The Investigation of a Highway T-Beam Bridge

A Thesis Submitted to

The Faculty of

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of

AGRICULTURE AND APPLIED SCIENCE

by

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L. A. Robert

And to all other members of the Michigan State College Civil Engineering faculty in grateful appreciation of their help in preparing this volume.

Preface

My intention in choosing this subject for a Senior problem was not to investigate any new path of thought, but rather to ground myself in one of the fundamentals of Engineering. Through a short engagement with the Warren S. Holmes Co., architects, in a part time capacity in 1947, I discovered that I had a weakness. This weakness being in the field of structural design. It was not a lack of theoretical knowledge but rather of matters of practical experience such as reading blue-prints, etc. I determined at that time to avail myself of the means of correcting this matter at every opportunity while still in school.

Although this work is not of revolutionary character, I respectfully submit it with the hope that some future student of Engineering, faced with a problem like mine, will perhaps reap some benefit from it.

C. H. B.

Michigan State College
May, 1948.

Chapter I
Introduction

The problem of bridging expanses of water or impassable terraine is one of man's oldest problems. The materials have varied but the general construction has remained essentially the same. Reinforced concrete is a comparatively new development. With its additional strength and quality of being cast in place, it has made the construction of bridges and other structures a far simpler matter than it was with the more primitive masonry.

The bridge concerned in this problem is of the reinforced concrete T-Beam type. This type is quite universally used for short spans as it combines strength with a pleasing appearance and economical construction. This particular bridge is unusual because of the fact that the roadway does not run directly over the slab but is carried on top of an intervening earth fill. This allows any impact to be disregarded in the design.

Bridges generally may be divided into two main parts, the superstructure and the substructure. The superstructure of this bridge consists of a concrete roadway slab; an earth fill; and a bridge slab, T-Beams and side beams integrally cast. The substructure consists of a concrete abutment which acts as a retaining wall and also carries the superstructure. The abutment has wing walls which retain the earth at the shoulders of the bridge. Underneath the abutment there is a footing of plain tremie concrete. This merely spreads the load from the bottom of the abutment to bring the soil pressure within the allowable limits.

There are two sections of the bridge which are not designed by theoreticalpprinciples but are merely dictated by standard office practice in the Michigan State Highway Department. These are the side beams and the bridge slab. Therefore the investigation of the bridge will begin with the T-Beam.

Chapter II

Investigation of the T-Beam

Dimensions: (See Fig. I)

Compressive Flange:

The effective flange width to be used in the design of symetrical T-Beams shall not exceed one-fourth the span length of the beam, and its overhanging width on either side of the web shall not exceed eight times of the thickness of the slab nor one-half the distance to the next beam. (M.S.H.D. Specs.)

Span Length = 28' (approx.)

28'/4 = 7' or 84" Total Width

Minimum Thickness of Slab = 8"

8 x 8" = 64" Overhang.

Clear Span of Slab = 4'9" or 57"

57"/2 = 28½" Overhang (Controls.)

Total Effective Width of Flange

= 28½" x 2 -- 18" = 75"

Total Effective Thickness of Flange = 8"

Effective Tension Steel:

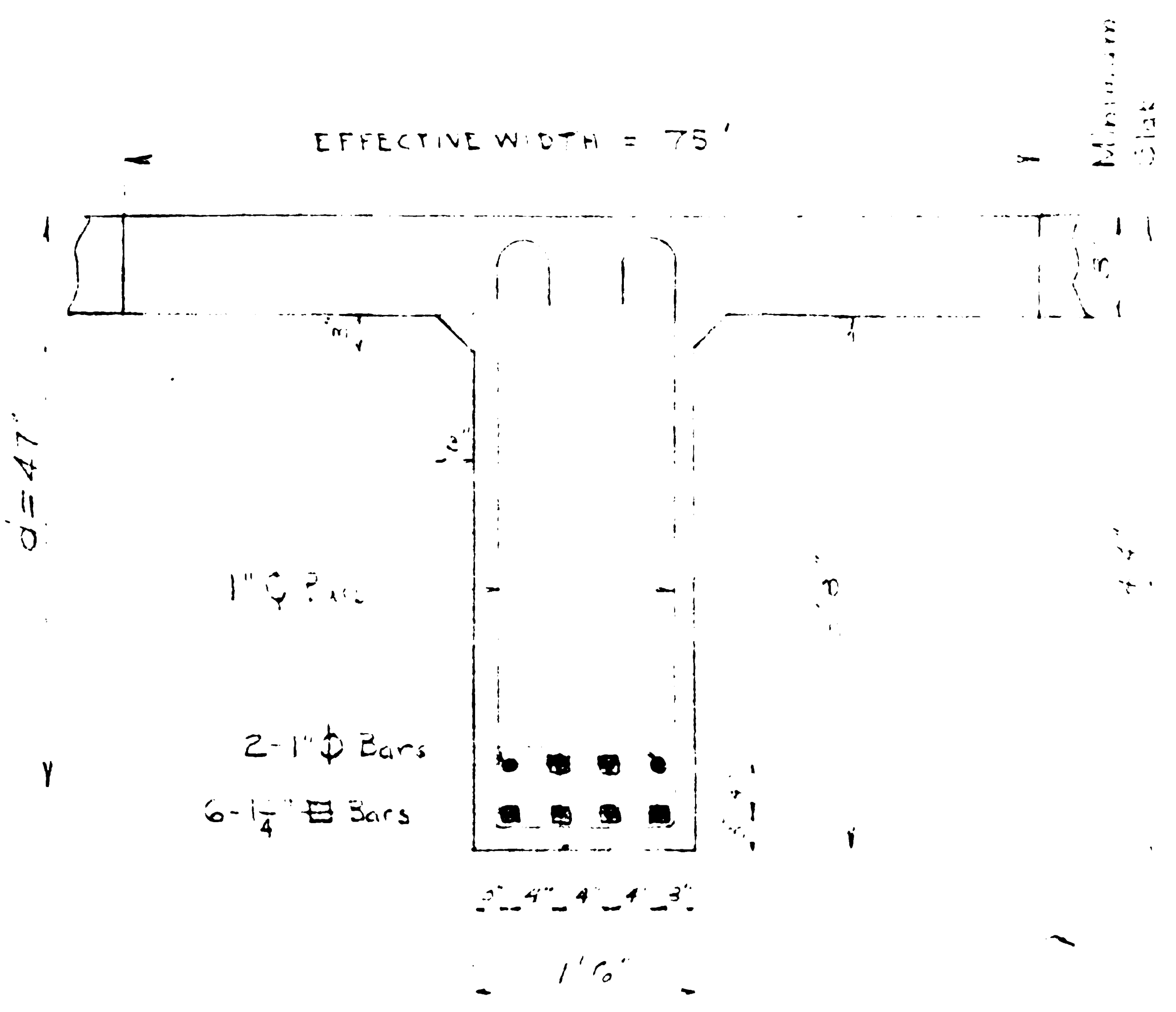
6 - 1¼" Sq. Bars Area = 6 x 1.56 = 9.36 Sq. in.

2 - 1" Rd. Bars Area = 2 x 0.79 = 1.58 sq. in.

