A Check of the Design of an I-Beam Highway Bridge

A Thesis Submitted to

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of

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by

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Introduction

The purpose of this thesis is to check the design of a single span simply supported highway bridge.

This bridge as proposed will be constructed on the relocation of U.S. highway number 31, also to be known as the Muskegon East Belt Road, where it crosses Black Creek in Fruitport Township, Muskegon County, in the state of Michigan. Previous to the opening of this new highway it has been necessary for all thru traffic to travel over the already congested streets of Muskegon. Thus, by the construction of the East Belt Road this situation will be greatly alleviated.

The centerline of this highway makes a 56 degree angle with the abutment wall which runs parallel to the flow of the Black Creek. The span length from center to center of bearings is 57'-1" exact and the overall width of roadway from curb to curb is 42'-0" exact.

The drainage area tributary in this crossing is 56 square miles and the waterway area required based on Talbot's formula (using $c = 0.1$) is 260 square feet.

The existing bridge one mile downstream has a tributary drainage area of 57.5 square miles, provides 246 square feet and appears adequate. The proposed structure provides 238 square feet.

All of the above mentioned conditions concerning the site and the river were taken into account both in the analysis and the design of the bridge.

Symbols

A	Area
b	breath or width
d	depth of beam to center of steel
e	eccentricity of application of load
f	unit stress
I	moment of inertia
j	ratio of lever arm of resisting couple to depth, d
k	ratio of depth of neutral axis to depth, d
L	length in feet
l	length in inches
M	moment
n	ratio of modulus of elacticity of steel to the modulus of elacticity of concrete
R	reaction
v	unit shear
w	load per unit of length
z	section modulus
ø	round
u	bond

Abbrevations

M.S.H.D.	Michigan State Highway Department
A.A.S.H.O.	American Association of State Highway Officials
A.I.S.C.	American Institute of Steel Construction

The specifications and maximum unit stresses used are those of the Michigan State Highway Department published in 1944 and from the specification for highway design published by the American Association of State Highway Officals in 1944.

f_c'	3000 psi
f_c	1200 psi
f_s	18000 psi
j	.867
k	.400
n	10
V	60 psi
v	90 psi
u	150 psi
u	300 psi (where special anchorage is provided)

Piles	20 ton per square foot supported
Loading	H 20 S 16-44
Shear	13500 psi power driven rivets
Shear	10000 psi unfinished bolts

Design of Railing

Substantial railings shall be provided along each side of the bridge for the protection of traffic. The top of railing shall not be less than 3'-0" above the top of curb and when on a sidewalk, not less than 3'-0" above the top of the sidewalk. Railings shall contain no opening of greater width than eight (8) inches. Ample provision shall be made for inequality in the rate of movement of the railing and the supporting superstructure, due to temperature or erection conditions.

(M.S.H.D. Spec. 25 p. 10)

Railings shall be designed to resist a horizontal force of not less than 150 pounds per lineal foot, applied at the top of the railing, and a vertical force of not less than 100 pounds per lineal foot. For railings adjacent to the roadway, the bottom rail shall be designed for a horizontal force of 300 pounds per lineal foot of rail.

(M.S.H.D. Spec 35 p. 14)

Railings:

Bolts 3/4" ø in single shear capacity 4420 #

(for 10,000 psi steel)

Load 300 x 8.167/2 = 1225 #

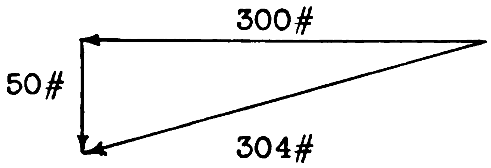
Strap shear capacity

(1 3/4 - 13/16)5/8 x 10000 = 5860 #

Design of Railing

Using 50 pounds per lineal foot as the dead weight of the railing

(M.S.H.D. Standard Design of Railing)



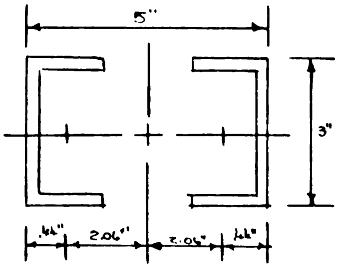
Load: 304 x 8.167/2 = 1240#

Bolts control as they have minimum capacity

1 (max) 1 7/8" + 3/8" = $2\frac{1}{4}$"

f (horizontal)	$\frac{1225 \times 2.25 \times 6}{.625 \times (1.75)}$	8650 psi
f (vertical)	$\frac{1225 \times 2.25 \times 6}{6 \times 1.75 \times (6.25)}$	4050 psi
f (total)		12700 psi
f (allowable)		18000 psi

Design of Railing



Rail: Lower rail only considered (max. case) For the first computation only the two side channels are considered to simplify the computations.

