A DESIGN FOR A HIGHWAY BRIDGE AT THE PROPOSED BOGUE STREET CROSSING OF THE RED CEDAR

Thesis for the Degree of B. S.

MICHIGAN STATE COLLEGE

G. W. Michel — D. L. Welling

1949

A Design for a

Highway Bridge at the Proposed Bogue

Street Crossing of the Red Cedar

A Thesis Submitted to

The Faculty of

MICHIGAN STATE COLLEGE

of

AGRICULTURE AND APPLIED SCIENCE

by

G. W. Michel D. L. Welling

Candidates for the Degree of

Bachelor of Science

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Mr. William Bradley: For acting as advisor and checker on our design. For explaining design details and procedures of thesis composition.

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The campus of Michigan State College has of recent years expanded itself to the east to quite an extent. The construction of Snyder-Phillips and Shaw Dormitories in addition to the existing Mason-Abbot Dormitory, as well as the construction of the new Physics Building and the Natural Science Building, have made this part of the college property a more integral part of the campus proper. More so than ever before.

The development of this eastern portion of the campus has brought a tremendous increase in the amount of vehicular traffic to this area. The traffic increase comes not only from the numbers of private vehicles of visitors of the dormitory residents, but also from the commercial vehicles supplying food and services necessary for efficient operation and maintenance of these buildings.

The existing means of crossing the Red Cedar do not conveniently serve this more recently developed area. In view of the growth in importance of this eastern portion of the campus, we have proposed the design of a new crossing of the Red Cedar.

The proposed sight of this new crossing would be on a southern extension of the west land of Bogue Boulevard. This extension would cross the Red Cedar and intersect a future extension of South Shaw Lane, east of the new Shaw Dormitory. It is felt that such a crossing would greatly increase the accessibility of the men's housing and college property in this area.

Another decided advantage of such a crossing would be as a traffic easing measure. After the completion of large college activities which have brought considerable traffic on campus, present outlets from campus are very congested. An additional means of crossing the Red Cedar would provide more ready access to the main traffic arteries.

After several observations of the proposed bridge site, the following preliminary decisions were made:

In view of the relatively high level of the proposed Shaw Lane extensions and the existing Bogue Street pavement with respect to river level, the proposed bridge would have to be set correspondingly high with respect to the existing ground level to eliminate any great negative grade on the bridge approaches.

A bridge in such a setting, to attain more pleasing lines, should probably be of a single span design in view of the appearing awkwardness that could be associated with a high pier supporting section. In addition, a single span bridge would eliminate construction difficulties connected with pier construction and the absence of a pier would eliminate the accompanying obstruction to stream flow. Such a lack of obstruction would be most keenly appreciated during the relatively high flood stages of the Red Cedar as compared with normal flow.

For economy of construction, it was decided that a rolled beam rather than a plate girder would be used if at all possible. The rolled beam was selected and at this point the possibility of the use of ALPHA Composite Construction was considered. This type of design makes possible the use of longer unsupported spans. Such results are possible by considering that the concrete deck slab and the bridge beams act as though compositely constructed. These results accomplished by welding spiraled reinforcing bars to the tops of the beams and thus insuring that the concrete and steel act as one develop greater moments of inertia than if only the steel beam sections were considered.

Studies Made for Preliminary Plans

(From Michigan State Bridge Department Specifications)

Span Lengths: (From the Mississippi Valley Conference, 1940) The discussion relative to the best method of determining span lengths for new structures substantiated the practice of the M. S. H. D. The majority of states are using existing structures wherever possible to determine span lengths and are supplementing this determination by a calculation based upon either Talbot's or Donn's formula.

Stream Data and Waterway Area: The stream gauging stations, maintained by the U. S. Department of Interior, on various streams throughout Michigan have been located on our county map.

Data for each station showing maximum one-half minute discharge, drainage area, runoff data, etc., are on file in Lansing. This data should be checked when determining span lengths for proposed structures.

<u>Railing</u>: In general, concrete bridge railings are used on all concrete structures and occasionally they are used on steel structures also. The height of the rail is two feet, nine inches, and a standard eight feet, six inch panel is used between with the exception of the center and end panel, where a necessary adjustment may be made.

<u>Abutments</u>: Cantilever abutments will be used for walls eight feet, zero inches to twenty feet, zero inches, measured from the top of footing to the bridge seat. Usually the wall thickness at the base is to be approximately 0.14 of the distance between the top of footing and the crown of road. Footings to be as thick as walls at base. If the thickness of the wall at its base as determined above becomes three feet or greater, the footing thickness in the case where no piles are used shall be increased from the usual two feet, six inches to three feet, zero inches.

<u>Slopes in Front of Abutments</u>: Either rip rap or stone facing is to be used between the fascia lines on all structures where there is a considerable amount of fill in front of the abutments. In general, stone facing is to be used on these projects where appearance is an item of importance. On other structures, the

above slopes are to be reprapped.

Upon commencing this thesis, our ideas included all the data found in the finished plans of a proposed bridge. This included among other things the complete design of superstructure, abutments, wingwalls, earthwork computations, and complete detailing such as railing, steel take off, etc.