Total Tension Steel Area = 10.94 sq. in.

Fig. I

THE T-BEAM SECTION



(Approximate Method)

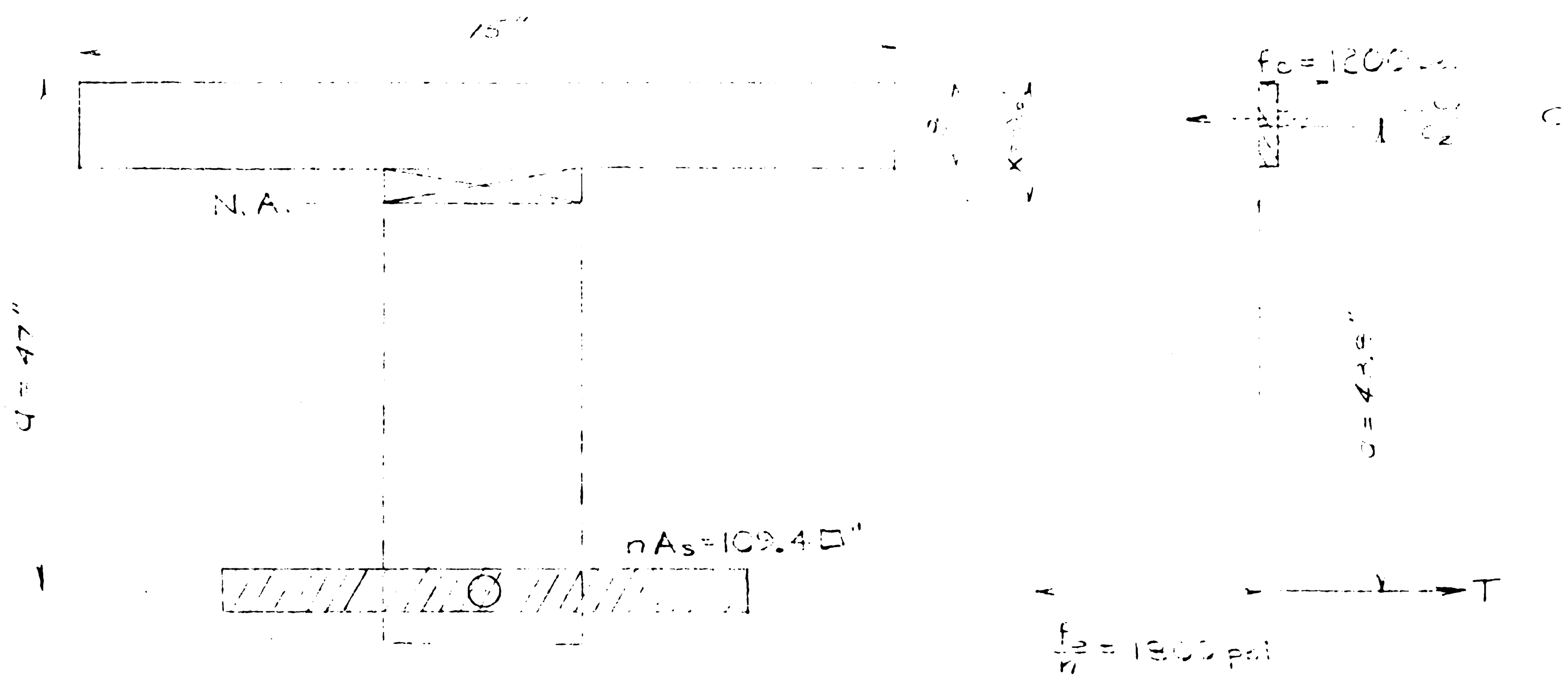


Fig. II

RESISTING MOMENT

Resisting Moment: (Approximate Method)

Moments are taken by the well known Transformed Area Method to locate the neutral axis, neglecting, however, the compressive stress in the stem of the beam.
(See Fig. II)

$M_{n.a.}$

$(75 \times 8)(X - 4) = 109.4 (47 - X)$

$600 X - 2400 = 5140 - 109.4 X$

$709.4 X = 7540$

$X = 10.6"$

The stress intensity on the underside of the slab is: $2.6/10.6 f_c = 0.245 f_c$

The total compression is equal to the sum of two forces; C_1 acting with equal intensity over the entire flange equal to $0.245 f_c$, and C_2 acting with varying intensity from zero to $0.755 f_c$.

	Compression	C	Arm	Moment about Top
C_1	$(0.245 f_c)(75 \times 8)$	$= 147.0 f_c$	4"	$588 f_c$
C_2	$\frac{1}{2}(0.755 f_c)(75 \times 8)$	$= 226.4 f_c$	2.67"	$606 f_c$
Total C		$= 373.4 f_c$		$1194 f_c$

Distance of C from Top = 3.2"

Moment Arm (a) = 47.0" - 3.2" = 43.8"

Limiting Value of Couple:

$C = 373.4 \times 1200 = 448,000$ #

$T = 10.94 \times 18,000 = 197,000$ # Use

Resisting Moment = T x a

$R. M. = \frac{197,000 \# \times 43.8"}{12} = 720,000$ '#

Shear Resistance :

$V_r = v\ b\ j\ d$

For beams with properly designed web reinforcement and special anchorage of longitudinal steel :

$v = 270$ p.s.i. (M.S.H.D. Specs. p. 60)

$b = 18"$ (Fig. I)

j (actual) $= 43.8" / 47.0" = 0.933$

j (specifications) $= 7/8$ or 0.875 Use

$d = 47.0"$ (Fig. I)

$V_r = 270$ psi. x 18" x 0.875 x 47"

$V_r = 200,000$ #

Bond Resistance :

$V_r = \mu\ \Sigma o\ j\ d$

In beams with deformed bars :

$\mu = 150$ psi.

For 6 - $1\frac{1}{4}$" Sq. Bars	$\Sigma o = 6 \times 5.0"$	=	30.00"
For 2 - 1" Rd. Bars	$\Sigma o = 2 \times 3.14"$	=	6.28"
Total Σo		=	36.28"

$V_r = 150$ psi. x 36.28" x 0.875 x 47"

$V_r = 229,000$ #

Stirrup Spacing and Bends :

The distance S measured in the direction of the axis of the beam between two successive stirrups, between two successive points of bending up or bending down of bars, or from the point of bending up of a bar to the edge of the support, shall not be greater than :

$$S = \frac{45\ d}{X + 10}$$ In which :