Horizontal bending $\quad 8.167 - 2 \times 1\ 5/8 = 7.90'$

$$M = wl^2/8 = \frac{300 \times (7.90)^2 \times 12}{8} = 28100 \text{ in. lbs.}$$

$A = 2 \times 1.46 = 2.92$ sq. in. $\quad e = 2.06 \quad Ig = 2 \times .25 = .50''^3$

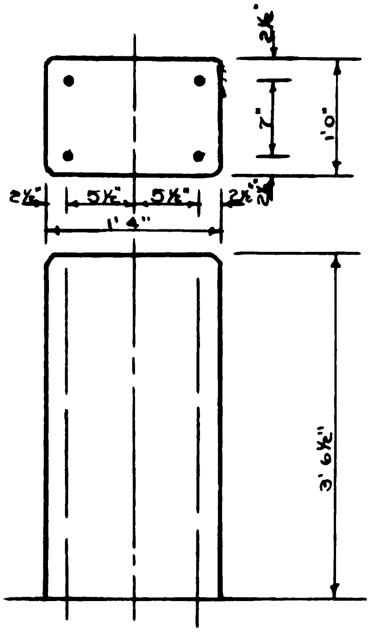
$I = Ae \; + Ig = 2.92 \times (2.06) + .50 = 12.89$ in.

$$f = Mc/I = \frac{28100 \times 2.5}{12.89} = 5450 \text{ psi}$$

18000 psi allowable

Since the stresses are so low we may neglect the top channel of the lower railing in the design. This top channel would increase the strength at the railing which is already 330% over designed.

Posts



RC 3 Intermediate Posts

b 16"

d $9\frac{1}{2}$"

As 2 x .44 0.88 sq. in.

(2 bars in compression)

(2 bars in tension)

$$k = \frac{1}{fs/nfc + 1} = \frac{1}{1800/10 \times 1200 + 1} = .400$$

$$j = (1 - k/3) = (1 - .400/3) = .867$$

Bending moments:

150 x 9.54 x 32	=	45800"#
300 x 9.54 x 5	=	14300"#
Total	=	60100"#

$d = \sqrt{60100/2880 \times .867}$ = 4.92 inches required

9.50 inches actual

fs = M/Asjd = 60100/.88 x .867 x 9.5 = 8300 psi

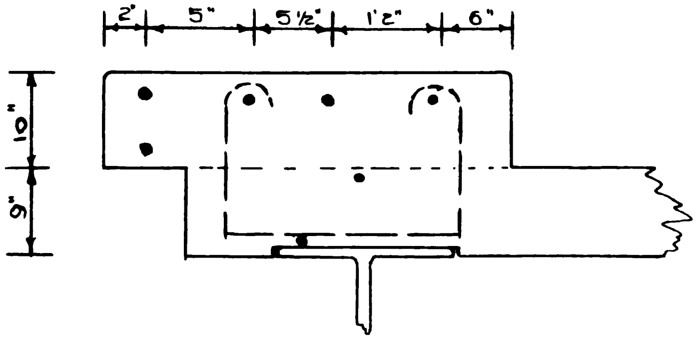
18000 psi all.

Posts

The posts are able to bear a far greater load then that to which they are subjected. The actual stressin the steel of the post is only 46% of the allowable stress.

The railing and posts are greatly over designed, however, a large post of this greater size is obviously to produce a more massive appearance and greater architectural beauty throughout.

Design of Sidewalk and Curb



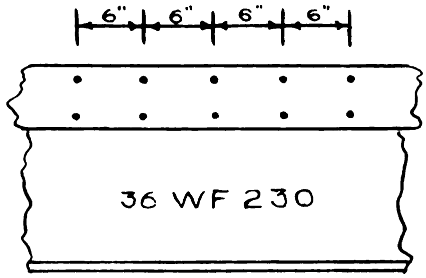
Curbs shall be designed to resist a force of not less than five hundred (500) pounds per lineal foot of curb applied at the top of the curb.

(M.S.H.D. Spec. 36 p. 14)

The specified roadway loading shall apply to all sidewalks constructed without guard rails between the sidewalk and roadway to prevent encroachment of roadway loads.

As this sidewalk is fully supported the use of this part of the bridge serves only as an additional safety measure for pedestrians and as a more or less decorative measure. The only steel actually required is temperature steel with the other being more or less superfulous.

Design of Concrete Slab



Calculate bending moment by A.A.S.H.O. art. 3.2.2 p. 138

Main reinforcing perpendicular to center line of roadway.

Distribution of wheel loads

For span 2-7 ft.

$E = 0.6s + 2.5$

Bending moment for freely supported span

$M = 0.25 \times P/E \times s \times$ (100 + I + 10% for longitudional forces)

Bending moment for continuous span

$M = 0.2 \times P/E \times s \times$ (100 + I + 10% for lonitudional forces)

Where,

E = width of slab over which wheel load is distributed

P = maximum wheel load in pounds

s = distance between flanges plus ½ width of girder flanges

$$I = \frac{L + 20}{6L + 20}$$

Design of Concrete Slab

The forces due to traction or sudden braking of vehicles shall be considered as longitudinal forces having a magnitude of 10% of the gross live load that can be placed in one traffic line. This load shall be assumed as acting in the direction of traffic movement and applied at the top of pavement.

