AN ANALYSIS OF THE DESIGN OF A SINGLE SPAN HIGHWAY BRIDGE

Thesis for the Degree of B. S.

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Donald M. Tubbs

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An Analysis of the Design of a Single Span Highway Bridge

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by

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Candidate for the Degree of

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Contents

Introduction

This thesis was chosen by the author so that it would be possible to learn something about bridge design and design in general. It is also the object to give the author a little knowledge in methods used in design and to give some practice in reading blueprints.

The bridge used in this thesis was built in 1947 in Lenawee County, located on M52(called Adrian road), five tenths of a mile south of the village of Jasper. This bridge was built to replace the old out dated bridge, that crossed Black Creek.

The creek was rerouted so that it would cross the road at right angles, where as before the intersection was at an angle other than ninety degrees. By this change in route, the placing of this bridge was accomplished without the use of piles for a supporting foundation.

LIST OF ABBREVIATIONS AND SYMBOLS

Abbreviations

AASHO	American Association of State Highway Officals.
AISC	American Institute of Steel Construction.
MSHD	Michigan State Highway Department.
D.L.	dead load.
k	kips.
L.L.	live load.
N.A.	neutral axis.
WF	wide flange beam section.

Symbols

a	distance, length, or thickness.
A	Area.
A_s	area of tensile reinforcement.
b	breadth or distance.
c	distance such as that to the extreme fiber.
d	effective depth of flexural members.
e	eccentricity.
E	modulus of elasticity.
f	fiber stress.
f_c	compressive stress (allow.)
f'_c	ultimate compressive strength of concrete.
f_s	stress in tensil reinforcement.
I	moment of inertia.
j	ratio of distance (jd) between resultants of compressive and tensile strees to effective depth.

LIST OF ABBREVIATIONS AND SYMBOLS

Symbols - continued

l,L	lengths.
M	bending moment
n	ratio of modulus of elasticity of steel (E_s) to that of concrete (E_c).
Q	section modulus.
t	thickness.
u	bond stress.
v	shearing stress.
V	total shear.
w	uniform load per unit length or area
W	total load.
$\bar{y}$	distance to the center of gravity.
Δ	deflection.
Σ	summation.
Σo	sum of perimeters of bars.

Railings

Specifications:

M.S.H.D. Spec. 25;

Substantial railings shall be provided along each side of the bridge for the protection of traffic. The top of the railing shll not be less than 3'-0" above the top of the curb and when as a sidewalk, not less than 3'-0" above the top of the sidewalk. Railings shall contain no openings of greater width than eight (8) inches. Ample provision shall be made 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. 35;

Railing 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.

Design:

Bolts 3/4" in single shear (for 10,000psi steel)

Area- .302 .302x 10,000 = 3,020 lbs.

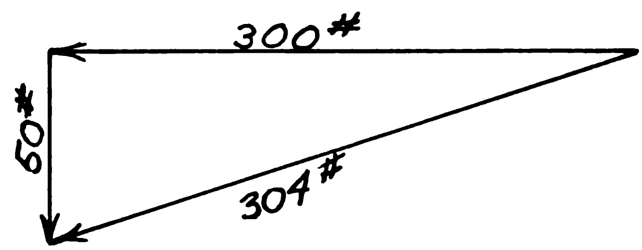
Load- 300 x 8.208/2 = 1230 lbs.

Strap shear capacity,

(13/4 - 13/16) 5/8 x 10,000 = 5860 lbs.

Railings (cont.)

Using 50 pounds per lineal foot as the dead weight of the railing, as specified in M.S.H.D. Standard Design of Railings.



Load; 304 x 8.208/2 = 1247 lbs.

Bolts control as they have minimum capacity.

l(max.) 1-7/8" + 3/8" = 2-1/4"

f(horzontal) $\dfrac{1230 \times 2.25 \times 6}{.625 \times (1.75)}$ = 8650 psi

f(vertical) $\dfrac{1230 \times 2.25 \times 6}{6 \times 1.75 \times (6.25)}$ = 4050 psi