X = The angle of the bar with the axis of the beam

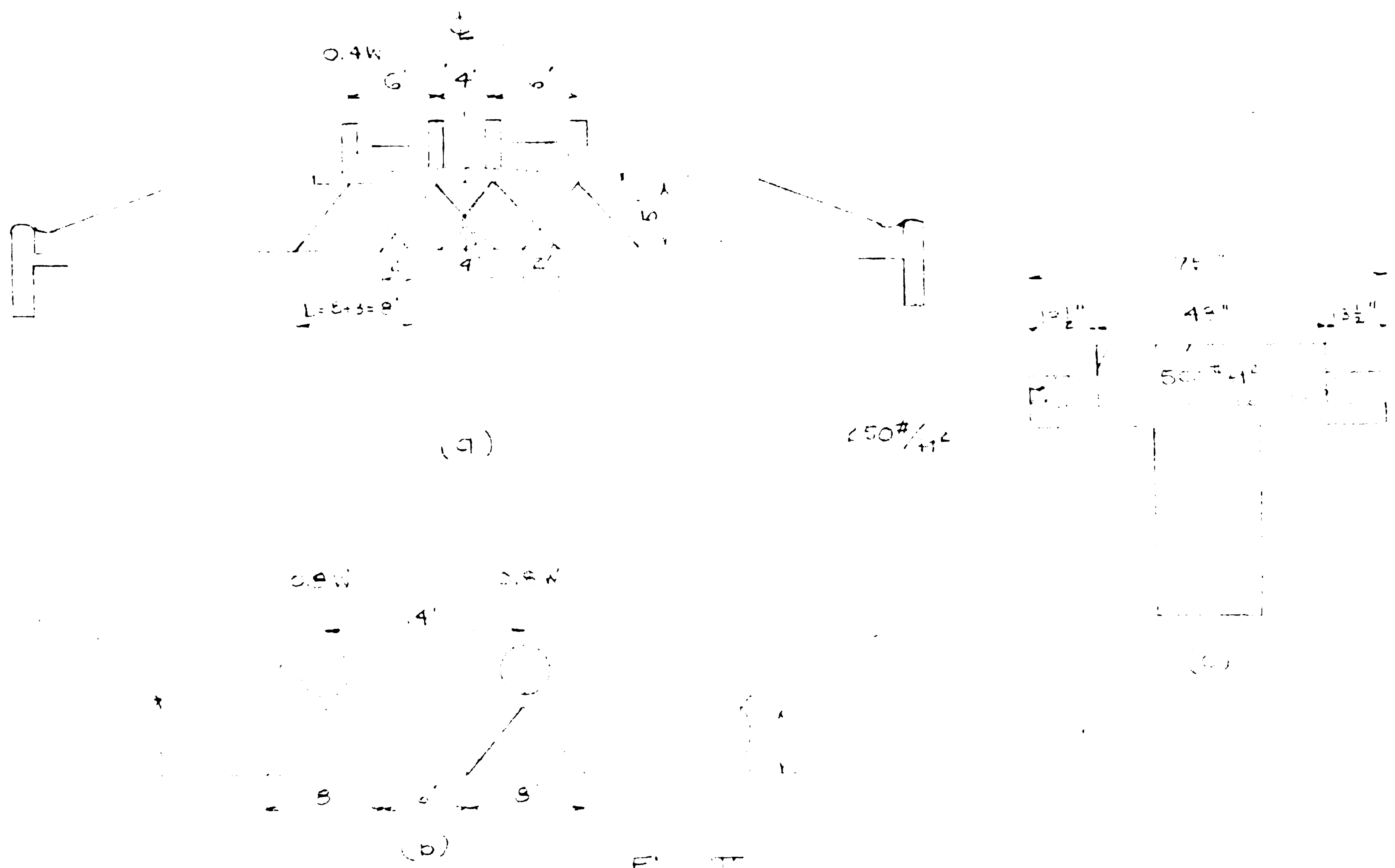


Fig. III
LIVE LOADING

expressed in degrees. (M.S.H.D. Specs. Art. 65 f)

Stirrups ;

$$S = \frac{45 \times 47}{90 + 10} = 21.2''$$

Bends :

It is unnecessary to apply this specification when they are used concurrently with stirrups.

The distance from the edge of the support to the first vertical stirrup shall not exceed S/2 as given by the above formula. (ibid. p. 4)

$$S/2 = \frac{21.2''}{2} = 10.6''$$

<u>Loading on Beam</u> :

Only the loads are considered which act directly on one T-Beam, the center one being the maximum case.

Dead Load :

Wt. of Concrete Pavement	$= \frac{75 \times 9\frac{1}{2}}{144} \times 150$	= 750#/'
Wt. of Earth Fill	$= \frac{75 \times 60}{144} \times 100$	= 3125#/'
Wt. of Beam Flange	$= \frac{75 \times 9}{144} \times 150$	= 700#/'
Wt. of Beam Stem	$= \frac{18 \times 44}{144} \times 150$	= 825#/'
Total Dead Load per Ft. of Beam		= 5400#/'

Impact :

No impact allowance shall be made for forces on substructures, nor on superstructures on which the loads are separated from the supporting slab * * * by an intervening fill (including pavement) of two feet or more. (M.S.H.D. Specs. Art. 37)

Distribution of Concentrated Loads Through Fill :

Concentrated loads on concrete pavement may be assumed as uniformly distributed over an area below the pavement whose lateral and longitudinal dimensions are given by the following formula :

$L = d + 3$ where :

$L =$ lateral or longitudinal distribution in feet.

$d =$ depth of fill below pavement to plane of distribution.

$L = 5.00 + 3 = 8.00'$

When the areas thus determined for concentrated loads overlap, the pressures on the overlaping portions shall be taken as the combined pressure from each such load. (M.S.H.D. Specs. Art. 45)

Live Load :

The live load shall consist of a train of standard motor trucks in each traffic lane as hereinafter specified :

Minimum Loading for Class AA Bridges = H 20 , S 16 Loading. (See Fig. III & A.A.S.H.O. Specs.)

Concentrated Load for one rear wheel of Tractor
= 0.4 W or 8 ton = 16,000 #

Distributed Load for one rear wheel of Tractor
= 16,000#/ 8' x 8' = 250 #/sq. ft. (Fig. III a & b)

Double for overlap of 48" (Fig. IIIc)

Load per foot of beam for rear wheels of Tractor :

4 x 250 x 2	= 2000 #/'
27/12 x 250	= 560 #/'
Total load per foot	= 2560 #/'

Note: The front wheel of the trailer has the same loading as the rear wheel of the tractor, and lies 14' from it.

Bending Moment :

Since the ratio between the loadings of the two axels is one to one, the maximum bending moment will occur when the rear wheel of the tractor and the front wheel of the trailer are equidistant from the centerline of the bridge. The effects of all other wheels fall off the bridge.

See Fig. IVa

Effective span for moment = 27' 10¾" or 27.90'

$R_1 = R_2$

Taking moments about R_2

$27.9\ R_1 = 13.95 \times 27.9 \times 5400 + 6.95 \times 8.0 \times 2560 + 20.95 \times 8.0 \times 2560$

$27.9\ R_1 = 2,671,000$	:	2560×8.0	$= 20,500\#$
$R_1 = 95,800\ \#$	:	5400×13.95	$= 75,300$
			95,800# Ck

$B.M. = 95,800 \times 13.95 - 2560 \times 8 \times 7 - 5400 \times 13.95 \times 6.98$

B.M. = 666,000 '# O.K.

Specifications Check ;

The bending moment carried by each interior beam or stringer shall be taken not less than that determined by the following formulae :

M = Bending moment for one traffic lane.

$N = \dfrac{\text{Width of Traffic Lane (not to exceed 10')}}{\text{Spacing of stringers or beams}}$

C = Coefficient based on type of floor.