(M.S.H.D. Spec. 38 p. 15)

Design

$E = 0{,}6 \times 5 + 2.5 = 5.5$ ft.

$M = 0.25 \times 16000/5.5 \times 5 \times 1.31 = 4760$ ft. lbs.

$d = \sqrt{M/Rb} = \sqrt{(4760 \times 12)/(208 \times 12)} = 4.78$ in.

For slabs the distance from the surface of the concrete either top or bottom, to the center of the nearest bar shall be not less than one and one-half times the diameter of the bar nor less than one and one-half inches.

Thickness required: $4.78 + 1.50 = 6.28$ in.

Actual: 7.00 in.

$As = M/fsjd = (4760 \times 12)/(18000 \times 7/8 \times 5.5) =$

.663 sq. in. requimed

1.000 sq. in. actual

Steel at bottom of slab for lateral distribution.

Percent of main steel required

$100/\sqrt{s} = 100/\sqrt{5} = 44.8\%$

(A.A.S.H.O. art. 3.2.2 p. 140)

$As = .663 \times .448 = .296$ sq. in./ft. required

Design of Concrete Slab

Slabs designed for bending moment in accordance with the foregoing shall be considered satisfactory in bond and shear.

(A.A.S.H.O. art. 3.2.2 (d) p. 140)

Diaphragm Design

The forces due to wind and lateral vibrations shall consist of a horizontal moving load equal to 30 pounds per square foot on $1\frac{1}{2}$ times the area of the structure as seen in elevation, including the floor system and railing and on one-half the area of all trusses and girders in excess of two in the span.

(M.S.H.D. Spec. 38 p. 15)

(a) Size, Rivets shall be of the size specified but generally shall be 3/4 inch or 7/8 inch in diameter.

(b) Pitch of Rivets, The minimum distance between centers of rivets shall be three times the diameter of the rivet but preferably shall be not less than the following:

For 3/4 inch diameter rivets - $2\frac{1}{2}$ inches

(M.S.H.D. Spec. 92 p. 39 - 40)

Diaphragms shall be provided at the third points of all I-beams spans of forty feet or more.

(M.S.H.D. Spec. 124 p. 48)

(a) Design, Lateral, longitudinal and transverse bracing shall be composed of angles or other shapes and shall have riveted connections.

(M.S.H.D. Spec. 123 p. 48)

The end connections angles of floorbeams and stringers shall be not less than 3/8 inch in finished thickness.

(M.S.H.D. S pec. 120 p. 47)

Diaphragm Design

Area of structure as seen in elevation

7.833 x 59.582   468 square feet

1.5 x 468	702 square feet
.5 x 6 x 3 x 57	512 square feet

Total effective area 1214 square feet

Moving load   30 x 1214   =   36420 pounds

Area required   End diaphragm

36420/18000   =   2.02 square inches required

Area furnished

4" x 4" x 3/8"   furnishes 2.86 square inches

The intermediate diaphragms meet all the necessary specifications provided by the M.S.H.D. for depth of web, size of angles, pitch of rivets, depth of hitchangles, and number of stiffeners.

Girder Design

Main trusses and girders shall be space a sufficient distance apart center to center to be secure against overturning by the assumed lateral and other forces.

(M.S.H.D. Spec. 76 p.36)

For the calculation of stresses, span lengths shall be assumed as follows:

Beams and girders, distance between centers of bearings.

(M.S.H.D. Spec. 77 p.36)

Rolled beams shall be proportioned by the moments of inertia of their net sections.

(M.S.H.D. Spec. 80 p.36)

For structures with concrete slab floors without separate wearing surface, a minimum allowance of 20 pounds per square foot of roadway shall be made, in addition to the weight of any monolithically placed concrete wearing surface, to provide for future wearing surface.

(M.S.H.D. Spec. 30 p.12)

When provision is made for three or more lanes of traffic, the design shall provide for the following percentages of the simultaneous maximum loading of all lanes;

For four or more traffic lanes--------80%

(M.S.H.D. Spec. 33 p.13-14)

Using H-20 s 16-14 loading from appendix A AASHO page 229

Girder Design

Dead Load:

Slab	4.98 x 7.5/12 x 1 x 150	467#/' span
Girder	(assumed 36 WF 230)	230#/' span
Future wearing surface	20 x 4.98	100#/' span
	Total Dead Load	797#/' span

Dead Load Moment

$M = wl^2/8 = (797 \times 12 \times 57^2)/8 = 3885000$ in. lbs.

Live Load Moment

$M = 385400 \times 12 \times .80 = 3600000$

Impact $3600000 \times .21 = 757000$

Total live load moment	4357000 in lbs.
Total moment	8242000 in. lbs.