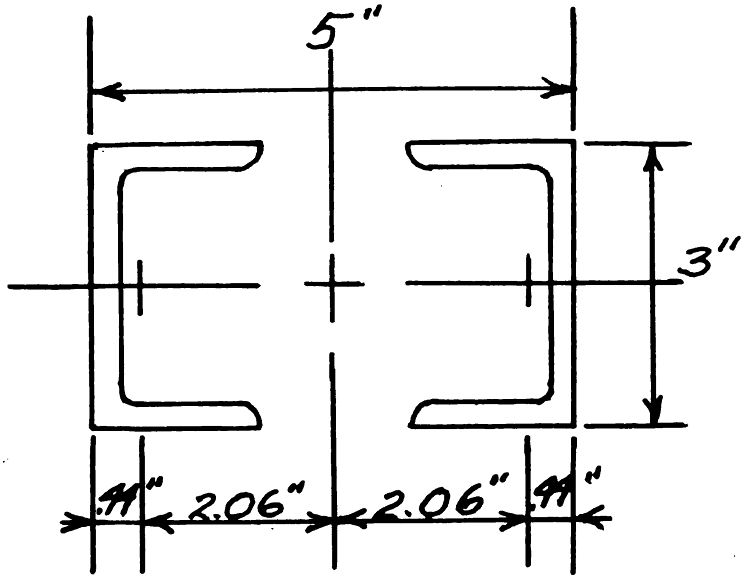
f(total) 12700 psi

f(allowable) 18000 psi

Rail: (max. case)

The lower rail will only be considered. For the computations only the two side channels are to be considered to simplify the computations.

Railings (cont.)



Span: $8.208 - 2 \times 1\text{-}5/8" = 7.94$ ft.

Moment(M): $M = 1/8\ wl^2$

$$M = \frac{1}{8} \times 300\ (7.94 \times 12)^2$$

$M =$, 34,000 in.-lbs.

$I = I_o + Ad^2$ $\quad d = 2.06$ in. $\quad I_o = 2 \times .25 = .50\ in^4$

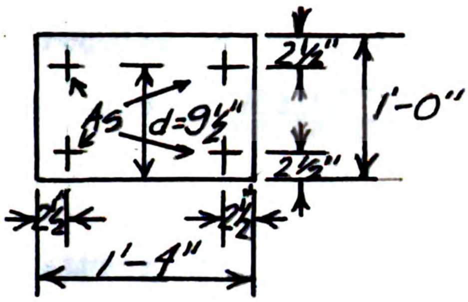
$A = 2 \times 1.46 = 2.92$ sq. in.

$I = .50 + 2.92\ (2.06)^2 = 12.89\ in^4$

$$F_s = \frac{Mc}{I} = \frac{34{,}000 \times 2.5}{12.89} = 7{,}070 \text{ psi}$$ (18,000 allow.)

The railings are very much overdesigned but are designed more for the looks than for any given load.

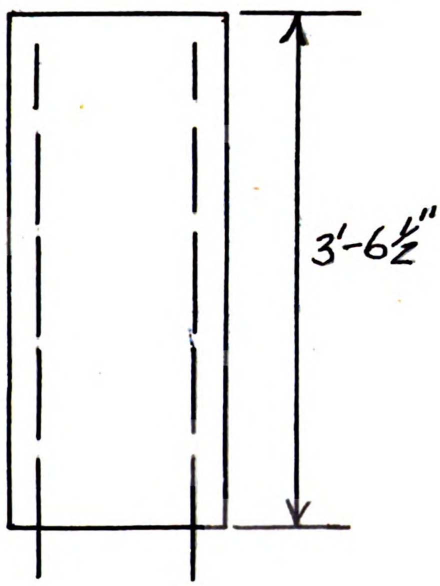
Posts



Intermediate Posts

b = 16 in.

d = 91/2 in.



A_s = 2 x .44 = 0.88 sq. in.

2 bars in compression

2 bars in tension

$$K = \frac{1}{\frac{f_s}{n f_c} + 1} = \frac{1}{\frac{18000}{10 \times 1200} + 1} = 0.40$$

$$j = (1 - K/3) = 1 - \frac{.40}{3} = .867$$

Bending moments;

150 x 9.54 x 32 = 45,800 in. lbs.

300 x 9.54 x 5 = 14,300 in. lbs.

total 60,100 in. lbs.

Posts (cont.)

requird (d)

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

9.50 inches furnished

stress (f_s)

$$f_s = \frac{M}{A_s\, jd} = \frac{60100}{.88 \times .867 \times 9.5} = 8300 \text{ psi}$$

(18000 psi allowable)

The end posts will not be checked as they are of greater size, with one-half more reinfercement. They also do not carry as much load.