M'= Bending moment on one beam or stringer.

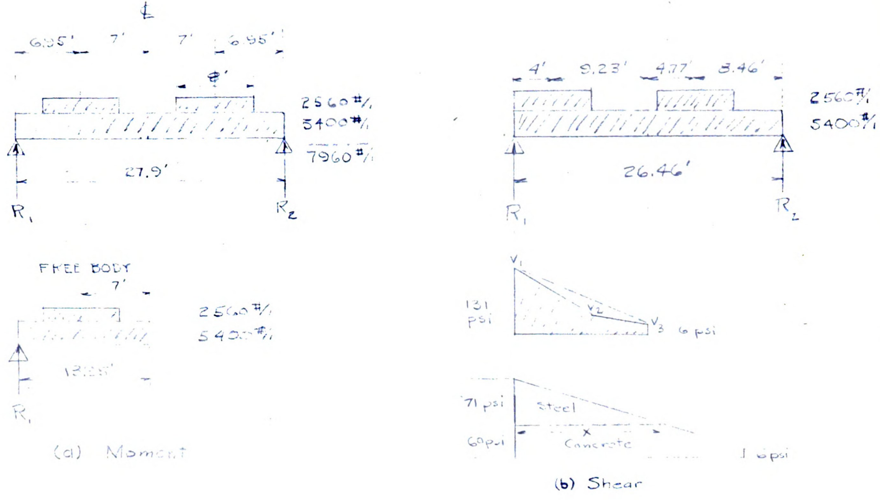


Fig. IV Shear & Bending Moment

The Formula;

$$M' = C \frac{M}{N}$$

Values of C shall be as follows :

Reinforced Concrete Slab - - - - - - - - C = 1.0

(M.S.H.D. Specs. Art. 43)

Bending Moment for one 10' Traffic Lane :

Dead :

Load/ft. = 120/75 x 5400 = 8640#/'

B.M. = $WL^2/8$ = 8640 $(27.9)^2/8$ = 840,000'#

Live:

Load/axel = 32,000-250x32 = 24,000#

B.M. = P x a = 24,000 x 6.95 = 167,000'#

Total B.M./10' Lane = M = 1,007,000'#

$$M' = C \frac{M}{N} = 1.0 \times \frac{1,007,000}{10/6.25} = 630,000'\# \quad \text{O.K.}$$

<u>Shear and Web Reinforcement</u> :

Place the live load so that the edge of the distributed load from the rear axel of the tractor is at the end of the beam and both the rear axel of the tractor and the front axel of the trailer fall on the beam.

Effective span for Shear = 26' 5½" or 26.46'

See Fig. IVb

Taking moments about R_2 ;

$26.46\ R_1 = 13.23 \times 26.46 \times 5400 + 8.46 \times 8 \times 2560 + 22.46 \times 8 \times 2560$

$26.46\ R_1 = 2,558,000$

$R_1 = 96,600$ #

$V_1 = R_1 = 96,600$ #

$V_2 = 96,600 - 8 \times 7960 = 32,900$ #

$V_3 = 32,900 - 5.23 \times 5400 = 4,700$ #

$jd = 7/8 \times 47" = 41.2"$

$v_1 = \frac{V_1}{bjd} = \frac{96,600}{18 \times 41.2} = 131$ psi.

$v_2 = \frac{32,900}{18 \times 41.2} = 44$ psi.

$v_3 = \frac{4,700}{18 \times 41.2} = 6$ psi.

Assume a shear trapazoid with a maximum ordinate of 131 psi. and a minimum ordinate of 6 psi., and allow 60 psi. for tension in the concrete as shown in Fig. IVb.

Stirrup Spacing :

The stirrup spacing is worked out by the method shown on page 30 of the Reinforced Concrete Design Handbook (A.C.I.).

See Fig. IVb

$131 - 6 = 125$ psi.

$131 - 60 = 71$ psi

$S = 13.23 \times \frac{71}{125} \doteq 7.5'$ or 7'6"

Given :

Max. $v' = 71$ psi.

$S = 7'\ 6"$; $b = 18"$; $d = 47"$

$f_v = 14,000$ psi. ; $f'_c = 3000$ psi.

½" Rd. U Stirrups

From Diagram 17 (A.C.I.) for $f_v = 14,000$ and ½" Rd. U Stirrups : $A_v f_v = 5600$

$$\max. \frac{1}{s} = \frac{(\max. v')b}{A_v f_v} = \frac{71 \times 18}{5600} = 0.228$$

$$N = 6S \left(\max. \frac{1}{s} \right) = 6 \times 7.5 \times 0.228 = 11 \text{ Stirrups}$$

There are 12 Stirrups in the end 7'6" spaced : 1 @ 2 3/8", 1 @ 4", 4 @ 5", 4 @ 7", and 2 @ 1'4".

12 Stirrups in 7' 2 3/8" Check.

The remaining stirrups are supplied to comply with Art. 65 f ; M.S.H.D. Specs. They are spaced : 2 @ 1'4", 2 @ 1'8", plus 1 @ 1/8" off center.

4 Stirrups in 6'2"

6'2" + 7' 2 3/8" = 13' 4 3/8" Check.

Summary and Comparison of Results :

Resisting Moment of T-Beam Section	= 720,000'#
Max. Bending Moment, H 20, S 16 Loading	= 666,000 '#
Shear Resistance of T-Beam Section	= 200,000 #
Max. Shear, H 20, S 16 Loading	= 96,600 #

The beam is amply strong to resist both shear and bending moment. Diagonal tension is adequately resisted by the stirrups alone.

Chapter III

Investigation of the Abutments

The investigation of the abutments follows the office practice of the Michigan State Highway Department. The practice is to select a section and check it for sliding overturning. This allows it to be readily adapted to this problem.

Loading :

Superstructure Dead Load :

Total Concrete = 145.0 cu. yds. (See Plans, Sheet 6)

145 x 27 x 150 = 587,000 #

Earth Fill = 51.25 x 5 x 28.5 = 7300 cu.ft.

7300 x 100 = 730,000 #

Pavement = 22 x 0.79 x 28.5 = 495 cu.ft.

495 x 150 = 74,300 #

Total Dead Load = 1,391,300 #

Length of Main Wall = 65.75'

Then ; $\frac{1,391,300}{65.75 \times 2}$ = 10,600 #/ft. of Abutment

Superstructure Live Load : (A.A.S.H.O. Specs. p.245)

Use 27.9' span.

Note : Moving the wheels closer than 4 ft. from the end of the bridge diminishes the reaction due to distribution through fill.

$27.9 R_1 = 23.9 \times 32,000 + 9.9 \times 32,000$

$27.9 R_1 = 1082,000$

R_1 = Max. Reaction for One Lane = 38,800 #

It's possible to have 4 lanes loaded.