Z required $8242000/18000 = 456$ in.^{3}

Deck plate girders with compression flanges continuously stayed in a concrete slab may have a depth not less than 1/20 span.

(M.S.H.D. Spec. 79 p. 36)

$(57.08 \times 12)/20 = 34.2$ in.

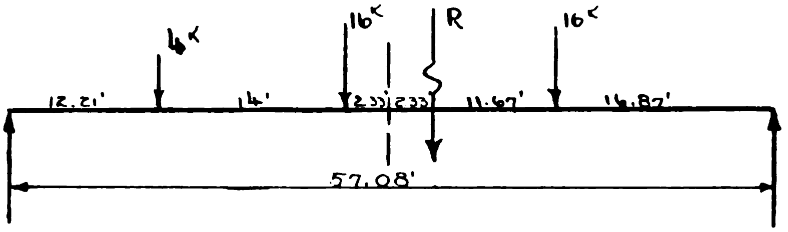
It is therefore necessary to use a 36 inch beam.

Z furnished by 36 WF 230 is 835.5 in.

Girder Design

Check deflection

Beam loaded as max. bending moment.



deflection @ midpoint (x 1/2) = $\frac{Pbx\ (l - b - x\ )}{6EIl}$

$$\frac{16(16.85 \times 12)(28.54 \times 12)(467000-41000-117500)}{6 \times 30 \times 10 \times 14990 \times (57.08 \times 12)} = .186 \text{ in.}$$

$$\frac{16(26.21 \times 12)(28.54 \times 12)(467000-99000-117500)}{6 \times 30 \times 10 \times 14990 \times (57.08 \times 12)} = .235 \text{ in.}$$

$$\frac{4(12.21 \times 12)(28.54 \times 12)(467000-21500-117500)}{6 \times 30 \times 10 \times 14990 \times (57.08 \times 12)} = .038 \text{ in.}$$

Total L.L. Deflection	.459 in.
Impact (21%)	.096 in.
Total Deflection	.555 in.

Live load plus impact deflection 1/1000 span length

Allowable deflection (57.08 x 12)/1000 = .684 inches

A 36 WF 230 beam is the smallest beam which does not exceed the allowable deflection.

Abutment Design

Retaining walls, abutments and structures built to retain fills shall be designed to resist pressures determined in accordance with the "Rankine" theory of pressure distribution in noncohesive granular material, provided that no structure shall be designed for an equivalent fluid pressure of less than 30 pounds per square foot.

Loading

Dead load (superstructure)

Total concrete	104/2 x 27 x 150	210500 pounds
Total steel	144100 x ½	72050 pounds
	Total	282550 pounds

Out to out of wings 86.875

Then 282550 x 1/86.875 = 3250 #/'

Live load

One lane reaction 60400 #

Total live load 60400 x 4 x .8 = 193000 pounds

Then 193000 x 1/86.875 = 2220 #/'