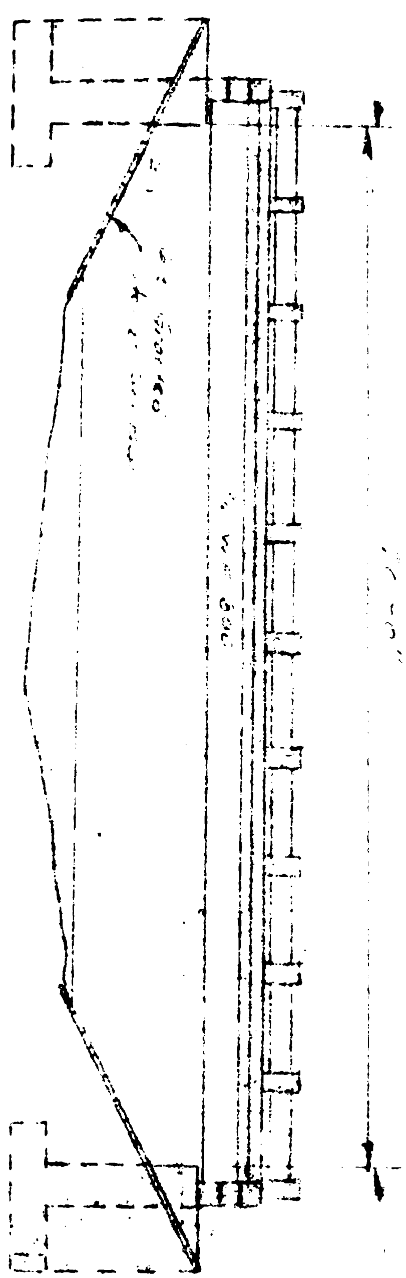
When we started our thesis with the proposed bridge site survey, it soon became most apparent to us that we had picked enough work for at least two if not more theses; consequently, as the design problem was of the most interest to us, we decided to obtain enough data to allow us to start the design and to commence there.

The design problem proved very interesting; however, it also became involved and time consuming. Due to the time allotted for the thesis, the design problem of the bridge will constitute our thesis. However, this does not complete this thesis as we would like to see it completed.

And as a suggestion to any person or persons who would like to finish this thesis, we offer the following topics:

1. Taking a log of borings and calculation of earth excavation and fill.

2. Using our designs - to draw up a complete set of final plans for the bridge, including the detailing.



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[illegible] by	[illegible]	[illegible]

Live Load [illegible] [illegible]

8000 x 6[illegible]' ÷ 7[illegible]' = [illegible]#

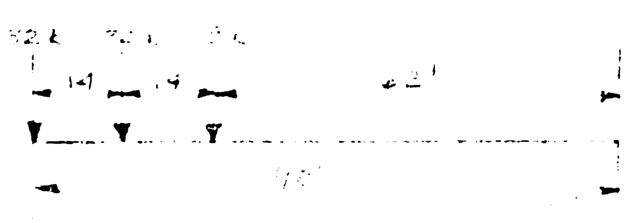
32,000 x 7[illegible]' ÷ [illegible]' = [illegible]#

32,000 x 1 = 32,000#

64,5[illegible]#

64,580 x 2 ÷ 44 = [illegible]# [illegible] ft of abutment

TABLE OF OVERTURNING MOMENTS			
Factors	Vert. Load	Arm	Moment
[illegible] x 2061 x 2	7076	[illegible] 47	45 [illegible]
280 x [illegible] 61	617	10 [illegible]	[illegible]
[illegible]			[illegible]
[illegible] x 7061	2741	10 [illegible]	[illegible]
[illegible]	[illegible]#	[illegible]	[illegible]



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Computed by [illegible] | Abutment Design | [illegible]

Checked by | | Sheet [illegible]

P = w h c

$P_1 = 100 \times$ [illegible] $=$ [illegible] #   [illegible]

$P_2 = 100 \times 54 \times .33 = 250$ #   [illegible]

$P_3 = 100 \times 450 \times .33 =$ [illegible] #

[illegible] by [illegible] | [illegible] | [illegible]

Table of Stunning [illegible]

Mass	Dimensions	Weight	Arm	[illegible] Mo.
(a)	[illegible]	[illegible]	[illegible]	31,100
(b)	[illegible]	7,447	[illegible]	[illegible] 450
(c)	1.50 x 4.2[illegible]	71[illegible]	7.25	[illegible]
(d)	[illegible] x 15.4[illegible]	[illegible]	[illegible]	[illegible] 400

Case I Structure W_I 20,[illegible]# M_I 1[illegible],41[illegible]″

\+ Superstructure [illegible] 1,03[illegible]# [illegible] 3[illegible]″

Case II [illegible] W_{II} 21,[illegible]60# M 1[illegible]2,[illegible]0″

\+ (e) 3.75 x 4.00 x [illegible] 1,4[illegible]# [illegible] 1[illegible]

Case III Condition W 21,[illegible]7[illegible]# [illegible]206,[illegible]″

1\. Superstructure 22 [illegible]# [illegible] 11,4[illegible]

[illegible]1,312# [illegible]77,41[illegible]″

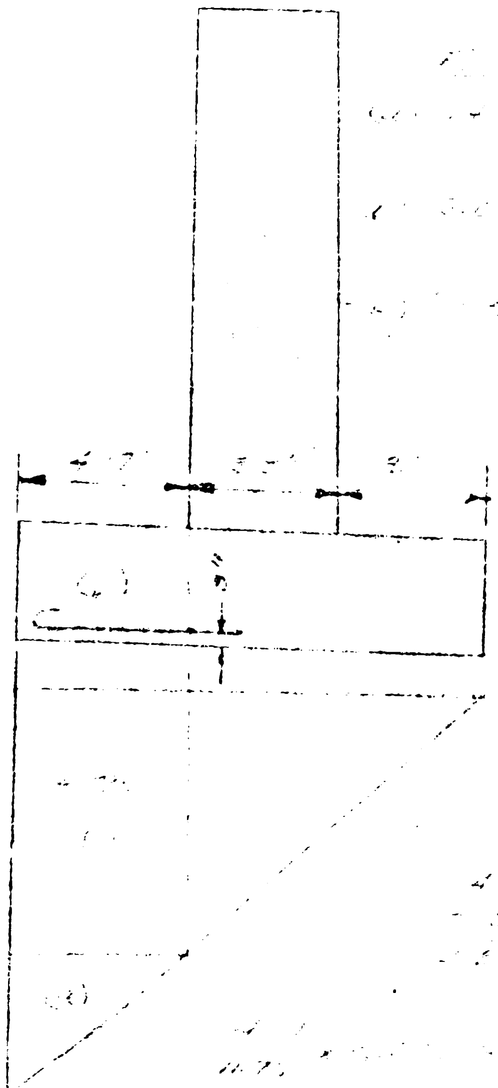
− (e) 1,4[illegible]# − 1[illegible]