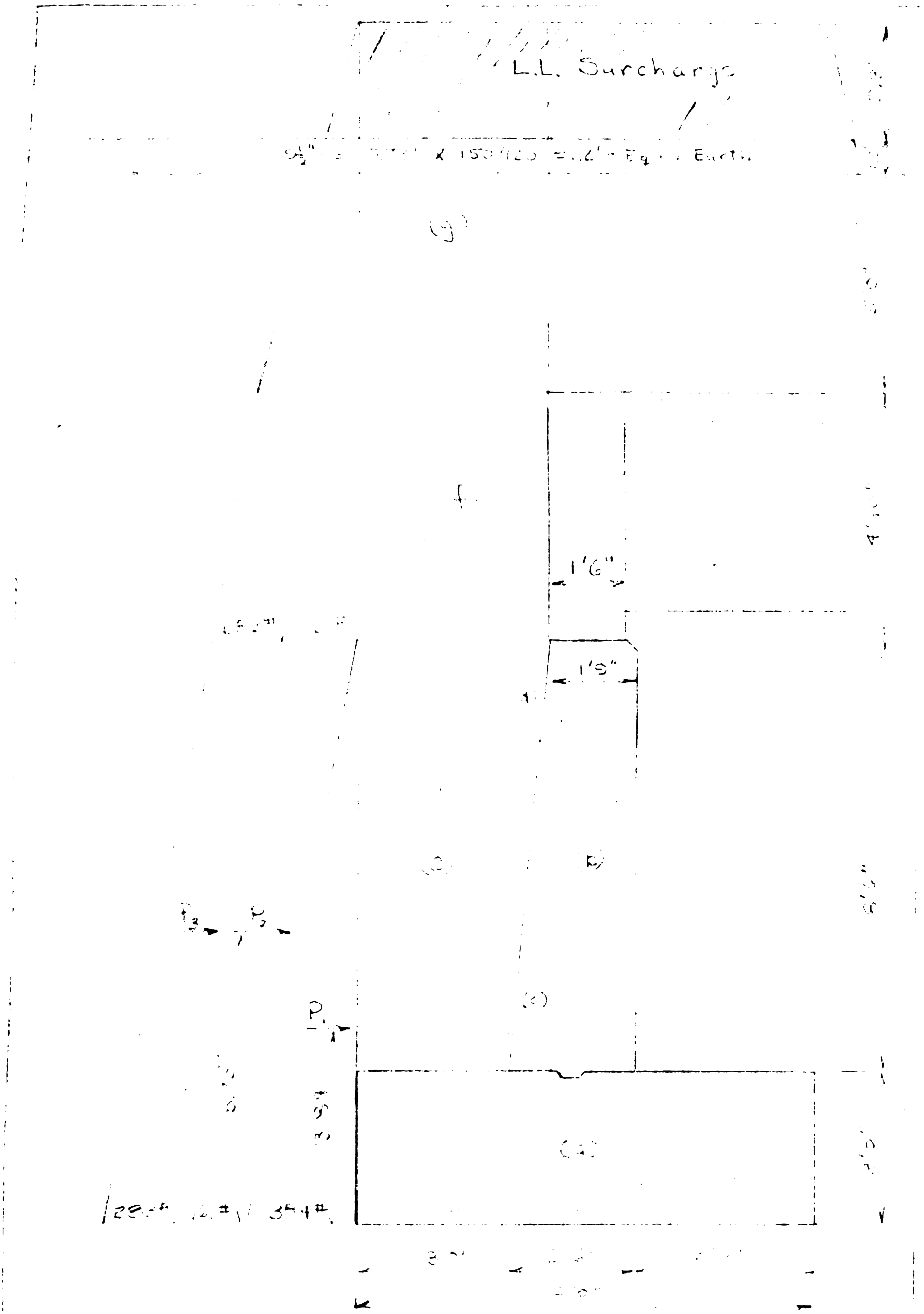


Fig. V Abutment - Main Wall

Total L.L. on Abutment = 38,800 x 4 x 0.8 = 124,000#
(M.S.H.D. Specs. Art. 33)

124,000 / 65.75 = Say 1900 #/ft. of Abutment.

Surcharge : (M.S.H.D. Specs. Art. 39)

Inside measure, Guardrails = 38.0'

Then : $\frac{38.0 \times 400}{65.75}$ = Say 230 #/ft. of Abutment

<u>Overturning and Stablity, Main Wall</u> : (See Fig. V)

Moments are taken about the toe of the section (one foot thick). The line of action of the resultant force is determined. This must lie within the middle third of the base.

w = 100 #/cu.ft. (earth) C = 1/3 (Rankin)

Overturning :

		Thrust			Moment
P_1 = 384# x 5.75'	=	2210#	x 3.84'	=	8,500'#
P_2 = 161 x 11.5	=	1850#	x 5.75	=	10,600'#
		M_{oI} = M_{oIII}		=	19,100'#
P_3 = 283 x 11.5	=	3260#	x 5.75	=	18,800'#
		M_{oII}		=	37,900'#

Stability :

	Weight		Moment
(a) 3.0 x 9.0 x 150	= 4050#	x 4.50	= 18,200'#
(b) 1.75 x 8.5 x 150	= 2230	x 4.38	= 9,800
(c) 0.375 x 8.5 x 150	= 478	x 5.50	= 2,630
(d) 0.375 x 8.5 x 100	= 318	x 5.75	= 1,830
(e) 3.0 x 8.5 x 100	= 2550	x 7.50	= 19,100
(f) 3.75 x 4.83 x 100	= 1810	x 7.13	= 12,900
(g) 3.75 x 6.2 x 100	= 2320	x 7.13	= 16,550
W_I	= 13,756#	M_{sI}	= 81,010'#
D.L.	10,600	x 4.50	= 47,700
W_{IV}	= 24,356#	M_{sIV}	= 128,710'#
Surch. = 3.75 x 2.3	= 864	x 7.13	= 6,150
W_{II}	= 25,220#	M_{sII}	= 134,860'#

	Weight		Moment
W_{IV}	= 24,356#	M_{sIV}	= 128,710'#
L.L.	= 1,900	x 4.5	= 8,550
W_{III}	= 26,256#	M_{sIII}	= 137,260'#

Case I : No Superstructure Load or Live Load Surcharge :

M_{sI} = 81,010 '#

M_{oI} = 19,100

M_I = 61,910 '#

$$R_I = \frac{M_I}{W_I} = \frac{61,910}{13,756} = 4.50'$$

Case II : Superstructure Dead Load and Live Load Surcharge.

$$M_{sII} = 134,860 \text{ '\#}$$
$$M_{oII} = 37,900 \text{ '\#}$$
$$M_{II} = 96,960 \text{ '\#} \qquad R_{II} = \frac{96,960}{25,220} = 3.87' \text{ O.K.}$$

Case III : Superstructure Dead and Live Load - No Surcharge.

$$M_{sIII} = 137,260 \text{ '\#}$$
$$M_{oIII} = 19,100 \text{ '\#}$$
$$M_{III} = 118,160 \text{ '\#} \qquad R_{III} = \frac{118,160}{26,256} = 4.50' \text{ O.K.}$$

Case IV : Superstructure Dead Load - No Live Load or Horizontal Earth Thrust.

$$R_{IV} = \frac{M_{sIV}}{W_{IV}} = \frac{128,710}{24,356} = 5.30' \quad \text{O.K.}$$

In Cases I and III the Resultant forces fall exactly on the centerline of the base which gives evenly distributed base pressures. In Cases II and IV the Resultants fall within the middle third of the base This meets specifications to prevent excessive crushing and gives the abutment stability even though the toe or heel may not be bearing completely on the surface underneath.