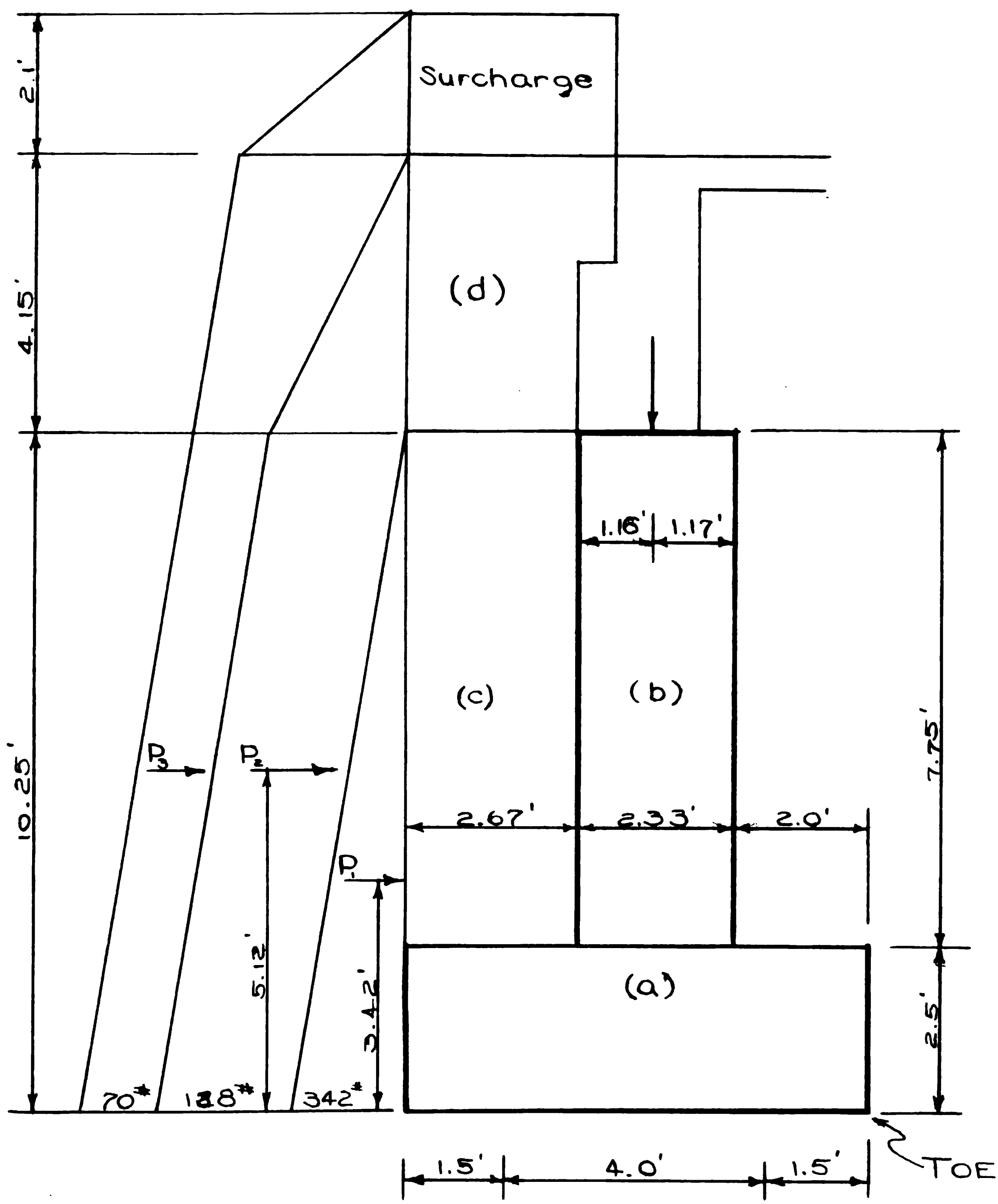
Surcharge

Face to face of rail 45' 0"

Then (45 x 400)/86.875 = 210 #/'

Trial section:

Toe projection	2' 0"
Wall	2' 4"
Heel projection	2' 8"



Surcharge
2.1'
4.15'
10.25'
(d)
(c)
(b)
(a)
1.16'
1.17'
7.75'
2.5'
P_3
P_2
P_1
2.67'
2.33'
2.0'
5.12'
3.42'
70#
128#
342#
1.5'
4.0'
1.5'
TOE

Abutment Design

Overturning		Thrust				Moment
P)	342 x 10.25/2 =	1750 #	x	3.42	=	6000'#
P)	138 x 10.25 =	1415 #	x	5.12	=	7240'#
		M(OI) = M(OIII)			=	13420'#
P)	70 x 10.25 =	717 #	x	5.12	=	3670'#
				M(OII)	=	16910'#

Stability			Weight				Moment
a)	7 x 2.5 x 150		2620 #	x	3.50	=	9180'#
b)	7.75 x 2.33 x 150		2710 #	x	3.17	=	8600'#
c)	7.75 x 2.67 x 100		2070 #	x	5.66	=	11720'#
d)	2.67 x 4.15 x 100		1110 #	x	5.66	=	6270'#
		W(I) =	8510 #		M(SI)	=	35770 '#
	Dead Load		3250 #	x	3.17	=	10600'#
		W(IV) =	11760 #	x	M(SIV)	=	46370'#
	Surcharge 2.67 x 2.1	=	561 #	x	5.66	=	3170'#
		W(II) =	12321 #		M(II)	=	49540'#
		W(IV) =	11760 #		M(SIV)	=	46370'#
	Live Load		2220 #	x	3.17	=	7040'#
		W(III) =	13980 #		M(SIII)	=	53410'#

Abutment Design

Case I

No load or live load surchage

M(SI)	35770'#				
M(OI)	<u>13240'#</u>				
M(I)	22530'#	R(I)	=	22530/8510 = 2.65'	OK

Case II

Superstructure dead load and live load surcharge

M(SII)	49540'#				
M(OII)	<u>16910'#</u>				
M(II)	32630'#	R(II)	=	32630/12321 = 2.65'	OK

Case III

Superstructure dead load and live load, no surcharge

M(SIII)	53410'#				
M(OIII)	<u>13240'#</u>				
M(III)	40170'#	R(III)	=	40170/13980 = 2.87'	OK

Case IV

Superstructure dead load, no live load or surcharge

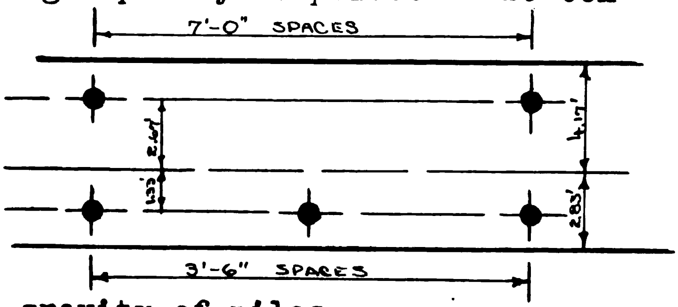
R(IV) = 46370/11760 = 3.95 OK

Note: Case IV assumes that theearth exests no horizontal force against the abutment wall.

