19.5	70.2	16.1	58.0	12.7	45.7	9.1	32.8
9 L	3.5	12.6	44.1	21.2	74.2	18.2	63.7	15.3	53.6	12.2	42.7
10	3.4	12.9	43.9	21.7	73.9	19.1	65.0	16.7	56.8	14.0	57.6
9 R	3.5	12.6	44.1	21.2	74.2	19.0	66.5	17.1	59.9	14.7	51.4
8 R	3.6	11.6	41.8	19.5	70.2	17.8	64.1	16.2	58.3	14.3	51.5
7 R	3.0	10.2	38.8	17.2	65.4	15.8	60.0	14.6	55.5	13.1	49.8
6 R	4.1	8.0	32.8	13.5	55.3	12.4	50.9	11.5	47.2	10.4	42.6
5 R	4.4	5.7	25.1	9.5	41.8	8.8	38.7	8.2	36.1	7.5	33.0
4 R	4.9	2.9	14.2	4.9	24.0	4.6	22.6	4.3	21.1	3.9	19.1
Σ			437.5		734.8		596.3		490.5		406.4

TABLE 5 (cont'd)

Load Point	Load K/PS	Point 7		Point 8		Point 9		Point 10	
		M F	M	M F	M	M F	M	M F	M
4 L	4.9	.9	4.4	1.4	6.9	1.9	9.3	2.3	11.3
5 L	4.4	1.6	7.1	2.5	11.0	3.4	14.9	4.3	18.9
6 L	4.1	1.5	6.1	2.4	9.8	4.2	17.2	5.5	22.6
7 L	3.8	1.2	4.6	3.0	11.4	4.8	18.3	6.6	25.1
8 L	3.6	5.4	19.4	1.7	6.1	4.0	14.4	6.1	22.0
9 L	3.5	8.9	31.1	5.6	19.6	2.3	8.0	4.9	17.1
10	3.4	11.1	37.8	8.3	28.2	5.4	18.4	2.4	8.2
9 R	3.5	12.3	43.1	9.9	34.7	7.5	26.3	7.9	17.1
8 R	3.6	12.3	44.3	10.3	37.1	8.3	29.9	6.1	22.0
7 R	3.8	11.5	43.7	9.9	37.6	8.3	31.5	6.6	25.1
6 R	4.1	9.2	37.7	8.0	32.8	6.8	27.9	5.5	22.6
5 R	4.4	6.7	29.5	5.9	26.0	5.1	22.4	4.3	18.9
4 R	4.9	3.5	17.2	3.1	15.2	2.7	13.2	2.3	11.3
		326.0		276.4		251.7		242.4	

EXPLANATION AND SAMPLE
COMPUTATIONS FOR TABLE 5

This table gives the total maximum possible moments from both the dead and live loads.

MF = Moment Factor = Taken from the influence

Table 3 (or from influence lines when used)

$M = MF \times \text{load} = 2.9 \times 4.9k = 14.2k$ (for point
2 with the load at point 4)



TABLE 6
TEMPERATURE STRESSES

Point	Y	+HY	-HY	Point	Y	+HY	-HY
2	8.8	12.1	6.1	7	15.8	21.8	10.9
3	14.8	20.4	10.2	8	15.9	22.0	11.0
4	15.1	20.8	10.4	9	16.0	22.0	11.0
5	15.5	21.4	10.7	10	16.0	22.0	11.0
6	15.7	21.6	10.8				

c = Coeff. of thermal expansion = .0000065

t = temp. change = +35 or - 70

l = Span length in ft. = 80

E = Modulus of elasticity of concrete.

E = 144 x 2,000,000 = 288,000,000

s = length of axis divisions = 6'

$H = (Ectl)/(s \sum y^2/I) = \frac{288,000,000 \times .0000065 \times 35 \times 80}{6 \times 1265}$

H = + 690# or - 1380#

EARTH PRESSURE

The earth pressure in this case is nearly balanced by water pressure inside the bridge and there are no unequal moments set up since the earth fill on each end is of the same height. The small excess produces positive moments and are thus on the safe side and may be neglected.



TABLE 7

EARTH PRESSURE

P o n t	Vert. dist. From o	Vert. Dist. Betw o	Earth Pressure rt. H Loads				Hor. Dist. = X	Mom B V ₂	Earth Pressure - rt. H + V Loads				P o n t		
			Loads K	Σ loads	Inc. or Mom	Partial Mom. A = X			Sub Tot. M _A + M _B	y	I	M _y I		H _y	Total M
H _R	0		2.4												
1R	2.5	2.5		2.4	6.0		0	0	+6.0	2.8	2.93	+5.73	0	+5.7	1R
E ₁	5.8	3.3		2.4	7.9		0	0							
			1.8		1.8										
2R	8.8	3.0		.6	1.8		0	0	+15.7	8.8	7.79	+17.7	0	+17.7	2R
E ₂	11.8	3.0		.6			0	0							
3R	14.8	3.0					0	0	+17.5	14.8	23.48	+11.1	0	+11.1	3R
4R	15.1	.3					6	0	+17.5	15.1	10.09	+26.4	0	+26.4	4R
5R	15.5	.4					12	0	+17.5	15.5	6.48	+41.8	0	+41.8	5R
6R	15.7	.2					18	0	+17.5	15.7	5.03	+54.6	0	+54.6	6R
7R	15.8	.1					24	0	+17.5	15.8	3.63	+76.1	0	+76.1	7R
8R	15.9	.1					30	0	+17.5	15.9	3.16	+88.1	0	+88.1	8R
9R	16.0	.1					36	0	+17.5	16.0	2.78	+100.8	0	+100.8	9R
10	16.0	.0					42	0	+17.5	16.0	2.99	+93.6	0	+93.6	10

EXPLANATION AND SAMPLE
COMPUTATIONS FOR TABLE 7

This table assembles the moments at each point caused by the earth pressure against the abutment. The earth pressure was computed according to Rankin's theory with $C_e = .27$. This pressure was broken up into 2 forces and placed as shown on the diagram.

The table is self explanatory through the headings of the columns.

TABLE 8
SUMMARY OF MAXIMUM MOMENTS
AND NORMAL THRUSTS

Loading	Point 2		Point 3		Point 4		Point 5		Point 6		Point 7		Point 8		Point 9		Point 10	
	M	N	M	N	M	N	M	N	M	N	M	N	M	N	M	N	M	N
L.D Load	+437.5	+372	-734.8	+58.9	546.3	+482	-490.5	+48.2	-406.4	+48.2	-326.0	+48.2	-276.4	+48.2	+182	-242.4	+48.2	
E.r.t h	+17.7	+0	+11.1	0	+26.4	0	+41.8	0	+54.6	0	+76.1	0	+88.1	0	+102.8	0	+93.6	0
Sub. Tot.	-419.8	+372	-723.7	+58.9	546.4	+48.2	-448.7	+48.2	-351.8	+48.2	-249.9	+48.2	-188.3	+48.2	+48.2	-148.8	+48.2	
Temp -	-6.1	+7	-10.2	+7	-18.4	+7	-10.7	+7	-10.8	+7	-10.9	+7	-11.0	+7	+7	-11.0	+7	
Temp +	+12.2	-1.4	+20.4	-1.4	+20.8	-1.4	+21.4	-1.4	+21.6	-1.4	+21.8	-1.4	+22.0	-1.4	+22.0	-1.4	+22.0	-1.4
Max. Tot.	-425.9	+379	-733.9	+59.6	-580.3	+48.9	-459.4	+48.9	-362.6	+48.9	-260.8	+48.9	-199.3	+48.9	+48.9	-159.8	+48.9	

EXPLANATION AND SAMPLE
COMPUTATIONS FOR TABLE 8

This table is a summary of the maximum moments and normal thrusts at each point. The values used in it are obtained from the previous tables and these values are added in the manner which gives us the largest possible total.

The table is self explanatory

TABLE 9
STEEL CALCULATIONS

Point	Moment		N	$e = \frac{M}{N}$	t inches	d = t - 2	$e' = \frac{e + \frac{t}{2} - 2}{e + \frac{t}{2} - 2}$	$\frac{e'}{d}$	$K = \frac{Ne'}{bd^2}$	f_c	f_s	Required p	Req. A_s in ² /width
	Ft. lb.	Inch lb											
2	425,900	5,110,000	37,900	135	52	50	159	3.2	202	800	6200	.0250	15.0
3	733,900	8,800,000	58,600	148	74	72	183	2.5	172	800	11,500	.0125	10.8
4	580,300	6,960,000	45,900	142	56	54	168	3.1	234	800	3,500	.0280	18.1
5	459,400	5,500,000	43,900	112	49	47	135	2.9	250	800	1,000	.0300	16.4
6	362,600	4,350,000	48,900	89	46	44	110	2.5	231	800	3,000	.0280	14.3
7	260,800	3,125,000	48,900	64	41	39	83	2.1	219	800	4,500	.0260	12.2
8	199,300	2,390,000	48,900	49	38	36	66	1.8	204	800	6,200	.0200	8.6
9	161,400	1,932,000	48,900	40	36	34	56	1.65	197	800	6,700	.0160	6.5
10	159,800	1,908,000	48,900	34	36	34	55	1.6	192	800	8,500	.0140	6.1

EXPLANATION AND SAMPLE
COMPUTATIONS FOR TABLE 9

This table is used to calculate the amount of tension steel needed at each point. By re-designing with compression steel however, the maximum tensile strength of the steel was developed and less steel was needed. These values are shown in the last two columns and the calculations for them may be found in the following pages.