The posta are also very much overdesigned and there for can carry a much greater load than used in design. The posts are design as are the railings to produce a massive appearance and greater architectural beauty.

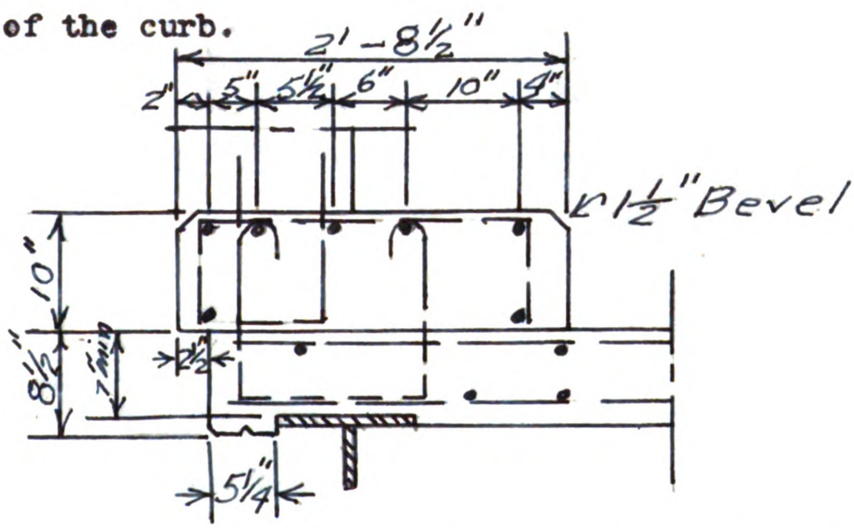
Sidewalks and Curbs

M.S.H.D. Spec. 20;

Substantial curbs shall be built on each side of the roadway and they shall have a width of not less than six(6) inches and a height of not less than nine(9) inches measured above the wearing surface at a point adjacent to the curb.

M.S.H.D. Spec. 36;

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.



The sidewalks are of much greater designed than specified by the M.S.H.D.. About the only way that they could fail would be by the crushing of the concrete. Therefore the only reinforcement needed in the sidewalks is that steel which is needed for temperature reinforcement. Also some anchorage which is used to hold the sidewalks in place.

Concrete Floor Slab

Specifications:

M.S.H.D. Spec. 37;

Calculate bending moment by A.A.S.H.O. art. 3.2.2 p. 138. Main reiforcing perpendicular to center line of roadway.

Distribution of wheel leads;

(for spans 2-7 ft.)

$E = 0.6s + 2.5$ ft.

Bending moment(M) for freely supported spans;

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

Bending moment(M) for continuous spans;

$M = 0.20 \times \frac{P}{E} \times s \times$ (100% + I + 10% for longitudional forces)

Where;

E = width of slab over which wheel load is distributed

P = maximum wheel load in pounds

s = distance between flanges plus one- half width of girder flanges

$I = \frac{L + 20}{6L + 20}$ in which L = span length

M.S.H.D. Spec. 38;

The forces due to traction or sudden braking of vehicles shall be consider as longitudinal forces having a magnitude of 10% of the gross live load that can be placed in one traffic lane. This load shall be assumed as acting in the

Concrete slab (cont.)

direction of traffic movement and applied at the top of the pavement.

M.S.H.D. Spec. 59;

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

Design:

$$E = 0.6s + 2.5 \qquad s = 5'\text{-}10\,3/4" = 5.146 \text{ ft.}$$

$$E = 0.6(5.146) + 2.5 = 3.1 + 2.5 = 5.6 \text{ ft.}$$

$$I = \frac{L + 20}{6L + 20} = \frac{47.583 + 20}{285.498 + 20} = \frac{67.583}{305.498} = .22 \text{ or } 22\%$$

$$M = 0.25 \times \frac{P}{E} \times s \times (100\% + I + 10\% \text{ for longitudional forces})$$

$$M = 0.25 \times \frac{16000}{5.6} \times 5.146 \times (1 + .22 + .1)$$

$$M = 4{,}940 \text{ ft.-lbs.}$$

required (d)

$$d = \sqrt{\frac{M}{Rb}}$$

where; M = moment in in.-lbs.

R = 208

b = width in inches (assume 1 ft. width(12"))

$$d = \sqrt{\frac{4940 \times 12}{208 \times 12}} = 4.88 \text{ inches}$$

Total thickness = d + 1-1/2" for cover

$$t = 4.88 + 1.5 = 6.38 \text{ inches}$$

Concrete slab (cont.)

thickness required by design is 6.33 inches

actual thickness furnished is 7.00 inches mim.