Abutment Design

Maximum abutment load WIII 13980 #

Bearing capacity of piles: 20 ton 40,000 #



Center of gravity of piles

Back	5.5 x 1 =	5.5
Front	1.5 x 2 =	3.0
		8.5

8.5/3 = 2.83 feet from face of footing

I = Ad

Back	1 x 2.67 =	7.12
Front	2 x 1.33 =	3.56
	I =	10.68"

Load on pile P/A ± Mc/I

Case II $\frac{12321 \times 7}{3} + \frac{12321 \times 7 \times .18 \times 1.33}{10.68}$

28750 + 1935 = 30685 # OK Front row

$28750 - \frac{12321 \times 7 \times .18 \times 2.67}{10.68}$

28750 − 3880 = 24870 # OK Back row

Abutment Design

Case III $\frac{13980 \times 7}{3} \pm \frac{13980 \times 7 \times .06 \times 1.33}{10.68}$

32500 − 730 = 31770 # OK Front row

32500 + 1470 = 33970 # OK Back row

Case IV $\frac{11760 \times 7}{3} \pm \frac{11760 \times 7 \times 1.12 \times 1.33}{10.68}$

27400 − 11500 = 15900 # OK Front row

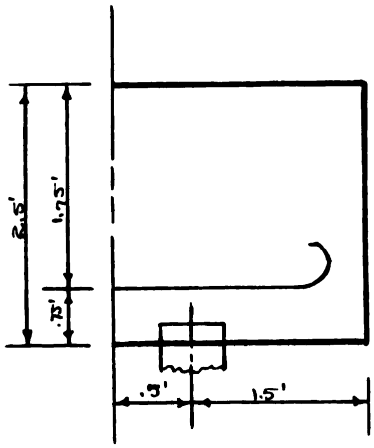
27400 + 23000 = 50400 # Overstressed 26%

Note: Case IV, back row is overstressed by 26% but this does not consider the horizontal force exerted by the earth. (M.S.H.D. allows 33% overstress in this case.)

Abutment Design

Reinforcement

Toe: Case II



Pile load 30685 #

Pile spacing 3.5'

Load/ft. of wall 30685/3.5

8750 #

Less wt. conc. 750 #

Shear 8000 #

Moment 8750 x .5 -750 x 1 =

3625 '#

$$A_s = M/f_s jd = \frac{3625 \times 12}{18000 \times 7/8 \times 21} = .132 \text{ sq. in.}$$

use 3/4" ∅ bars@ 1'-0" centers

A_s = .44 sq. in. Eo = 2.4 in.

p = .44/12 x 21 = .00175

j = .944 k = .169

$$f_s = \frac{3625 \times 12}{.44 \times .944 \times 21} = 4440 \text{ psi}$$

$$f_c = f_s k/n(1-k) = \frac{4440 \times .169}{8.31} = 91 \text{ psi}$$

$$v = V/bjd = \frac{8000}{12 \times .944 \times 21} = 33 \text{ psi}$$

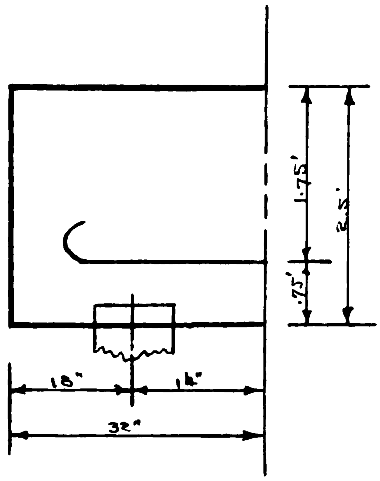
$$u = V/E_o jd = \frac{8000}{2.4 \times .944 \times 21} = 165 \text{ psi}$$

300 psi

(allowable)

Abutment Design

Heel: Case IV (bottom steel)



Pile load 50400 #

Pile spacing 7.0'

Load/ft. of wall 50400/7 7200#

Less: ftg.(2.5 x 2.67 x 150)

	1000 #	
(c)	2070 #	
(d)	1110 #	
		4180#
Shear		3020#

Moment = 7200 x 14 - 4180x16 = 34000 "#

Use 3/4" Ø bars @ 2'-0" centers from every other toe bar

As = .22 sq. in. Eo = 1.2" np = .0088

$k = \sqrt{2pn + (pn)^2} - pn = .124$ $j = .959$

$$fs = \frac{34000}{.22 \times .959 \times 21} = 7680 \text{ psi} \quad \text{OK}$$