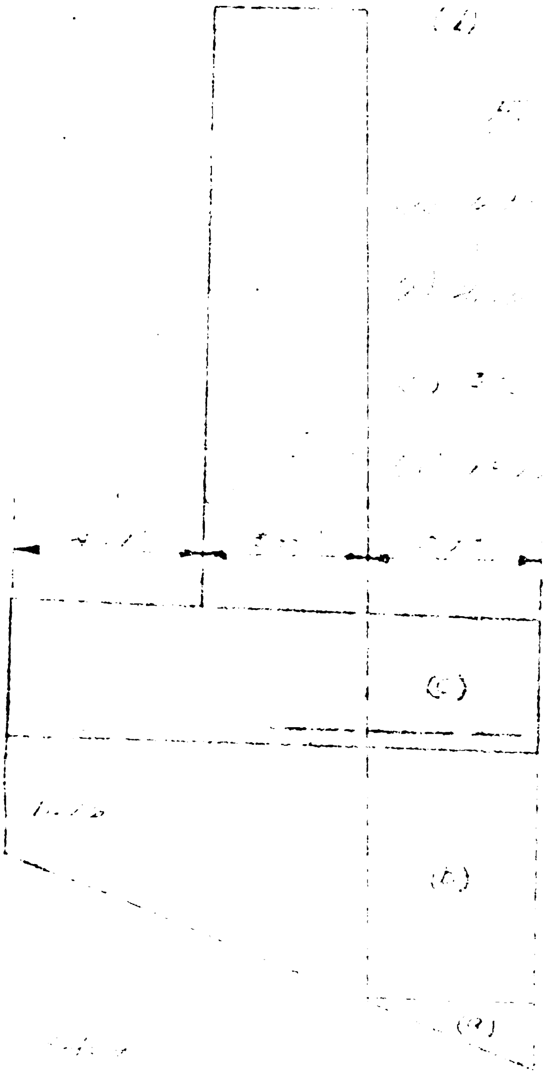
Case IV Condition W 2[illegible],352# M 20[illegible]″

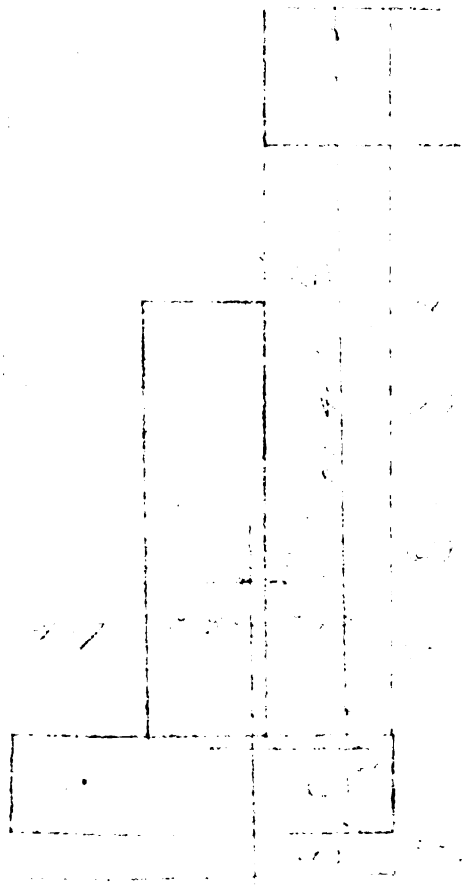
	Case I	Case II	Case III	Case IV
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	182,4[illegible]"	[illegible]"	[illegible]"	1[illegible]
[illegible]	74,[illegible]"	[illegible]"	[illegible]"	
Max.	71,710"	125,1[illegible]"	147,[illegible]"	[illegible]
$A = \frac{M}{[illegible]}$	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	= 1[illegible]	[illegible]	= 1,[illegible]
[illegible]	$\frac{20[illegible]}{1172}$	$\frac{2[illegible]}{1172}$	$\frac{[illegible]}{1172}$	$\frac{[illegible]}{11[illegible]}$
[illegible]	= 1,770	= 2,4[illegible]	= 2,[illegible]	= 2,[illegible]
[illegible]	(±) 4,0[illegible]	(±) [illegible]	(±) 1,[illegible]	(±) [illegible]

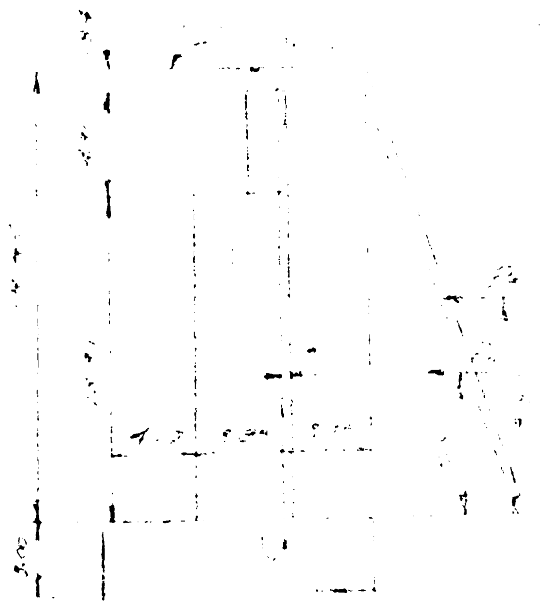
Ton

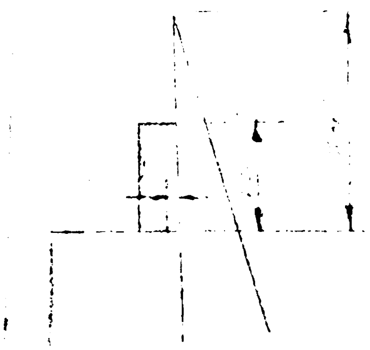
[illegible]	[illegible]23"[illegible]	4,5[illegible]"[illegible]	[illegible]45"[illegible]	[illegible]"
[illegible]	717"[illegible]	[illegible]	[illegible]4"[illegible]	[illegible]4[illegible]"