Ft. lb. = Taken from Table 8

Inch lb. = Ft. lb. x 12

N = Taken from Table 8

$e = M/N$

t = Measured from the diagram

$d = t - 2$

$e' = e + t/2 - 2$

e'/d = self explanatory

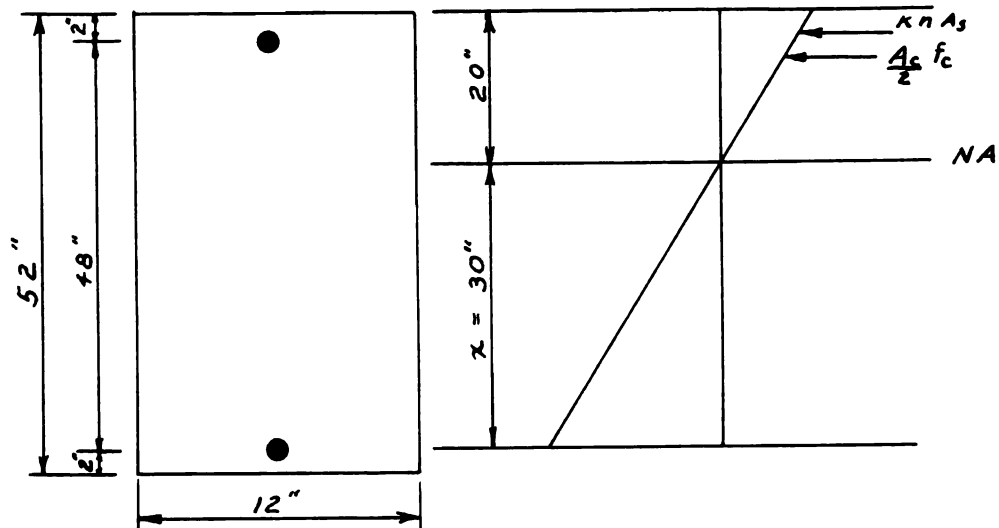
$k = Ne'/bd^2$

f_c , f_s , and p are taken from the chart pg. 280
of "The Rigid Frame Bridge" by Hayden.

$A_s = pbd$

POINT 2

Redesign With Compression Steel



$$1200/x = 2000/50$$

$$x = 30"$$

$$800/20 = k/18$$

$$k = 720\#/sq."$$

$$B.M. = 5,110,000\# = 800/2 \times 12 \times 20 \times 43.3 + 720 \times A_s \times 14 \times 48$$

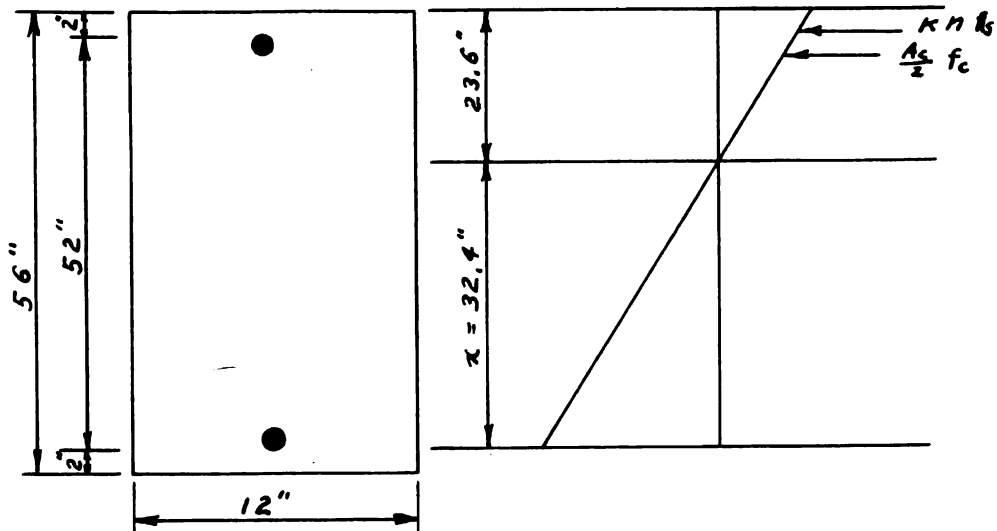
$$A_s = 1.96sq." \quad (\text{Comp. Steel})$$

$$A_s = 5,110,000\# / 18,000 \times 45$$

$$A_s = 6.3sq." \quad (\text{Ten. Steel})$$

POINT 4

Redesign With Compression Steel



$$1200/x = 2000/54$$

$$x = 32.4"$$

$$800/23.6 = k/21.6$$

$$k = 730\#/sq."$$

$$B.M. = 6,960,000\#/sq." = 800/2 \times 23.6 \times 12 \times 46 + A_s 730 52 \times 14$$

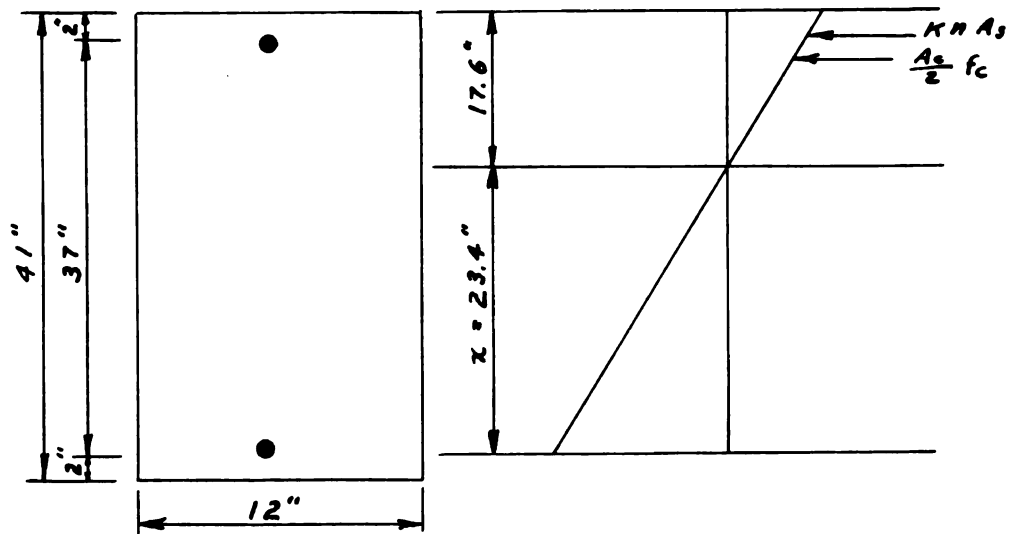
$$A_s = 3.54 sq." \text{ (Comp. Steel)}$$

$$A_s = 6,960,000\#/sq."/18,000 \times 48$$

$$A_s = 8.05 sq." \text{ (Ten. Steel)}$$

POINT 7

Redesign With Compression Steel



$$1200/x \text{ as } 2000/39$$

$$x \text{ equals } 23.4$$

$$800/17.6 \text{ as } k/16.6$$

$$k \text{ equals } 710$$

$$\text{B.M.} = 3,125,000\#/ \text{sq.inch} = 800/2 \times 17.6 \times 12 \times 33 \\ + x \times 710 \times 14 \times 37$$

$$A_s = .91 \text{ sq." (Comp. Steel)}$$

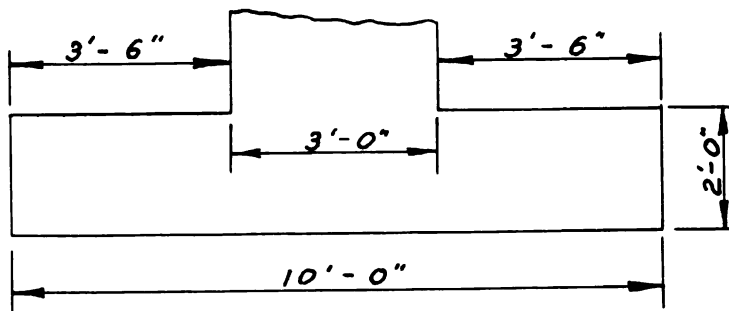
$$A_s = 3,125,000/18,00 \times 34$$

$$A_s = 5.26 \text{ sq." (Ten. Steel)}$$

FOOTING DESIGN

The "free end" condition under which this structure has been designed is based on the theory that the supporting earth under the toe or heel will give when the footing tends to tip due to the bending of the structure. The center of rotation is considered as being in the center of the bottom of the footing, thus all the earth toward the "toe side" of the tipping is stressed to its maximum bearing capacity and the bending moment is calculated accordingly. I chose 3,000 #/sq.ft. as the maximum bearing capacity of the soil.