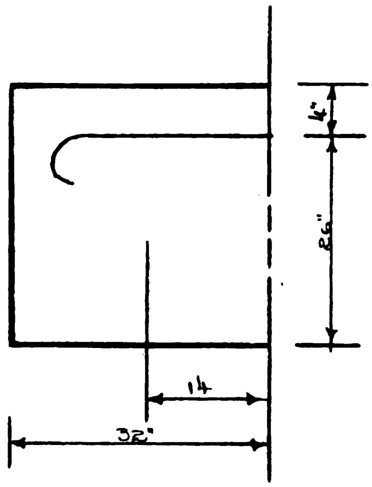
$$fc = \frac{7680 \times .124}{8.76} = 110 \text{ psi} \quad \text{OK}$$

$$v = \frac{3020}{12 \times .959 \times 21} = 12.5 \text{ psi} \quad \text{OK}$$

$$u = \frac{3020}{1.2 \times .959 \times 21} = 125 \text{ psi} \quad \text{OK}$$

Abutment Design

Heel: Case II (top steel)



Pile load 24870 #

Pile spacing 7.0'

Load/ft. of wall 24870/7 = 3550#

Surcharge	561 #
(c)	2070 #
(d)	1110 #
Footing	1000 #
	4741 #
	3550 #
Shear	1191 #

Moment 4741 x 16 - 3550 x 14 = 26200"#

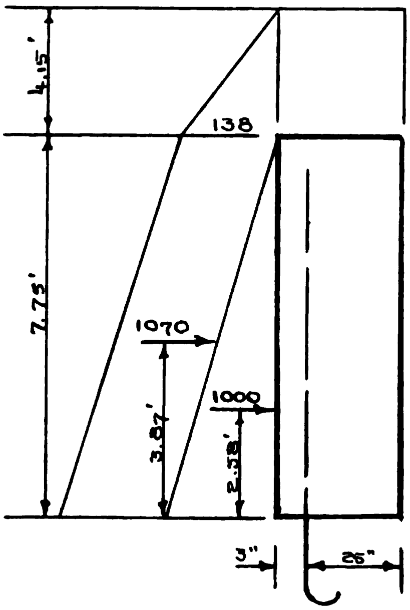
$$As = \frac{26200}{18000 \times 7/8 \times 26} = .06\ ''$$

Use 3/4" Ø @ 2'-0" centers

All unit stresses are OK by inspection.

Abutment Design

Wall:



	1070 x 3.87		4140'#
	1000 x 2.58		2580'#
Shear	2070 #	Moment	6720'#

$$As = \frac{6720 \times 12}{18000 \times 7/8 \times 25} = .205$$

Try 3/4" Ø @ 2'-0" centerss

As = .22 sq. in. Eo = 1.2 in.

j = .857 k = .429

$$p = \frac{.22}{12 \times 25} = .000733$$

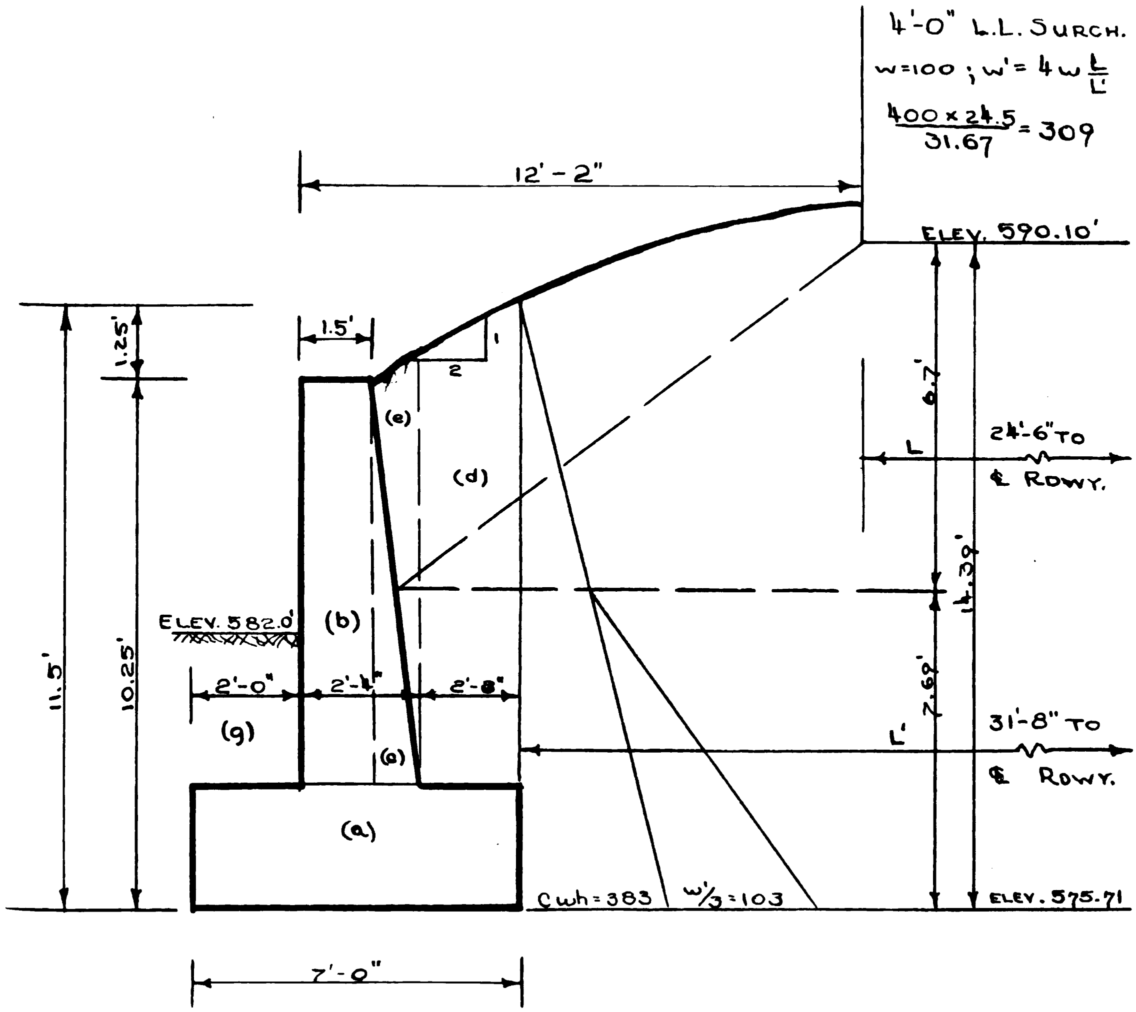
$$fs = \frac{6720 \times 12}{.22 \times .857 \times 25} = 17100 \text{ psi} \quad \text{OK}$$