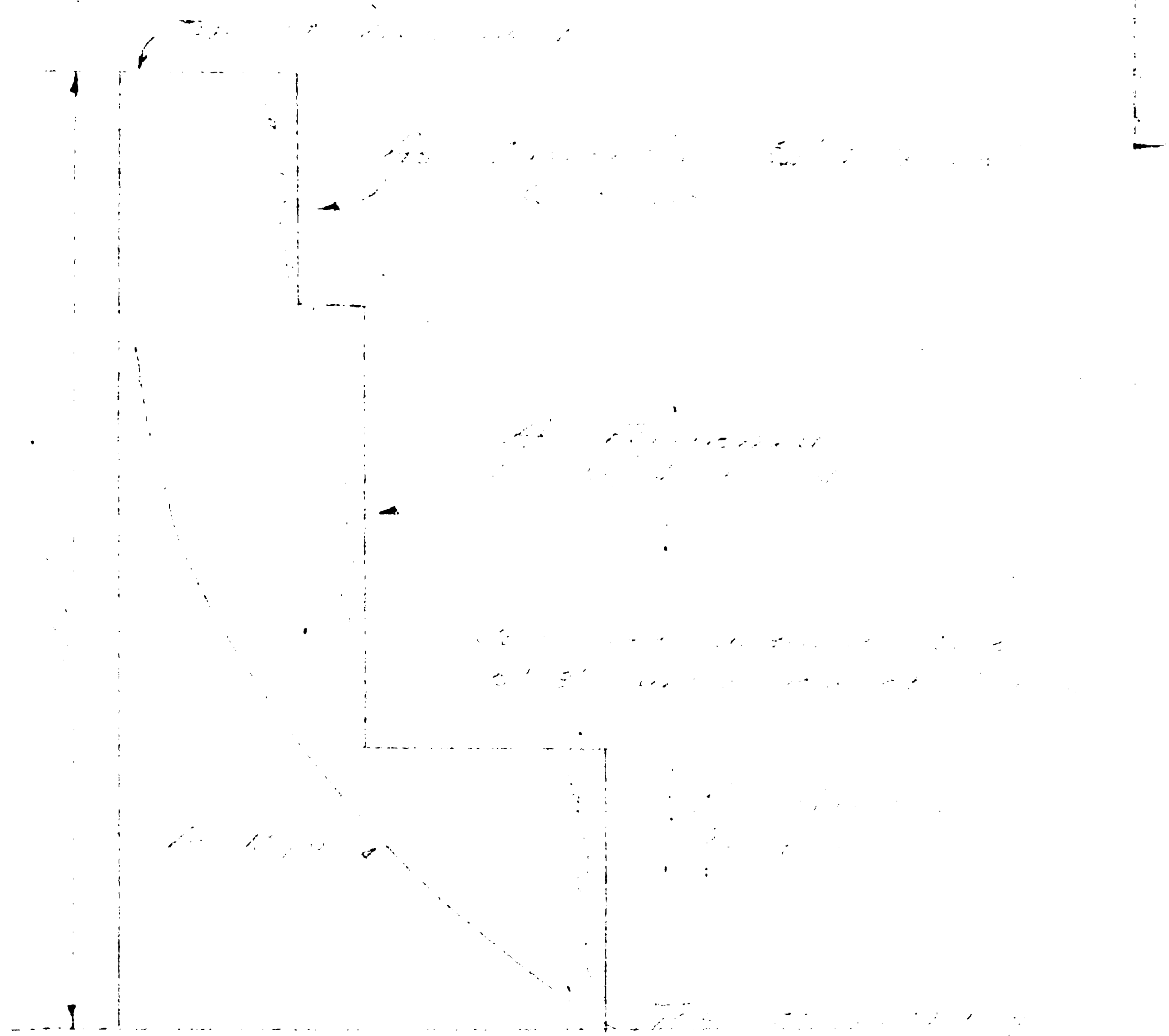


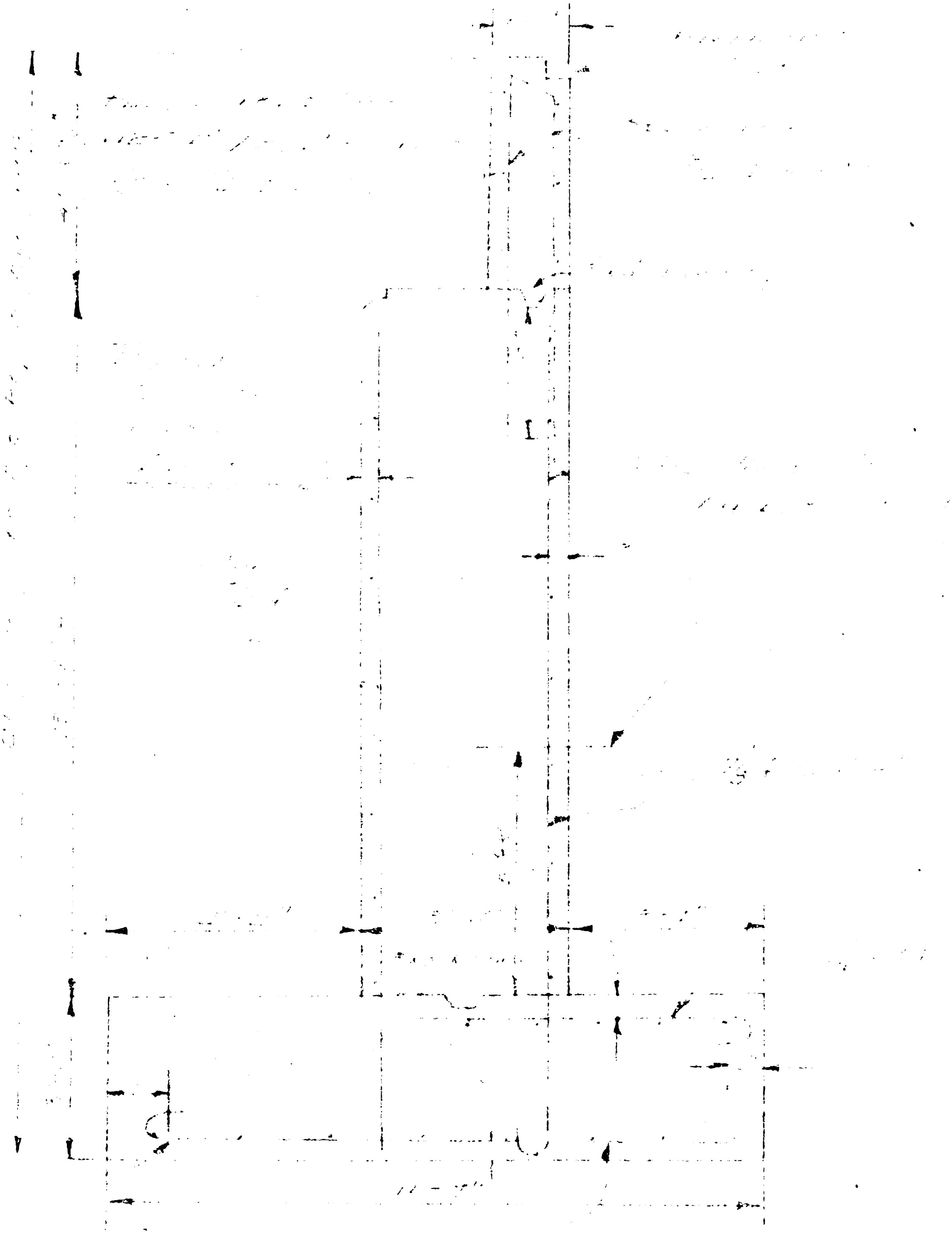


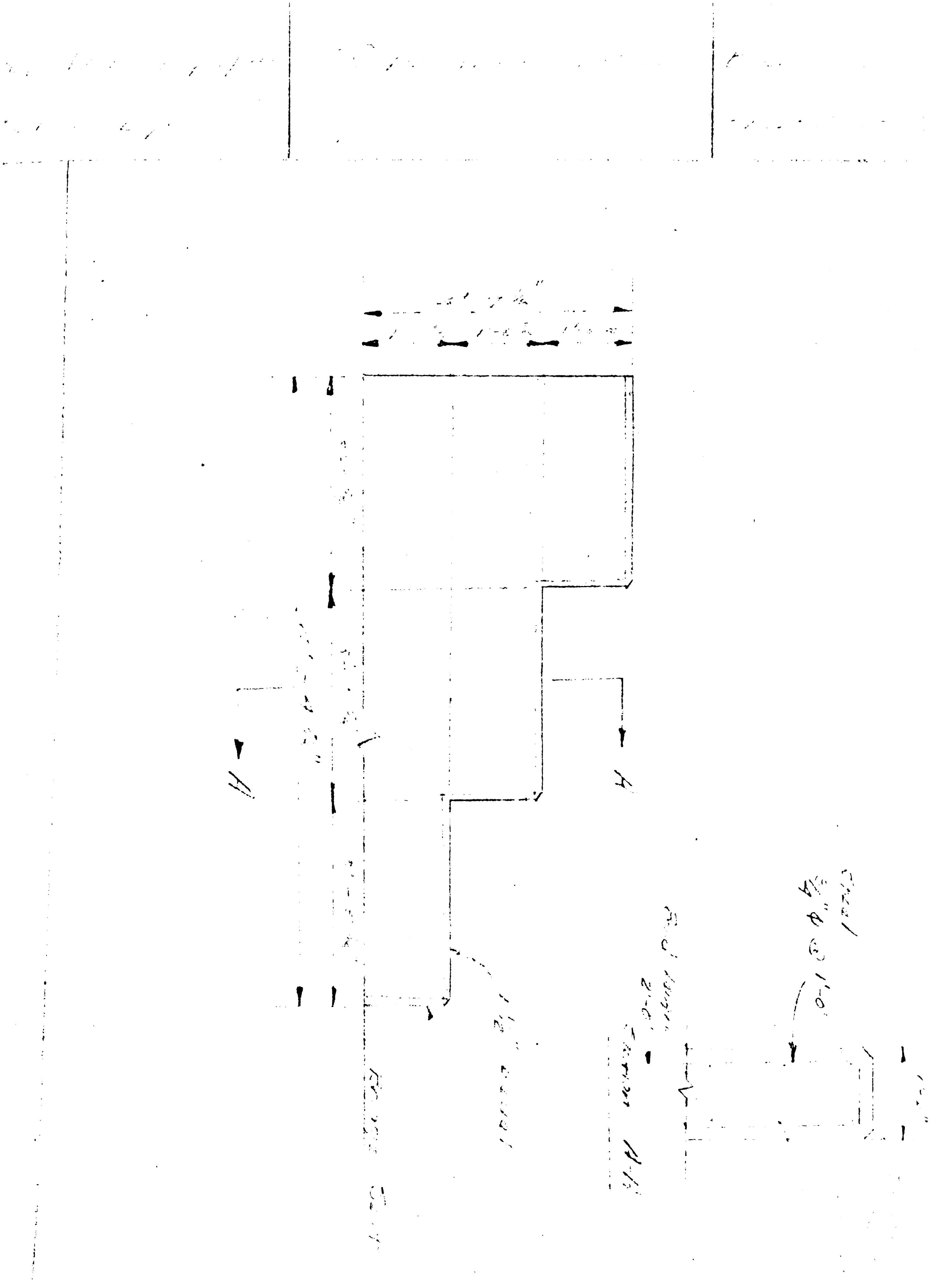












Superstructure Design

α System H-20 S-16-44 Loading

Live Load + Impact Deflection = $\frac{1}{800}$ Span

Use:

8 Beams Spaced @ 5'-3" = 63"

℄ Brg. to ℄ Brg. = 90'-0" = 90.0'

Try:

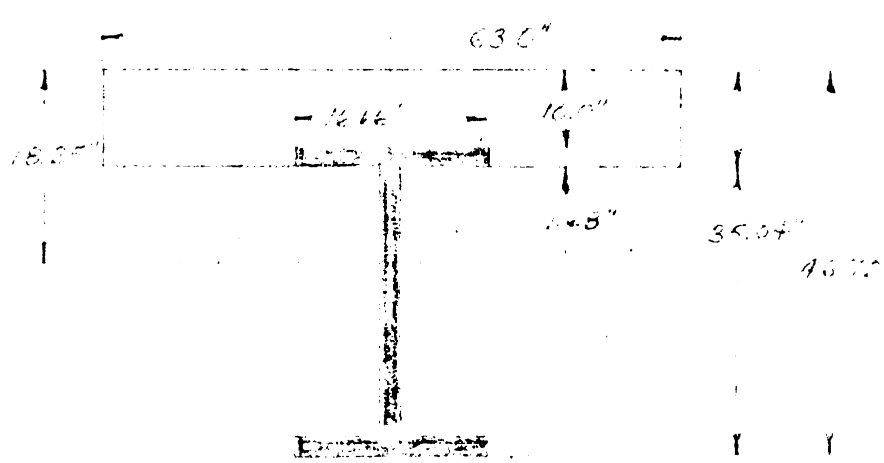
36 WF 300# Beams

$I = 20290.3 \text{ in}^4$ | $S = 1105.1 \text{ in}^3$

$A = 88.17 \text{ in}^2$ | $d = 36.72 \text{ in}$

$t = 1.680 \text{ in}$ | $b = 16.66 \text{ in}$

$t_1 = 11.65 \text{ in}$



Crown Section: Beam Precambered

Composite Beam for L.L. + Impact (n=10)

N.A.:

100 × 16.66 =	166.6	× 50	=	8330
116 × 46.34 =	541.0	× 5.84	=	31600
85.17 × 10 =	851.7	× 25.6	=	241200
	1559.3			258930

$\therefore \bar{y} = \frac{239930}{15593} = 12.25''$

I:

$\frac{16.66 \times (100)^3}{12} + 16.66 \times 100 (13.35)^2 = 1388 + 29300$

$\frac{46.34 \times (11.68)^3}{12} + 46.34 \times 11.68 (1.24)^2 = 5270 + 830$

$20290.2 \times 10 + 85.17 \times 10 (10.?)^2 = 202902.0 + 91300$

$204559.0 \qquad 202900$

$\therefore I = 546535 \text{ in}^4$

Composite Beam for D.L. (-formwork) & Add'l D.L. (+future W.S.) (n=30)

N.A.

concrete = 7078 ... 4173

Steel

85.17 × 30 =	2045	× 25.56	=	75...
	33527			78993

$\therefore \bar{y} = \frac{78993}{33527} = 23.14''$

Computed by	Detail	Bridge [illegible]
GWM	Bending Moments	Sheet [illegible]

Composite Beam for Reduced D.L. (n=30 continued)

I:

$$\frac{16.66 \times (10.0)^3}{12} + 16.66 \times 10.0 (15.24)^2 = 1386.0 + 38[illegible]$$

$$46.34 \times (11.08)^3 + 46.34 \times 11.08 (12.7[illegible])^2 = 5870.0 + 18[illegible]$$

$$20240.2 \times 30 + 5517 \times 30 (5.12)^2 = 6[illegible]70.0 + 4[illegible]$$

615862.0	263500

$\therefore I =$ 879160 in^4

Bending Moments:

Span Length = 90.0 (4 Bay to 1 Pier) + 1.0 (Bay Area)

= [illegible] = L

Stringers spaced at 5'-3" = 5.250'

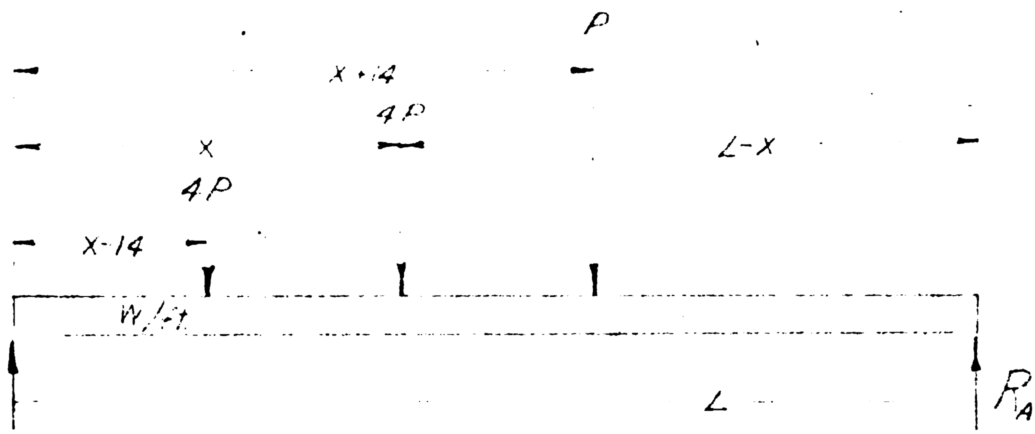
Impact = $\frac{1+[illegible]}{[illegible]} = \frac{111}{56[illegible]} = 0.196$

P = 5000 × 0.5250 × [illegible] = 5020 #/ft per stringer

Loading:

			W	W[illegible]
concrete	= 150 × 11.89 × 63 ÷ 144	=	765#	76[illegible]#
steel	= 300 × 1.10	=	330	330
future W.S.	20 × 5.250	=	[illegible]	–
forms	[illegible] × [illegible]	=	–	[illegible]
			1[illegible]#/ft	1147#/ft

Position of Absolute Maximum Moment



Proof:

$$R_A = \frac{4P(x-14) + 4Px + P(x+14)}{L} + \frac{WL}{2}$$

$$= \frac{P}{L}(9X+42) + \frac{WL}{2}$$

$$M_x = \frac{P}{L}(9X-42)(L-X) + \frac{WL}{2}(L-X) - P14 - W(L-X)\frac{(L-X)}{2}$$

$$= f_x = -\left(\frac{9P}{L} + \frac{W}{2}\right)x^2 + \left(9P + \frac{42P}{L} + \frac{WL}{2}\right)X - 56P \; *$$

*Note:

To find (x) for max. combined moment, take 2nd derivative of (f_x).
If (f_x'') is negative, the value for (x) giving max. comb. mom. is found by setting (f_x') = 0 and solving for (x).