Sliding :

The sliding resistance is computed by the friction formula; $F = \mu N$ where; $N = W$. The coefficient of friction (μ) for sandy clay (see Log of Borings , esp. T.H. 2, Sheet 2. Plans) was assumed by examining Table 4, p.63, M.S.H.D. Specs.

$\mu = 0.40$ for sandy clay.

Case I : $W_I = 13,756$ #

$F_I = 0.40 \times 13,756 = 5500$ #

$P_I + P_2 = 2210 + 1850 = 4060$ #

F_I is greater than $P_1 + P_2$ O.K.

Case II : $W_{II} = 25,220$ #

$F_{II} = 0.40 \times 25,220 = 10,090$ #

$P_1 + P_2 + P_3 = 4060 + 3260 = 7320$ #

F_{II} is greater than $P_1 + P_2 + P_3$ O.K.

Base Pressure :

The base pressure is computed by taking moments about the centerline of the base and using the formula:

$$S = \frac{W}{a} \pm \frac{M\,c}{I}$$ where:

S = Base Pressure W = Vertical Load (weight)

a = Area of Base M = Moment about Centerline

c = Distance between Centerline & Edge of Base

I = Moment of Inertia about Centerline

$$I = \frac{bh^3}{12} = \frac{1 \times (9)^3}{12} = 60.7 \text{ ft.}^4$$

Case II ;

	Thrust		Moment
$P_1 =$	2210 #	x 3.84'	= 8,500'#
$P_2 =$	1850	x 5.75	= 10,600
$P_3 =$	3260	x 5.75	= 18,800
Total			= — 37,900'#

	Weight		Moment
(a)	4050 #		none
(b)	2230	x 0.125 =	− 279'#
	M_{oII}	=	+37,900
Total Overturning Moment		=	−38,179'#
	+		
(c)	478#	x 1.00 =	+ 478'#
(d)	318	x 1.25 =	398
(e)	2550	x 3.00 =	7,650
(f)	1810	x 2.625 =	4,760
(g)	2320	x 2.625 =	6,100
Total =	13, 756 #		+19,386'#
D.L.	10, 600 #		none
Surch.	864 #	x 2.625 =	+ 2,270'#
W_{II} =	25, 220 #		
Total Righting Moment		=	+21,656'#

Resultant Moment =

$$- 38,179 + 21,656 = - 16,523'\#$$

$$S = \frac{25,220}{9} \pm \frac{16,523 \times 4.5}{60.7} = 2800 \pm 1250 \text{ psf.}$$

S toe = 4050 psf.

S heel = 1550 psf.

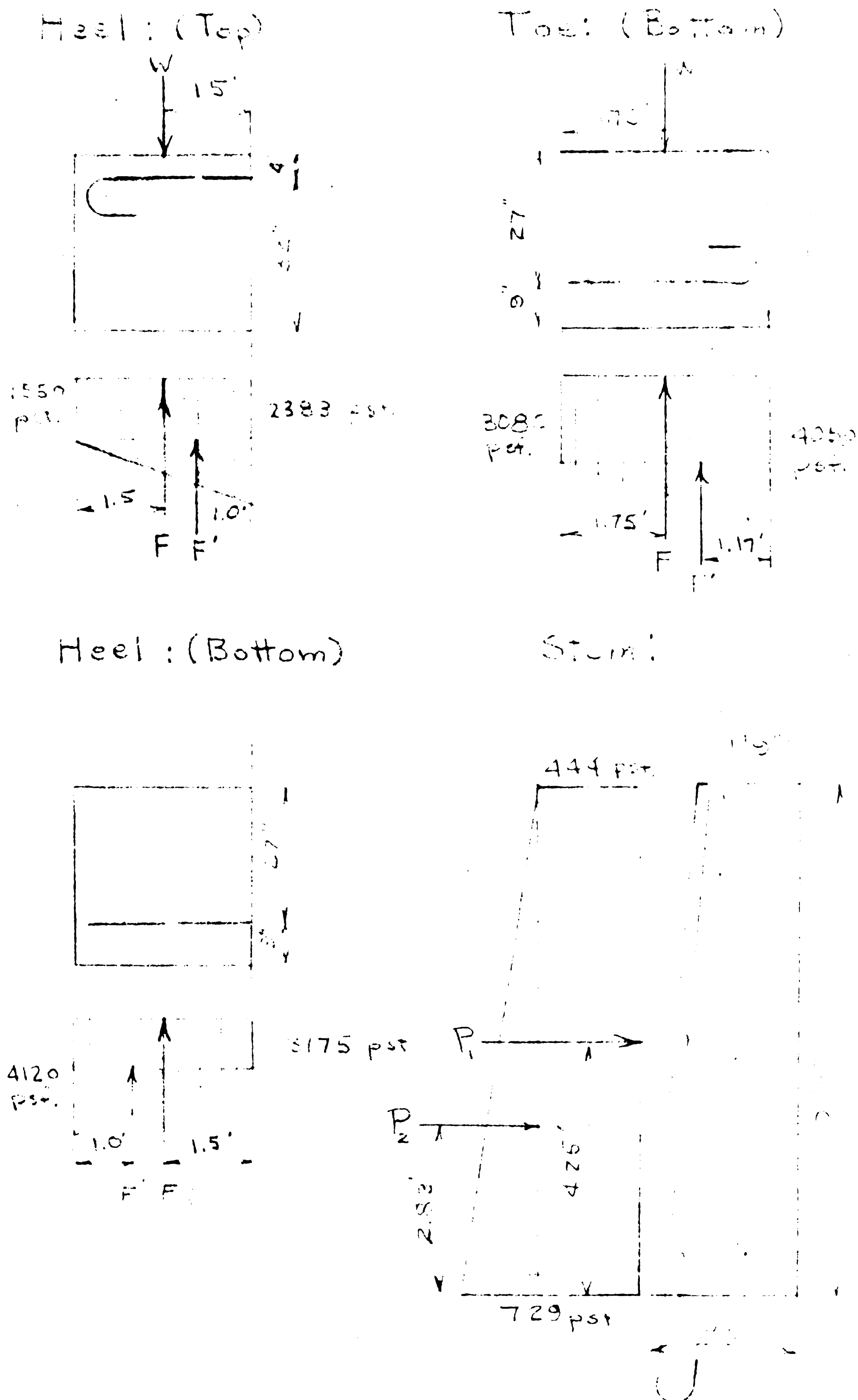


Fig. VI Abutment Steel

Case IV :

W_{IV} = 24,356 #

Resultant Moment = 19,386 — 279 = + 19,107'#

$$S = \frac{24,356}{9} \pm \frac{19,107 \times 4.5}{60.7} = 2700 \pm 1420 \text{ psf.}$$

S toe = 1280 psf.