$$fc = \frac{17100 \times .429}{5.71} = 1280 \text{ psi} \quad \text{OK}$$

$$v = \frac{2070}{12 \times .857 \times 25} \doteq 8.07 \text{ psi} \quad \text{OK}$$

$$u = \frac{2070}{1.2 \times .857 \times 25} = 80.7 \text{ psi} \quad \text{OK}$$

Use 3/4" Ø @ 2'-0" centers


4'-0" L.L. SURCH.
w=100 ; w' = 4w L/L'
400 x 24.5 / 31.67 = 309
12' - 2"
ELEV. 590.10'
1.5'
1
2
1.25'
11.5'
10.25'
(e)
(d)
(b)
(g)
(a)
ELEV. 582.0'
2'-0"
2'-4"
2'-8"
6.7'
14.39'
7.69'
L
24'-6" TO ℄ ROWY.
L'
31'-8" TO ℄ ROWY.
Cwh=383
w'/3=103
ELEV. 575.71
7'-0"

Abutment Design

Wing Wall

Overturning	Thrust	Moment
383 x 11.5 x ½	2200 # x 3.83 =	8420'#
	M(OI) =	8420'#
103 x 7.69 x ½	396 # x 2.56 =	1013'#
	M(OII) =	9432'#

	Stabilizing	Weight	Moment
a)	2.5 x 7 x 150	2620 # x 3.50 =	9200'#
b)	1.5 x 7.75 x 150	1745 # x 2.75 =	4800'#
c)	7.75 x .83 x 150 x½	482 # x 3.78 =	1825'#
d)	7.75 x 2.67 x 100	2070 # x 5.67 =	11750'#
e)	7.75 x .83 x 100 x ½	321 # x 4.05 =	1300'#
f)	1.25 x 2.67 x 100 x ½	167 # x 6.11 =	1020'#
g)	3.79 x 2 x 100	758 # x 1.00 =	758'#
		W = 8163 # M(SI'II') =	30653'#

Abutment Design

Wing Wall Stability:

Case I'

No live load surcharge

M(SI') 30653'#

M(OI') 8420'#

M(I') 22233'# $R(I') = \frac{22233}{8163} = 2.72'$ OK

Case II'

With live load surcharge

M(SII') 30653'#

M(OII') 9432'#

M(II') 21221'# $R(II') = \frac{21221}{8163} = 2.58'$ OK

Case IV'

No earth thrust

$R(IV') = \frac{30653}{8163} = 3.75'$ OK

Wing Wall Reinforcing Steel:

By a comparison of main wall and wing wall loads it is seen that the main wall is designed for a more severe condition of loading than would come on the wing. Further, it can also be seen that the same steel is used in the wall of the wing as that used in the wall of the main wall; therefore, the stresses in the wing wall are OK by inspection.

Conclusion

As the thesis progressed remarks were inserted in their related sections; however, it may be well to call attention to several items which are of standard design. Such items as the railing, posts, curb, concrete slab, and diaphragms are of Michigan State Highway Department standard; therefore, it will be noted that because they are designed for the most severe conditions they are overdesigned for this structure. The Highway Department through years of experience has found that the labor saving in design work far overshadows that saving in material if each item is to be designed separately.

As an overall conclusion it may be said that the entire structure is adequately designed and the above named sections are well overdesigned.

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