As the footing tips about its center, the heel part theoretically tips up and the soil pressure on the bottom is thus zero. The bending moment for the heel is calculated accordingly, using the weight of the earth over the heel plus the weight of the heel itself.



DESIGN TOE

$$\text{B.M.} = 3000 \frac{\#}{\text{ft}} \times 3.5^2/2 - 3.5^2/2 \times 2 \times 150 \\ - 3.5^2 \times 15 \times 100$$

$$\text{B.M.} = 18,380 - 1,840 = 16,540 \text{ '}\#$$

$$d = (16,540/138)^{\frac{1}{2}} = 11" \quad D = 14"$$

$$A_s = pbd = .0088 \times 12 \times 11 = 1.16 \text{ sq. inches}$$

$$\text{Try } 7/8" \text{ round at } 6" = 1.20 \text{ sq. inches}$$

$$V = 3,000 \times 3.5 - 3.5 \times 2 \times 150$$

$$V = 9,450 \text{ \#}$$

$$u = V/E_ojd = 9450/5.5 \times 7/8 \times 11 = 178 \text{ \#/sq." (bad)}$$

$$v = V/bjd = 9450/12 \times 7/8 \times 11 = 82 \text{ \#/sq." (bad)}$$

$$d = V/bjv = 9450/12 \times 7/8 \times 50 = 18" \quad D = 21"$$

$$d = V/E_oju = 9450/5.5 \times 7/8 \times 100 = 20" \quad D = 23"$$

Use:-

$$D = 24"$$

$$\text{Steel} = 7/8" \text{ round at } 6" = 1.20 \text{ sq. "/ lin. ft.}$$

FOOTING DESIGN (cont'd)

DESIGN HEEL

$$B.M. = 3.5^2/2 \times 15 \times 100 + 3.5^2/2 \times 2 \times 150$$

$$B.M. = 11,040 \text{'}\#$$

$$d = (11,040/138)^{\frac{1}{2}} = 9" \quad D = 12"$$

$$A_s = pbd = .0088 \times 12 \times 9 = .95 \text{ sq.}''$$

$$1" \text{ sq. at } 12" = 1 \text{ sq.}'' / \text{ lin. ft.}$$

$$V = 3.5 \times 15 \times 100 + 3.5 \times 2 \times 150$$

$$V = 6,300 \text{'}\#$$

$$u = 6300 / 4 \times 7/8 \times 9 = 200 \text{ (bad)}$$

$$v = 6300 / 12 \times 7/8 \times 9 = 66.7 \text{ (bad)}$$

u = controlling factor

$$d = 6300 / 4 \times 7/8 \times 100 = 18" \quad D = 21"$$

Use:-

$$D = 24"$$

$$\text{Steel} = 1" \text{ sq. rods at } 12" = 1 \text{ sq.}'' / \text{ lin. ft.}$$



WINGWALL DESIGN

The soil pressure for the wingwall design was computed according to Rankin's theory.

Due to the fact that the wall cuts the 2 - 1 fill at a 45 degree angle, the surcharge angle is equal to $\tan^{-1} 1/(2/.707) = 19 \text{ degrees } 30''$. According to the Michigan State Highway specifications the angle of internal friction (ϕ) for pourous backfill is 30 degrees. Let the angle of surcharge be θ and thus:-

$$C_e = \cos \theta \times \frac{(\cos \theta - (\cos^2 \theta - \cos^2 \phi)^{\frac{1}{2}})}{(\cos \theta + (\cos^2 \theta - \cos^2 \phi)^{\frac{1}{2}})} /$$

$$C_e = .408$$

Other Design Constants:-

$$f_s = 18,000 \text{ \#/sq."}$$

$$f_c = 800 \text{ \#/sq."}$$

$$n = 15$$

$$k = 138$$

"Reinforced Concrete" by Peabody

$$p = .0088$$

Appendix pg. 435

P ■ 5420 #

$P_v = 1800 \text{ \#}$

P_H= 5110 #

$$P_w' = 5 \times 2.67 \times 64 = 855 \#$$

$$M_{\text{toe}} = 1.3 \times 15 \times 150 \times 5.67 + 1.3 \times 12 \times 150 \times 6$$
$$+ 5.67 \times 15 \times 150 \times 9.17 + 5 \times 2.67 \times 64 \times 2.5$$
$$+ 1800 \times 6.3 + 512 \times \frac{4}{3} - 5110 \times \frac{16.3}{3}$$

$M_{\text{toe}} = +134,532 \text{ '}\#$

WINGWALL DESIGN (cont'd)

Total vertical load = 20,695 #

X (distance from toe to vertical resultant) =

$$134,532 \text{ '}\# / 20,695 \text{ #}$$

$$X = 6.5'$$

$$e = -.5' \quad (6' - 6.5')$$

$$P \text{ (soil pressure)} = P/A (1 \pm 6e/d)$$

$$P = 20,695/12 (1 \pm 6 \times .5/12) = 1290 \text{ or } 2150$$

Check for Sliding

$$f = 20,695 \times .4 / (5110 - 512) = 1.8 \quad (O.K.)$$

DESIGN STEM

$$P = C_e wh^2/2 = .408 \times 100 \times 225 / 2 = 4590 \text{ #}$$

$$P_v = 1530 \text{ #} \quad P_H = 4320 \text{ #}$$

$$B.M. = 4320 \text{ #} \times 5' - 1530 \text{ #} \times .67'$$

$$B.M. = 20,580 \text{ '}\#$$

$$d = (20,580 / 138)^{\frac{1}{2}} = 12.2 \text{ say } 13" \quad D = 16"$$

$$A_s = pbd = .0088 \times 12 \times 13 = 1.37 \text{ sq."}$$

$$\text{Use } 1" \text{ } \phi \text{ at } 6" = 1.58 \text{ sq."}$$

$$V = 4320 - 64 \times 2.67^2 / 2 = 4090 \text{ #}$$

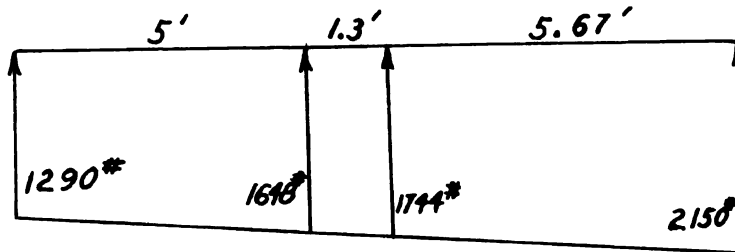
$$v = 4090/12 \times 7/8 \times 13 = 30.0 \text{ #/sq."} \quad (O.K.)$$

$$u = 4090 / 6.28 \times 7/8 \times 13 = 57.4 \text{ #/sq."} \quad (O.K.)$$



WINGWALL DESIGN (cont'd)

FOOTING DESIGN



TOE

$$B.M. = 1290 \times 5^2/2 + 358/2 \times 5^2/3$$

$$B.M. = 17,610' \#$$

$$d = (17,610 / 138)^{\frac{1}{2}} = 11.3 \quad \text{say } 12" \quad D = 15"$$

$$A_s = pbd = .0088 \times 12 \times 12 = 1.23 \text{ sq.} "$$

$$A_s = 1" \phi / \text{at } 6" = 1.58 \text{ sq.} "$$

$$V = 1290 \times 5 + 5/2 \times 358 = 6450 \# + 896 \#$$

$$V = 7346 \#$$

$$v = 7346 / 12 \times 7/8 \times 12 = 58.2 \#/\text{sq.} " \quad (\text{poor})$$

$$u = 7346 / 6.28 \times 7/8 \times 12 = .111.0 \#/\text{sq.} " \quad (O.K.)$$

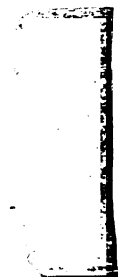
The unit shear is too high (try $d = 13"$)

$$v = 7346 / 12 \times 7/8 \times 13 = 52 \#/\text{sq.} "$$

Use:-

Steel - 1" ϕ at 6"

$$D = 16"$$



WINGWALL DESIGN (con't)