S heel = 4120 psf

<u>Check for Steel</u> : (See Fig.VI)

Heel (Top) Case II :

W = 3.0 x 21.83 x 100 + 9.0 x 150 = 7900#

Moment = 7900 x 1.5 = 11,900'#

		Force		Moment
F = 1550 x 3.0	=	4650#	x1.5	= 6980'#
F' = 833 x 1.5	=	1250	x 1.0	= 1250
Total		5900#		8230'#

B.M. = 11,900 — 8,230 = 3,670'# or 44,000"#

d = 32.0"

$$A_s = \frac{M}{f_s\, jd} = \frac{44,000}{18,000 \times 0.875 \times 32} = 0.088 \text{ sq.in./ft.}$$

1" Rd. Bars were used, spaced 2'6" o.c.

A_s = 0.79/2.5 = 0.316 sq.in./ft.

V = 7900 — 5900 = 2000#

$$v = \frac{V}{bjd} = \frac{2000}{12 \times 0.875 \times 32} = 6 \text{ psi.}$$ which is less than 60

$$\mu = \frac{V}{\Sigma o\, jd} = \frac{2000}{1.25 \times 0.875 \times 32} = 57 \text{ psi.}$$

$\Sigma o = 3.14/2.5 = 1.25"$

which is less than 150 psi.

No special anchorage required except by standard office practice.

Heel (Bottom) Case IV;

$W = 19.53 \times 3.0 \times 100 + 9.0 \times 150 = 7210\#$

Moment $= 7210 \times 1.5 = 10,800'\#$

		Force		Moment
F	$= 3175 \times 3.0 =$	9,530#	x 1.5	= 14,300'#
F'	$= 945 \times 1.5 =$	1,420	x 2.0	= 2,840
Total		10,950#		17,140'#

B.M. $= 17,140 - 10,800 = 6,340'\#$ or $76,000"\#$

$d = 27.0"$

$$A_s = \frac{M}{f_s\, jd} = \frac{76,000}{18,000 \times 0.875 \times 27} = 0.179 \frac{\text{sq.in.}}{\text{ft.}}$$

1" Rd. Bars were used, spaced 2'6" o.c.

$A_s = 0.316$ sq.in./ft.

$V = 10,950 - 7,210 = 3,740\ \#$

$$v = \frac{3740}{12 \times 0.875 \times 27} = 13 \text{ psi.}$$ is less than 60 psi.

$$\mu = \frac{3740}{1.25 \times 0.875 \times 27} = 127 \text{ psi.}$$ is less than 150 psi.

No special anchorage required.

Toe (Bottom) Case II :

$W = 3.0 \times 3.5 \times 150 = 1575$ #

Moment $= 1575 \times 1.75 = 2760$'#

		Force		Moment
F	= 3080 x 3.5	= 10,800 #	x 1.75	= 18,900'#
F'	= 970 x 1.75	= 1,700	x 2.33	= 3,970
Total		12,500 #		22,870'#

B.M. $= 22,870 - 2,760 = 20,110$'# or 242,000"#

$$A_s = \frac{M}{f_s\, jd} = \frac{242,000}{18,000 \times 0.875 \times 27} = 0.57 \ \frac{\text{sq.in.}}{\text{ft.}}$$

1" Rd. Bars were used, spaced 2'6" o.c.

$A_s = 0.316$ sq.in./ft.

Note : It is unlikely that the above discrepancy in the area of steel is the result of a mistake on the part of the designer. It is more probable that the weight of the earth above the toe was allowed for in the original design, although this is not a customary practice.

$V = 12,500 - 1,575 = 10,925$ #

$$v = \frac{10,925}{12 \times 0.875 \times 27} = 40 \text{ psi. is less than } 60 \text{ psi.}$$

$$\mu = \frac{10,925}{1.25 \times 0.875 \times 27} = 375 \text{ psi. (see above note)}$$

<u>The section does not check.</u>

Stem Case II ;

Taking moments about the toe of stem.

	Thrust		Moment
P_1 = 444 x 8.50	= 3780 #	x 4.25	= 16,000'#
P_2 = 285 x 4.25	= 1210	x 2.83	= 3,420
Total	4990 #		19,420'#

	Weight		Moment
(a) 1.75 x 8.5 x 150	= 2230 #	x 0.875	= 1,950'#
(b) 0.75 x 4.25 x 150	= 478	x 2.0	= 960
Total			2,910'#

B.M. = 19,420 — 2,910 = 16,510'# Or 198,000"#

j for stems = 0.98 (M.S.H.D.)

d = 27.0"

$$A_s = \frac{M}{f_s\, jd} = \frac{198,000}{18,000 \times 0.98 \times 27} = 0.40 \text{ sq.in./ft.}$$

1" Rd. Bars were used, spaced 2'0" c.c.

A_s = 0.395 sq.in./ft. O.K.

V = 4,940 #

$\Sigma o = 3.14/2.0 = 1.57"$

$$v = \frac{4940}{12 \times 0.98 \times 27} = 16 \text{ psi. is less than 60 psi.}$$

$$\mu = \frac{4940}{1.57 \times 0.98 \times 27} = 118 \text{ psi is less than 150 psi.}$$

No special anchorage required except by standard office practice.