Amount of tension steel required laterally;

$A_s = \frac{M}{f_s\, jd}$ where;

M = moment(max,) in in.-lbs.

f_s = allowable steel stress (18000 psi)

j = 7/8

d = effective depth

$A_s = \frac{4940 \times 12}{18000 \times 7/8 \times 5.5}$

A_s = .685 sq. in./ft. required

A_s = .694 sq. in./ft. furnished

Steål at bottom for lateral distribution;

Percent of main steel required

$100/\sqrt{S}$ A.A.S.H.O. Art. 3.2.2 p. 140

$100/\sqrt{5.146} = 100/2.268 = 44.1\%$ of main steel

A_s = .685 x .441 = .320 sq. in./ft. required

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)

Stringers

M.S.H.D. Spec. 76;

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

M.S.H.D. Spec. 77;

For the calculation of stresses, span length shall be assumed as follows;

Beams and girders, distance between centers of bearing.

M.S.H.D. Spec. 80;

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

M.S.H.D. Spec. 30;

For structures with concrete slab floors, with out separate wearing surface, a minimum allowance of twenty(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. 43;

Longitudienal beams-stringers

The bending moment carried by each interior beams or stringers shall be taken not less than that determined by the following formula:

Stringers (cont.)

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

M = bending moment for one traffic lane

$N = \dfrac{\text{width of one traffic lane (not to exceed 10')}}{\text{spacing of stringers or beams}}$

C = coefficient based on type of floor (equals 1.0 for reinforced slabs)

M' = bending moment on one beam or stringer

M.S.H.D. Spec

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

Design:

Dead load;

Slab	5.146 x 7/12 x 1 x 150	= 450 lbs/ft.
Stringers	(using 33"WF103#)	= 130 lbs/ft.
Future wearing surface 20 x 5.146		= 103 lbs/ft.
Total dead load per foot of span		= 683 lbs/ft.

Dead load moment

$$M = \frac{wL^2}{8} \qquad w = 683\#/\text{ft.} \qquad L = 47.583 \text{ ft.}$$

$$M = \frac{12 \quad 683 \times (47.583)^2}{8}$$

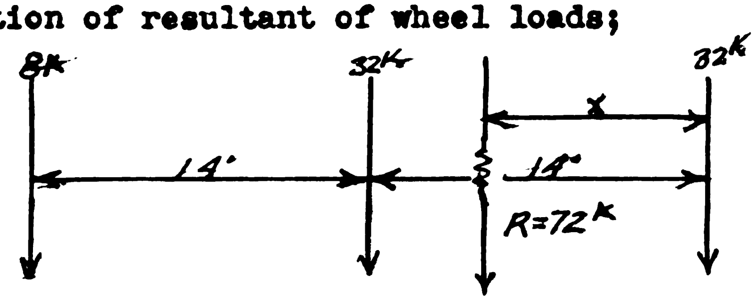
M = 2, 315,000 in.-lbs.

Stringers (cont.)

Live load;

Assuming one traffic lane as a width of unity. The lane will be assumed as a beam with a span of 47.583 ft. The maximum moment will then occur under the load which c causes maximum center moment when the loads are so placed that the center ofnthe beam lies midway between the load and the resultant of all the loads on the span.

Location of resultant of wheel loads;



Σ Mc

$\Sigma R(x) = b \times 14' + a \times 28'$

$72(x) = 32 \times 14' + 8 \times 28'$

$x = \dfrac{448 + 204}{72}$

$x = 9.333$ ft.

Live load moment;

End reactiond

$\Sigma MR_2 = 0$

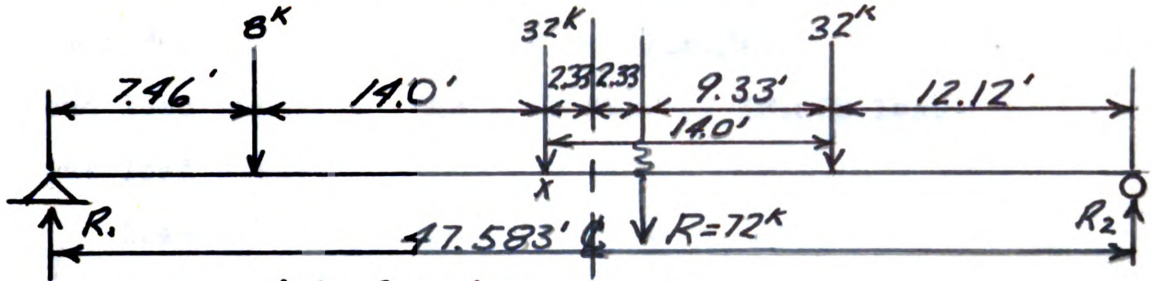
$R1 \times 47.583 - 72000 \times (9.33 + 12.12)$