HEEL DESIGN

$$\begin{aligned} \text{B.M.} &= 15 \times 5.67^2/2 \times 100 + 1.3 \times 5.67^2/2 \times 150 \\ &\quad - 1744 \times 5.67^2/2 - 406/2 \times 5.67^2 \times 2/3 \end{aligned}$$

$$\text{B.M.} = 24,100 + 3,130 - 28,000 - 4,450$$

$$\text{B.M.} = 5,220 \text{ '#}$$

$$d = (5,220 / 138)^{\frac{1}{2}} = 6.15 \text{ say } 7" \quad D = 10"$$

$$A_s = pbd = .0088 \times 12 \times 7 = .74 \text{ sq."}$$

$$\text{use } 1" \text{ } \phi \text{ at } 12" = .78 \text{ sq."}$$

$$\begin{aligned} V &= 5.67 \times 15 \times 100 + 1.3 \times 5.67 \times 150 \\ &\quad - (1744 + 2150)/2 \times 5.67 \end{aligned}$$

$$V = 8,500 + 1,108 - 11,050$$

$$V = 1442 \text{ #}$$

$$v = 1442 / 12 \times 7/8 \times 7 = 19.6 \text{ #/sq."} \quad (\text{O.K.})$$

$$u = 1442 / 3.14 \times 7/8 \times 7 = 75.0 \text{ #/sq."} \quad (\text{OK})$$

To keep symmetry with the toe use:-

$$D = 16"$$

Steel - 1" ϕ at 12"



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FORM USE ONLY



FORESTRY
CABIN



RED CEDAR

PROPOSED
STRUCTURE

Elev. 96.80'

DAM

canoe
shelter

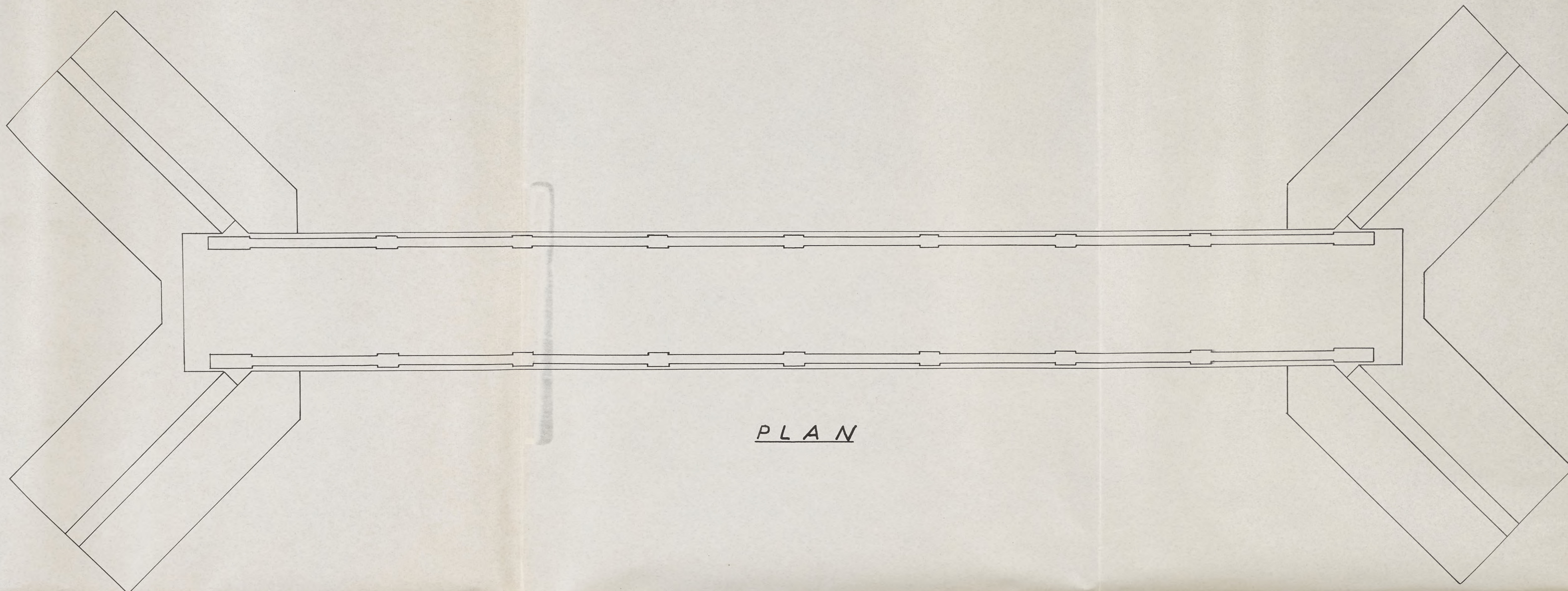
P.T. - 1+98.82

P.T. - 1+50

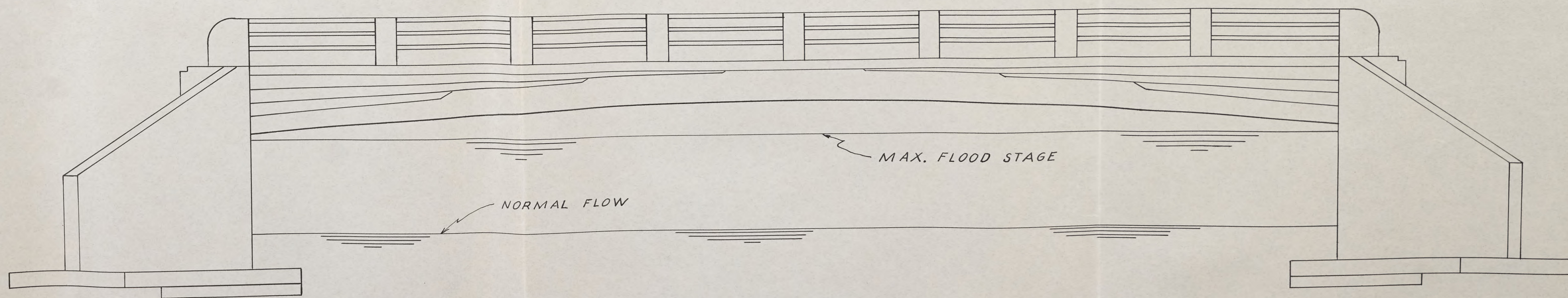
P.C. - 0+98.82

GROUND'S BLD'G.

MICHIGAN STATE COLLEGE SENIOR THESIS			
DESIGN OF RIGID FRAME FOOTBRIDGE			
SCALE: 1" = 30'	PLAN OF SITE		DWG. NO. 1
DATE: MAY 27, '42			SHEET NO. 1
DRAWN: B. Stinson	CHECKED	APPROVED	FILE NO.