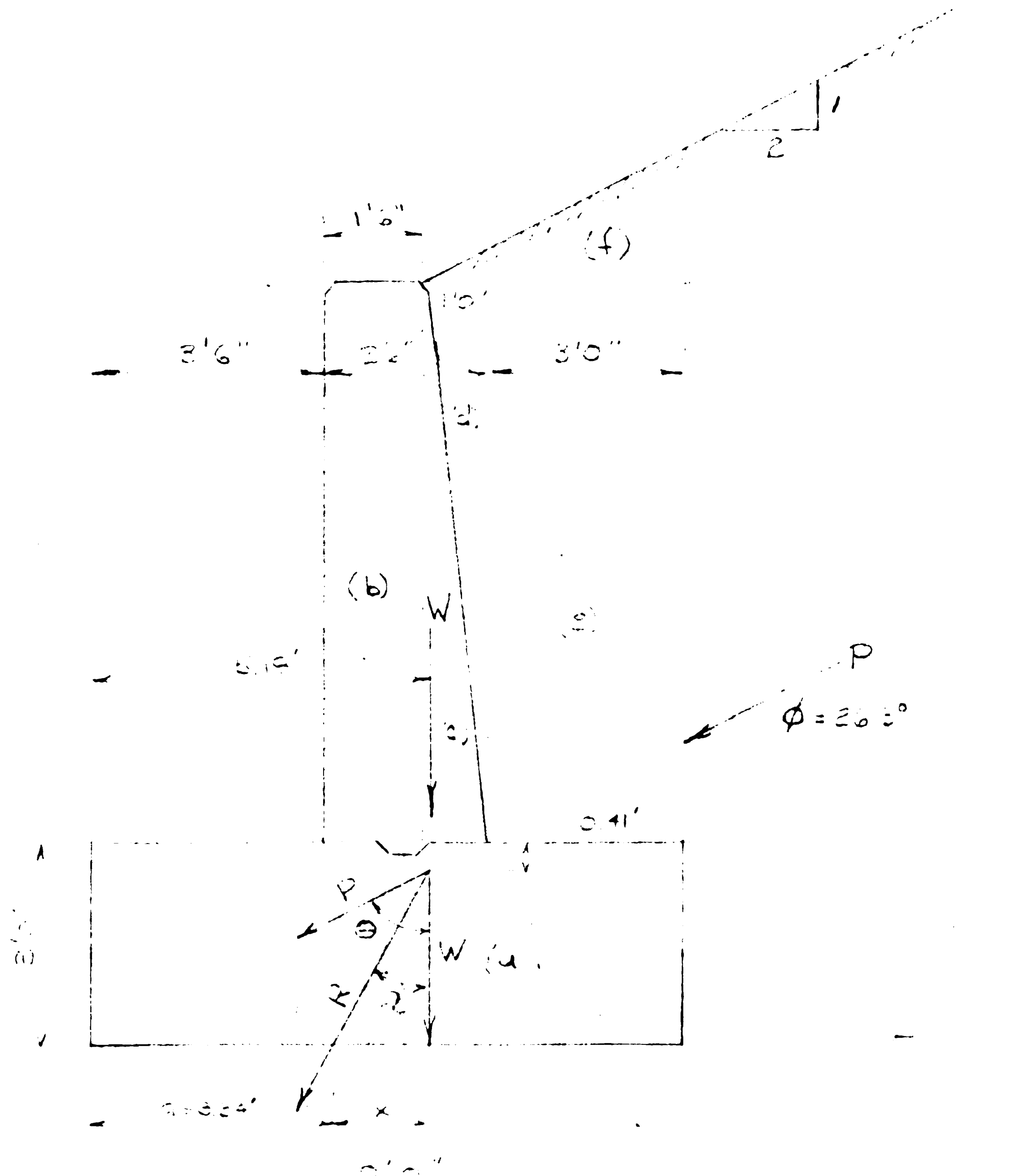


Fig. VII Abutment - Wing Wall

Overturning and Stablity, Wing Wall : (See Fig. VII)

Taking moments about the toe;

Overturning :

$P = \frac{1}{2} w (h + h')^2 \cos \phi$

$P = \frac{1}{2} \times 100 (11.5 + 2.0) \cos 26.3^\circ = 8,160\#$

Stablity :		Weight		Moment
(a)	3.0 x 9.0 x 150	= 4050 #	x 4.50	= 18,250'#
(b)	1.5 x 8.5 x 150	= 1915	x 4.25	= 8,140
(c)	1.0 x 4.25 x 150	= 637	x 5.33	= 3,400
(d)	1.0 x 4.25 x 100	= 425	x 5.67	= 2,410
(e)	3.0 x 8.5 x 100	= 2550	x 7.50	= 19,100
(f)	2.0 x 2.0 x 100	= 400	x 6.33	= 2,520
Total		9977 #	)	51,820'#

D = Distance of Resultant from Toe = 5.18'

By the Cosine Law : (modified)

$R^2 = a^2 + b^2 + 2 a b \cos \theta \qquad \theta = 63.7^\circ$

$R^2 = (8160)^2 -- (9977)^2 -- 2 \times 8160 \times 9977 \times \cos 63.7^\circ$

$R^2 = 238,300,000 \qquad R = 15,440 \#$

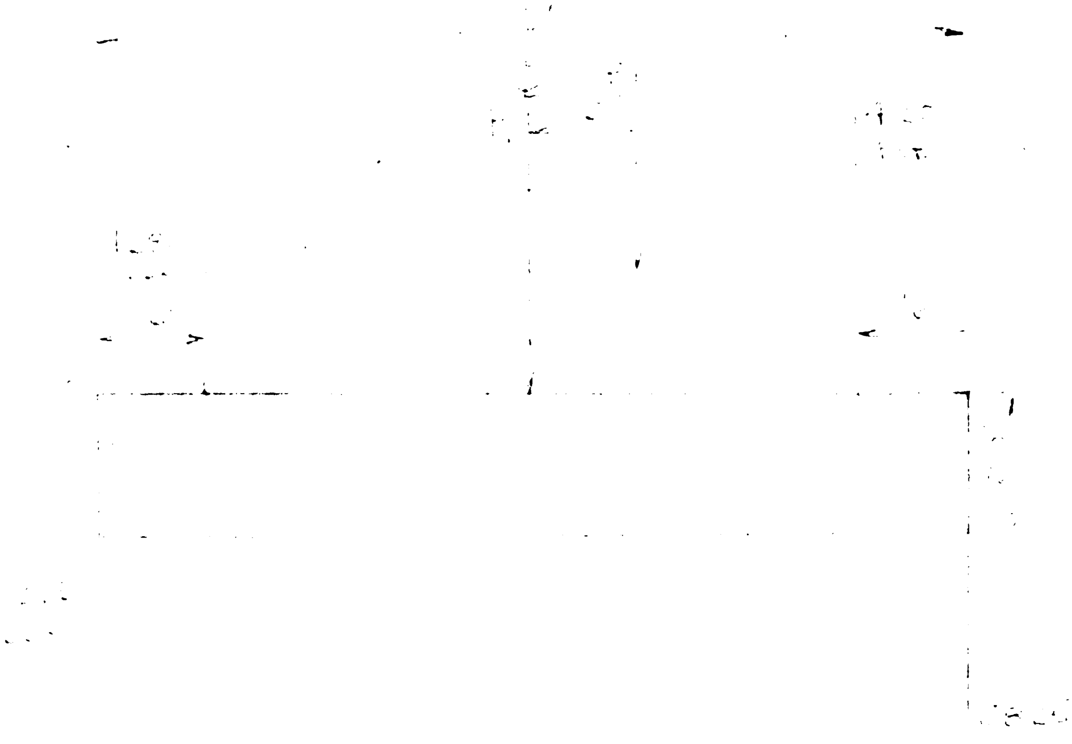
$\tan C = \frac{a \sin \theta}{b + a \cos \theta} = \frac{9977 \times \sin 63.7^\circ}{8160 + 9977 \cos 63.7^\circ}$

$\tan \theta = \frac{8940}{12,580} = 0.711 \qquad \theta = 35.4^\circ$

$X = 2.59 \times \tan 35.4^\circ = 1.84'$

$a = D - X = 5.18 - 1.84 = 3.34'$ from Toe.

Resultant falls within the middle third. O.K.



Steel; Wing Wall:

Due to the lack of time the wing wall steel will not be checked.

Soil Pressure:

Taking moments about the centerline of the footing. (See Fig. VIII).

		load	moment
P_1 =	1280 x 9.0 =	11,500#	none
P_2 =	2840 x 4.5 =	12,800	19,200'#
Total		24,300	19,200'#

$$s = \frac{P}{A} \pm \frac{Mc}{I} \qquad c = 6.0'$$

$$I = \frac{bh^3}{12} = \frac{1.0 \times (12)^3}{12} = 144 \text{ ft}^4$$

$$S = \frac{24,300}{12} \pm \frac{19,200 \times 6}{144} = 2025 \pm 800 \text{ psf}$$

S max = 2825 psf.

Allowable Soil Pressure = 2 tons / sq. ft. O.K.

(M.S.H.D. Specs. p. 63)

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