Position of absolute maximum moment (cont)

$$M_x = f_x = -\left(\frac{9P}{L} + \frac{W}{2}\right)x^2 + \left(\frac{9P}{1} + \frac{42P}{L} + \frac{WL}{2}\right)x - (56P)$$

$$-\left(\frac{9 \cdot 5020}{91} + \frac{1200}{2}\right)x^2 + \left(\frac{9 \cdot 5020}{1} + \frac{42 \cdot 5020}{91} + \frac{1200 \cdot 91}{2}\right)x - (56 \cdot 5020)$$

$$= -(496 + 600)\,x^2 + (45180 + 2320 + 54600)\,x - 282\,000$$

$$= -1096x^2 + 102{,}100x - 282{,}000$$

$$f_x' = -2192x + 102\,100$$

$$f_x'' = -2192$$

$$\therefore\ -2192x + 102{,}100 = 0 = f_x'$$

$$\underline{x = 466'}$$

$\underline{M_x\,(LL + Imp)}$ $= -(496)(46.6)^2 + (47500)(46.6) - 282\,000$

$= -1{,}090{,}000 + 2{,}200\,000 - 282\,000$

$= \underline{828\,000'\#}$

$\underline{M_x\,(D.L)}$ $= \dfrac{Wa\,(L-a)}{2}$

$= \dfrac{1147.5}{2}(44.4)(46.6)$

$= \underline{1\,180\,000'\#}$

$\underline{M_x\,(F.W.S)}$ $= \dfrac{105}{2}(44.4)(46.6)$

$= \underline{108\,500'\#}$

Computed by:	Detail:	Bridge Design
GWM	Stresses	Sheet of

Stresses:

Case ① D.L. + Forms

$M = 1,160,000'^{\#}$ $S = 1105.1\ in^3$

$$f_s = \frac{1,160,000 \times 12}{1105.1} = \underline{12600\ ^{\#}/in^2}$$

$f_c = \underline{0\ ^{\#}/in^2}$

Case ② L.L. + Impact

$M = 828,000'^{\#}$ $I = 412,358\ in^4$ $C_c = 18.25'$, $C_s = 28.47'$

$$f_s = \frac{828,000 \times 12 \times 28.47 \times 10}{412,358}$$

$= \underline{6810\ ^{\#}/in^2}$

$$f_c = \frac{828,000 \times 12 \times 18.25}{412,358}$$

$= \underline{452\ ^{\#}/in^2}$

Case ③ Fut Wear Surf

$M = 108,500'^{\#}$ $I = 879,162\ in^4$ $C_c = 23.24'$, $C_s = 23.48$

$$f_s = \frac{108,500 \times 30 \times 12 \times 23.48}{879,162}$$

$= \underline{1040\ ^{\#}/in^2}$

$$f_c = \frac{108,500 \times 12 \times 23.24}{879,162}$$

$= \underline{35\ ^{\#}/in^2}$

Case ④ Removal of forms (w ½ w in case ③)

$f_s = \underline{520\ ^{\#}/in^2}$

$f_c = \underline{18\ ^{\#}/in^2}$

Computed By:	Detail:	Bridge Design
GWM	Stress Summary	[illegible]

Summary of Stresses

Case	Loading	f_s #/in²	f_c #/in²
①	D.L. + Forms	12600	0
	− Forms	−520	−18
	D.L.	12080	−18
	L.L. + Impact	6810	452
②	D.L. + L.L. + Imp.	18890	434
	Fut. Wear Surf.	1040	35
③	D.L. + L.L. + Imp. + F.W.S. =	19930 #/in²	469 #/in²

Spiral Shear Connectors

Use 5/8" bars with 4½" mean coil diameter

Minimum cover req'd on spiral = 1½" *

Cover = 7" − (4½ + 5/8) = 1 7/8", > allowable

Spiral Pitch:

$$s = \frac{F_w}{V}\,\frac{I}{Q}\ ^{*}$$

where:

$\underline{s}$ = pitch of spiral, c. to c. of welds in inches

$\underline{F_w}$ = shearing force transmitted per pitch

$\underline{I}$ = moment of inertia of composite section about its neutral axis.

$\underline{Q}$ = static moment of effective concrete area about neutral axis of composite section

$\underline{V}$ = total vertical shear

s = ?

$F_w = 1104^k$

$I = 546{,}535\ in^4$

$Q = (707.6 \times 12.65)\ in^3$

*Information obtained from Composite Construction Engineering Handbook of Porete Manufacturing Company.

Spiral Shear Connectors (cont.)