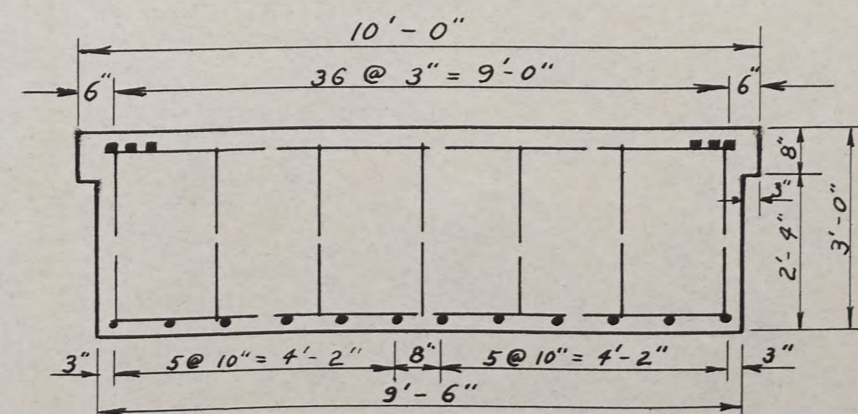
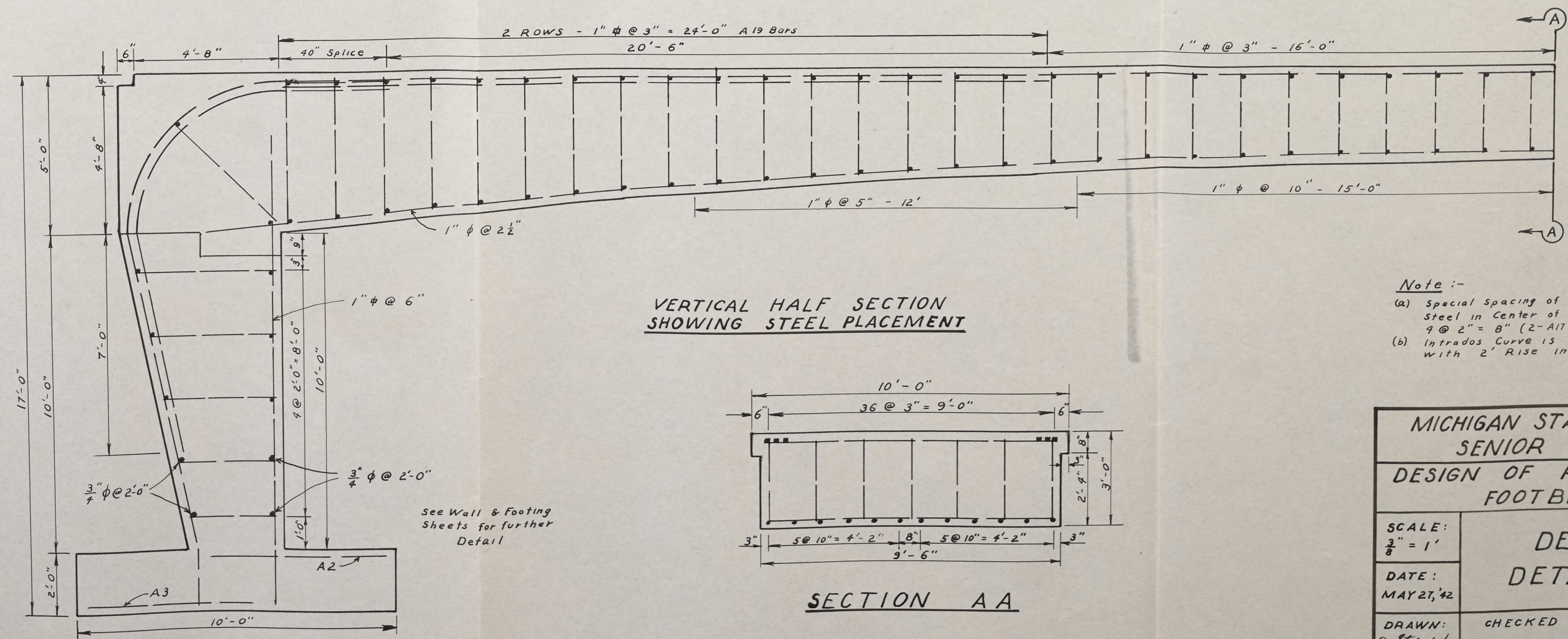
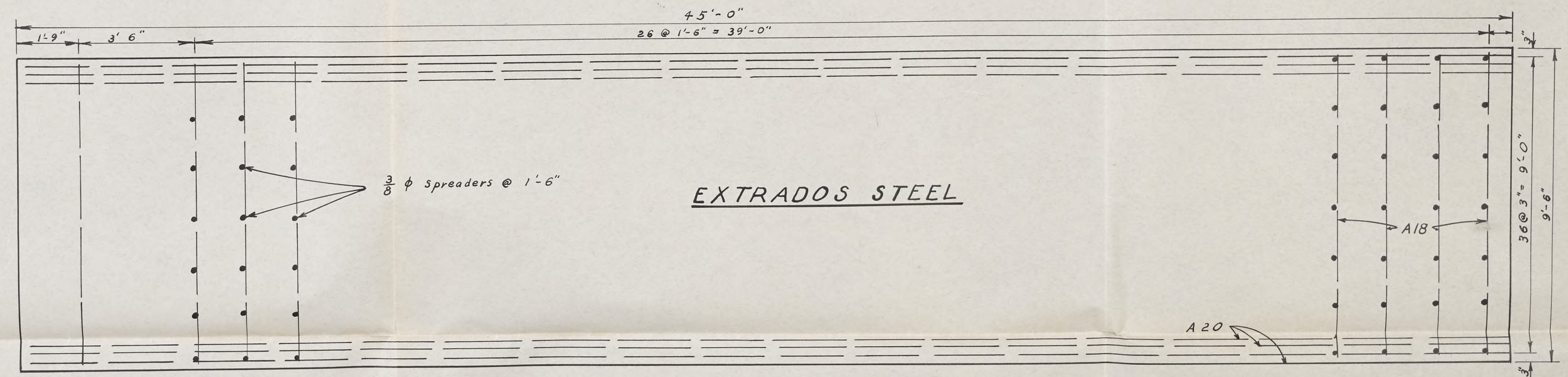
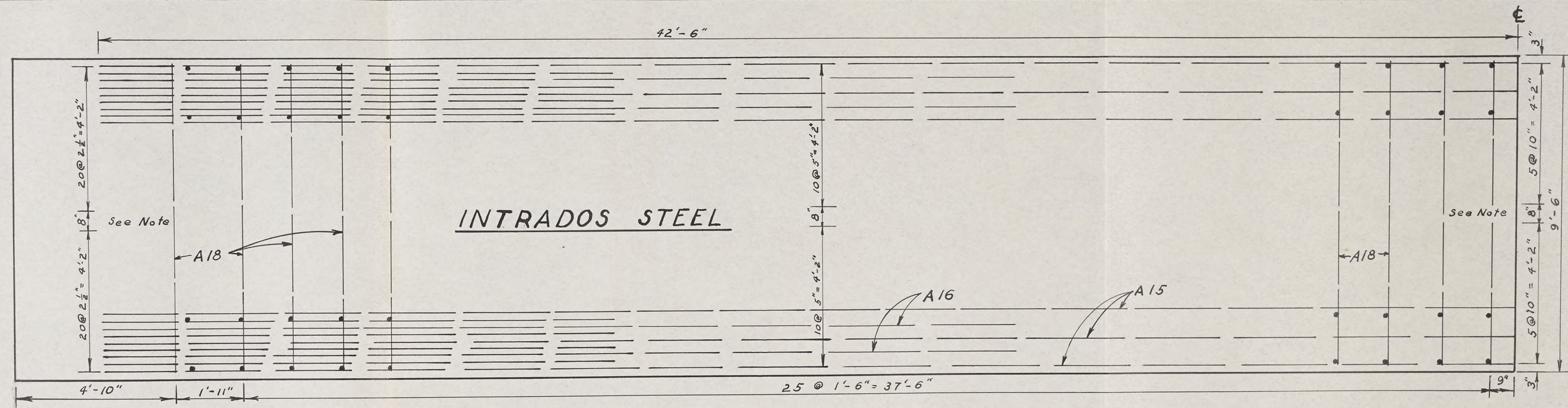


PLAN



ELEVATION

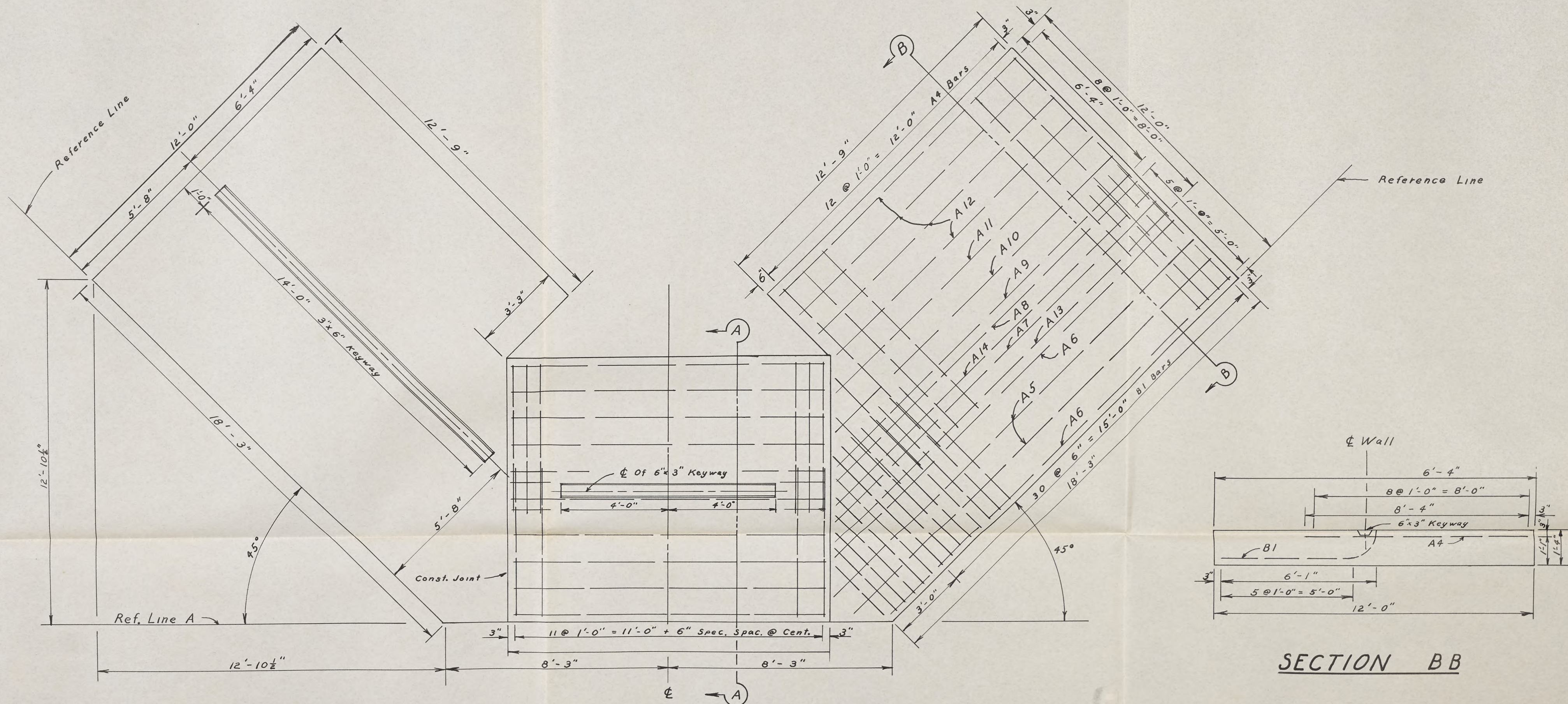
MICHIGAN STATE COLLEGE SENIOR THESIS DESIGN OF RIGID FRAME FOOTBRIDGE			
SCALE: $\frac{3}{16}'' = 1'$	PLAN & ELEVATION		DWG. NO. 2
DATE: MAY 27, '42			SHEET NO. 2
DRAWN: O. Stueber	CHECKED	APPROVED	FILE NO.



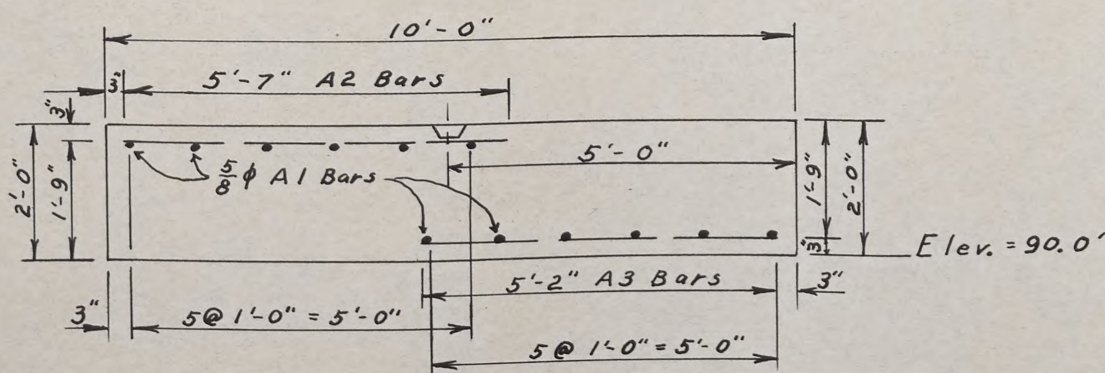
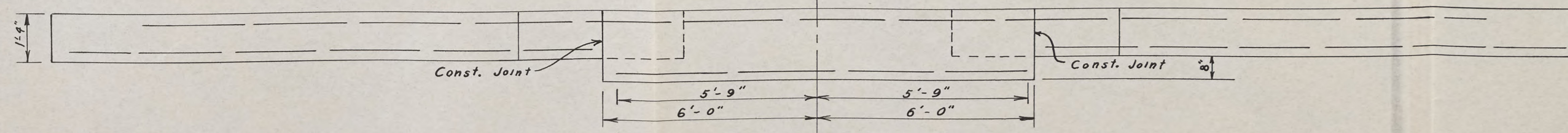
Note :-

- (a) Special Spacing of Longitudinal Steel in Center of Intrados
4 @ 2" = 8" (2-A17 Bars 1-A16 Bar)
- (b) Intrados Curve is Parabolic with 2' Rise in 40'

MICHIGAN STATE COLLEGE SENIOR THESIS			
DESIGN OF RIGID FRAME FOOTBRIDGE			
SCALE: $\frac{3}{8}'' = 1'$	DECK DETAILS		DWG. NO. 3
DATE: MAY 27, '42			SHEET NO. 3
DRAWN: B. Steinbock	CHECKED	APPROVED	FILE NO.