At end supports:

$V_{DL} = 1.2^k \times 45 \qquad = 54.0^k$

$V_{LL} = 0.64^k \times 45 + 40 \qquad = 68.8^k$

$122.8^k = V_1$

$\therefore S = \frac{11.04 \times 546{,}535}{122.8 \times 707.6 \times 12.65}$

$= 5.5''$

At 15' from supports:

$V_{DL} = 1.2^k \times 30 \qquad = 36.0^k$

$V_{LL} = 0.64^k \times \frac{75}{90} \times \frac{75}{2} + 40 \times \frac{75}{90} \qquad = 53.3^k$

$89.3^k = V_2$

$\therefore S = \frac{11.04 \times 546{,}535}{89.3 \times 707.6 \times 12.65}$

$= 7.5''$

At 30' from supports:

$V_{DL} = 1.2 \times 15 \qquad = 18.0^k$

$V_{LL} = 0.64^k \times \frac{60}{90} \times \frac{60}{2} + 40 \frac{60}{90} \qquad = 39.4^k$

$57.4^k = V_3$

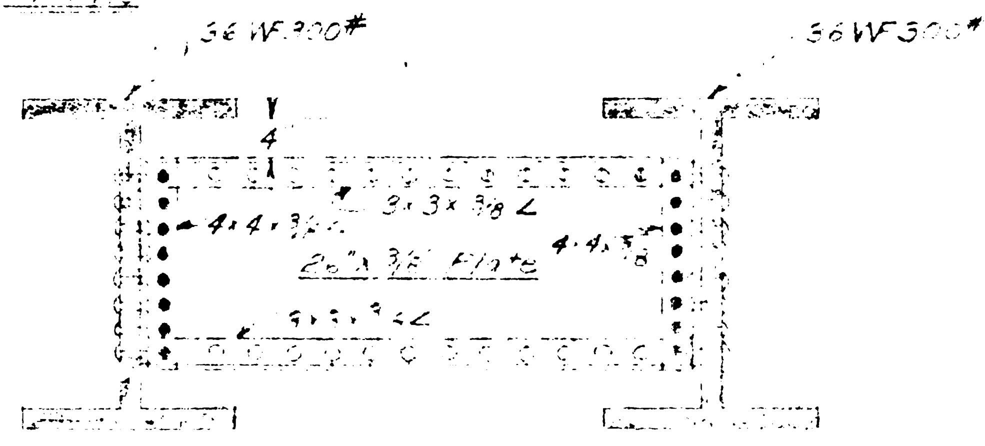
$\therefore S = \frac{11.04 \times 546{,}535}{57.4 \times 707.6 \times 12.65}$

$= 11.8''$, use $11.5''$

Spiral Shear Connectors (cont)

Spiral	No. of loops	Pitch	Spiral length c to c. of end welds	Req'd weld length*
1	33	5.5"	15.12'	1.75
2	25	7.5"	15.61'	1.75
3	16	11.5"	15.42'	1.75"

Diaphragms:



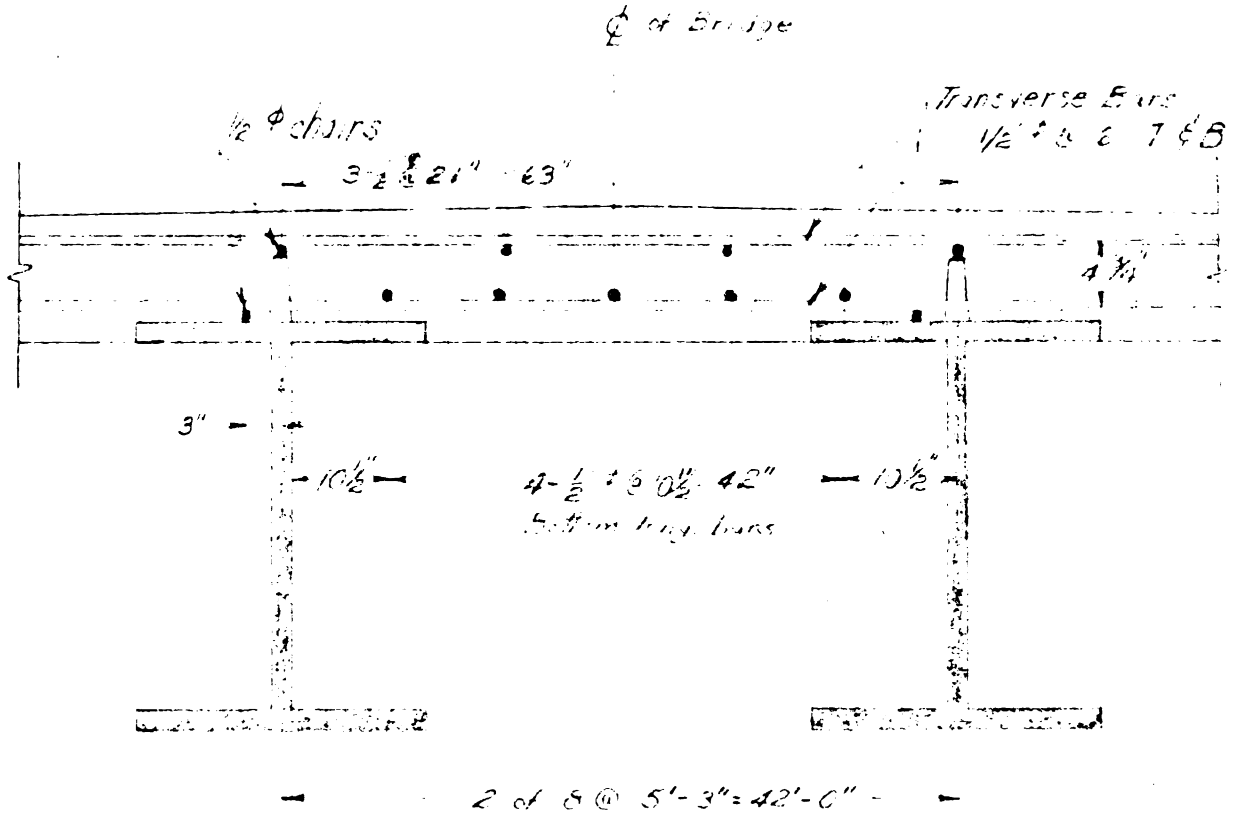
All rivets 3/4" @ max space = 4"

Diaphrams spaced @ 11'-4" c. to c.

* Porete Composite Construction Specification

GNM

Slab Reinforcing Steel



Note:

1. Reinforcement typical for all transverse sections
2. Concrete shall be flush with bottom of top flange