SECTION BB

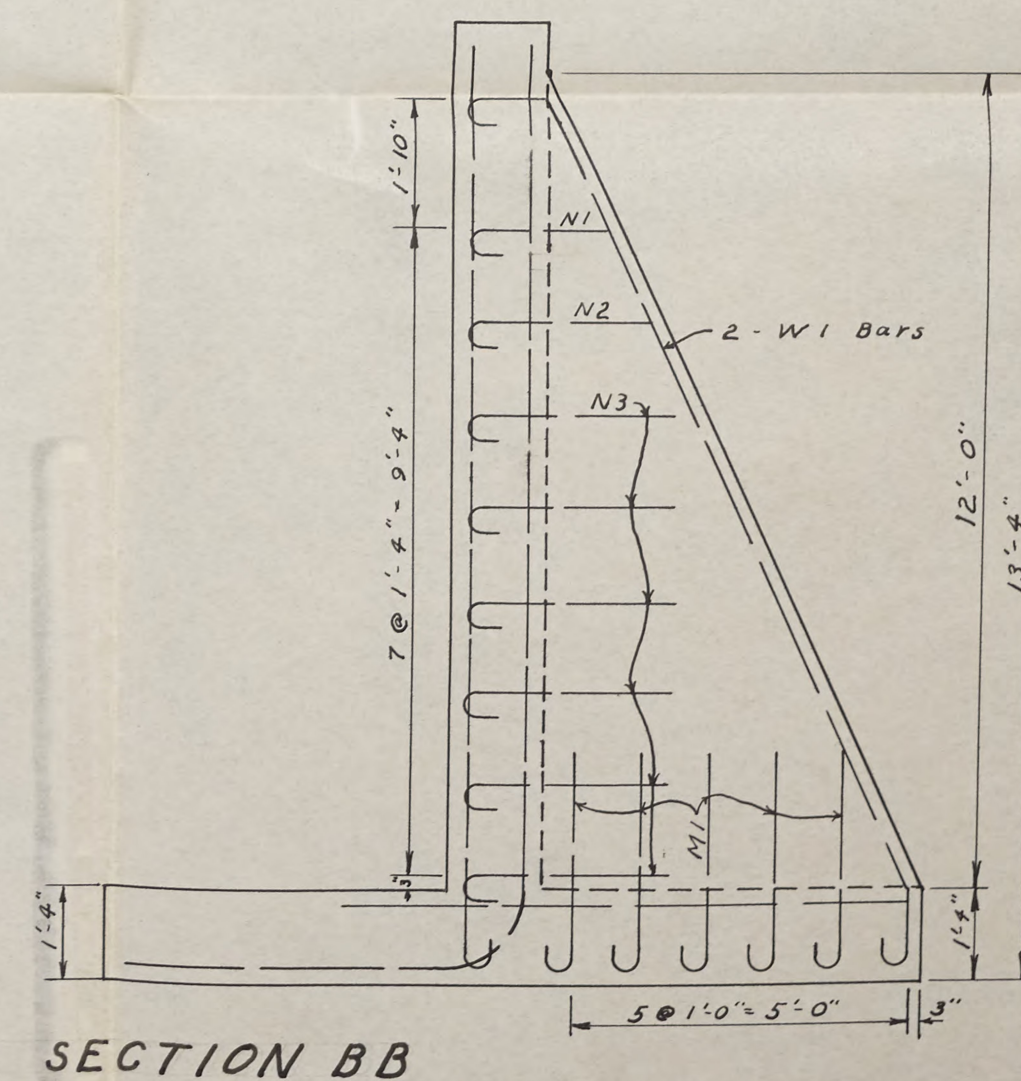
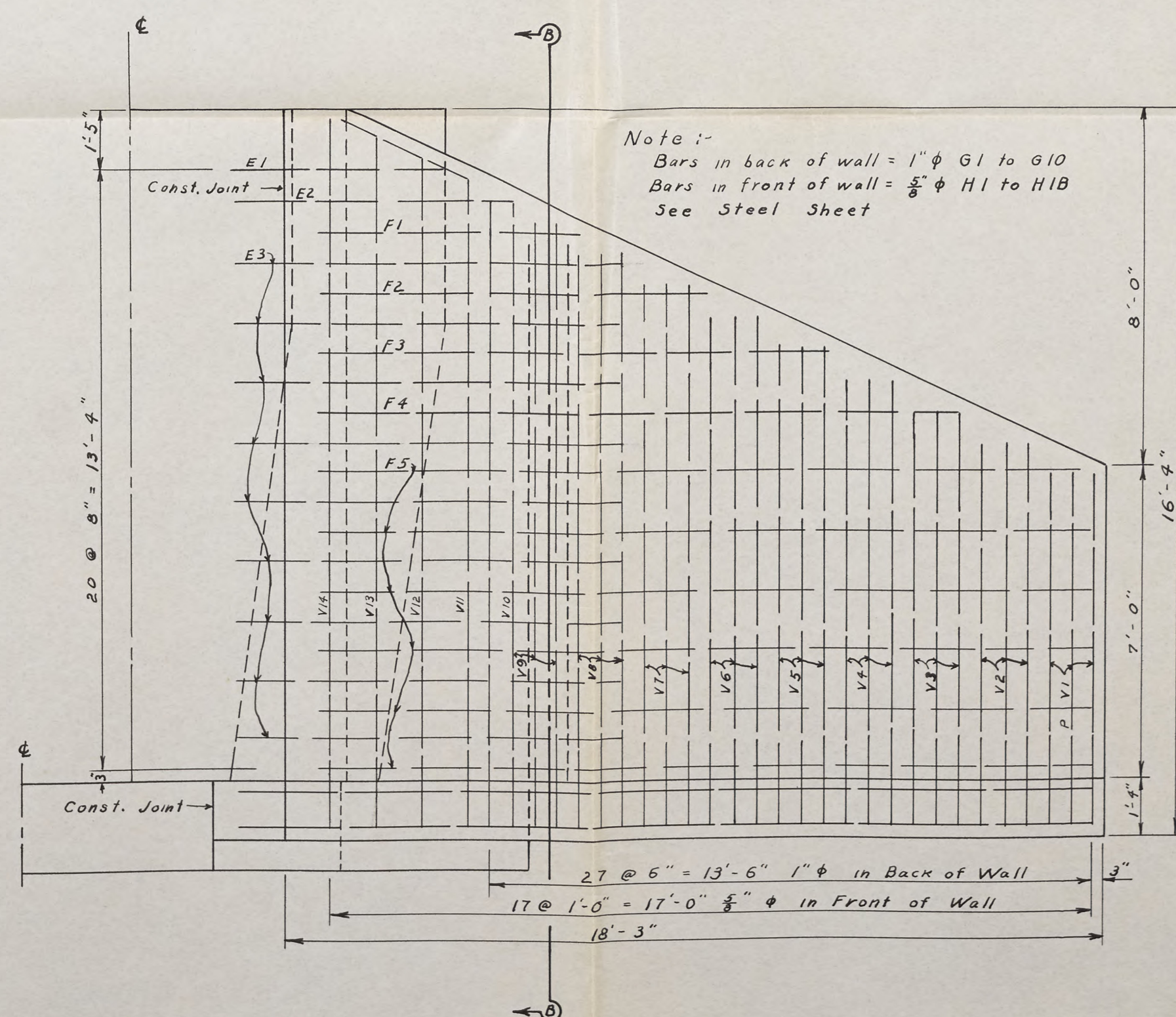
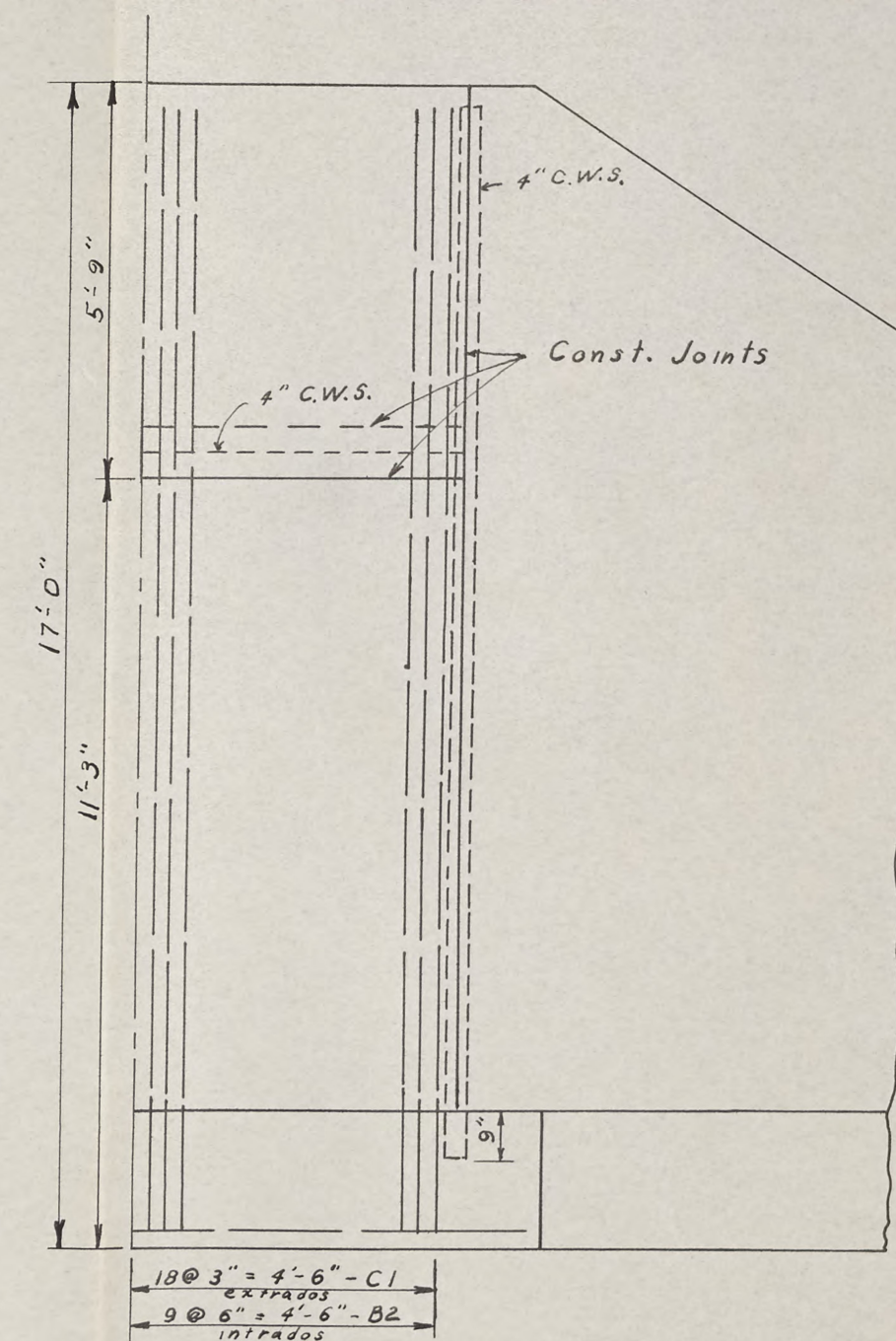
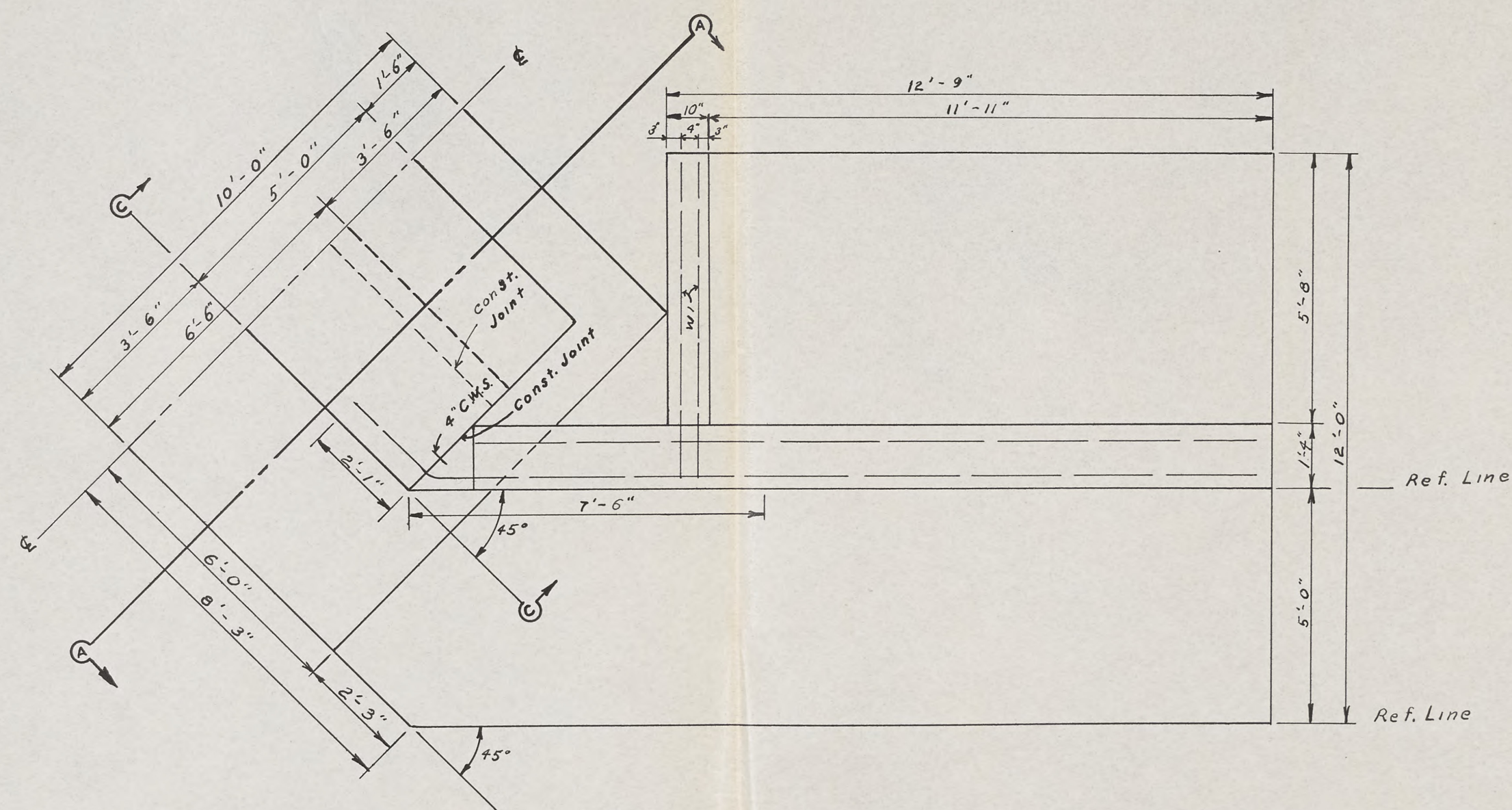


SECTION AA

Notes :-

- Maximum Foundation Soil Pressure = 3000 #/sq. ft.
- Steel Placement & Dimensions identical in Both Wingwall Footings.
- No Steel Shell Project Thru Construction Joint.
- All Material & Workmanship Shall be According to The Michigan State Highway Specifications (1941) Unless Otherwise Stated

MICHIGAN STATE COLLEGE SENIOR THESIS			
DESIGN OF RIGID FRAME FOOTBRIDGE			
SCALE: $\frac{3}{8}'' = 1'$	FOOTING DETAILS		DWG. NO. 4
DATE: MAY 27, '42			SHEET NO. 4
DRAWN: B. Stubbins	CHECKED	APPROVED	FILE NO.



SECTION AA
(SEE DECK DETAILS)

MICHIGAN STATE COLLEGE
SENIOR THESIS
DESIGN OF RIGID FRAME
FOOTBRIDGE

SCALE:
 $\frac{3}{8}'' = 1'$

DATE :
MAY 27, '42

DRAWN:
⑧ Steinbacher

WALL DETAILS

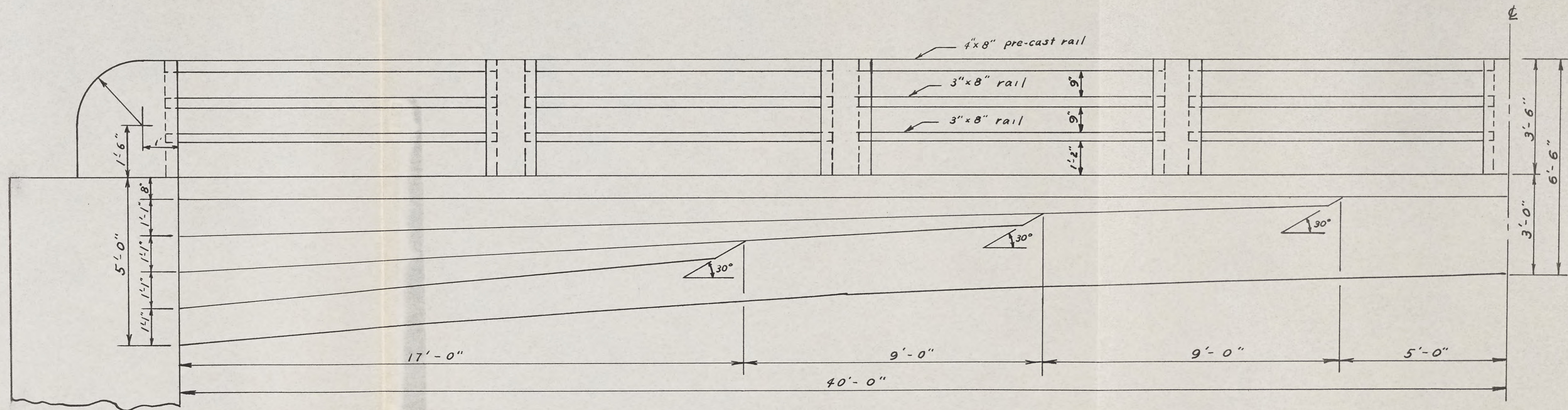
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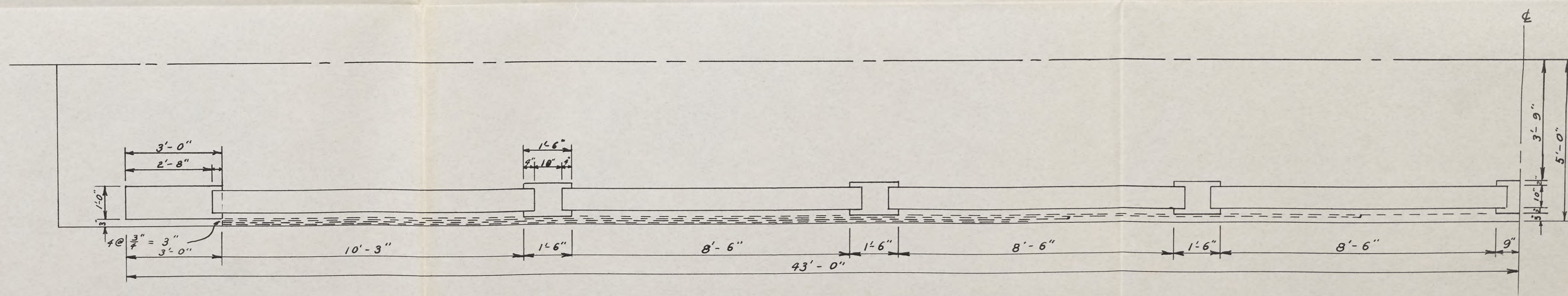
DWG. NO.
5

SHEET NO.
5

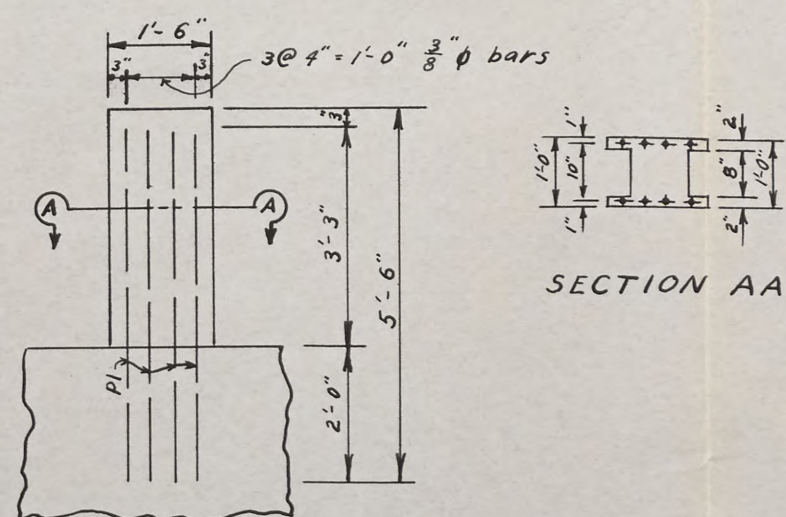
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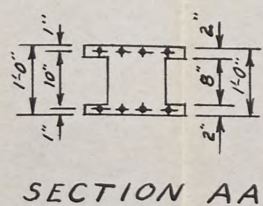
ELEVATION



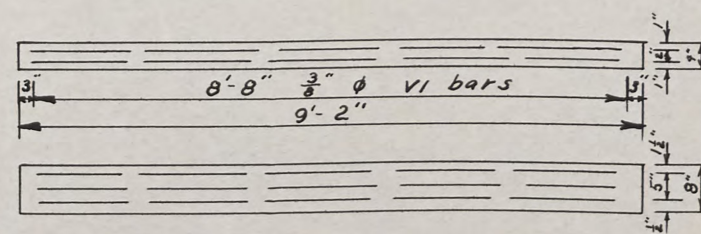
PLAN



POST DETAIL



SECTION AA



TOP RAIL DETAIL

MICHIGAN STATE COLLEGE SENIOR THESIS			
DESIGN OF RIGID FRAME FOOTBRIDGE			
SCALE: $\frac{3}{8}'' = 1'$	RAIL & FACIA DETAILS		DWG. NO. 6
DATE: MAY 27, '42			SHEET NO. 6
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Rocke
Schedule

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Schedule

STEEL SCHEDULE

MARK	NO.	SIZE	LENGTH	LOCATION
A1	12	5/8 ϕ	11'-0"	Footing
A2	7	3/4 ϕ	5'-0"	

NTARY



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