$R1 = \dfrac{72000 \times 21.45}{47.583}$

$R1 = 32{,}450$ lbs.

Stringers (cont.)

Beam loading for maximum moment;



x = point of maximum moment

bending moment(M) for one traffic lane

$M = 32{,}450 \times (14 + 7.46) - 8{,}000 \times 14$

$M = 696{,}000 - 112{,}000$

$M = 584{,}000$ ft.-lbs.

$M = 584{,}000 \times 12 = 7{,}008{,}000$ in.-lbs.

bending moment for each stringer

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

$M' = \frac{7{,}008{,}000 \times 5.146}{10}$

$M' = 3{,}600{,}000$ in.-lbs.

Total moment equals deadload moment plus live load moment.

$M = 3{,}600{,}000 + 2{,}315{,}000 = 5{,}915{,}000$ in.-lbs.

Section modulus required;

$Q = \frac{M}{s} = \frac{5{,}915{,}000}{18{,}000} = 329$ in.^{3}

Section modulus by 33" WF 130# beam = 408.8 in.^{3}

Stringers (cont.)

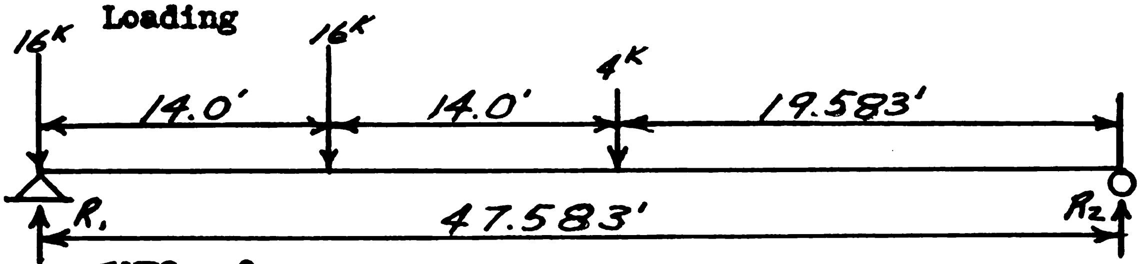
End shear;

Shear due to a dead load of 683 lbs./ft.

Dead load shear = 683 x 47.583 = 16,200 lbs.

Live load shear;

Loading



$\Sigma M_{R2} = 0$

$-R_1 \times 47.583 + 16^K \times 47.583 + 16^K \times 33.583 + 4^K \times 19.583 = 0$

$$R_1 = \frac{16^K \times 47.583 + 16^K \times 33.583 + 4^K \times 19.583}{47.583}$$

R_1 = 28.9 kips

Live load shear = R_1 + Impact

I = .22(28.9) = 6.36 kips

Total live load shear

= 28,000 + 6,360 = 35,260 lbs.

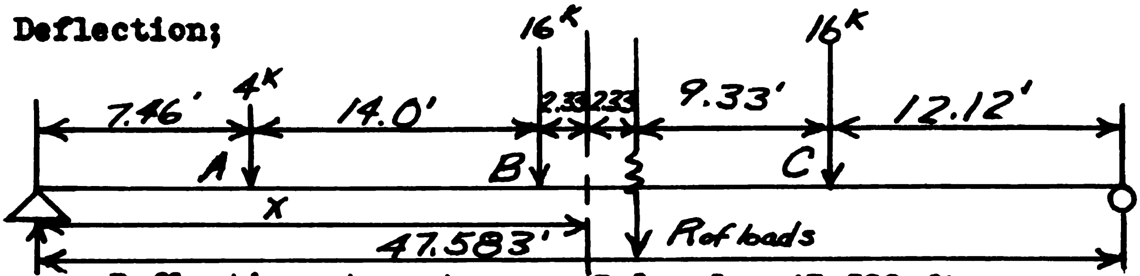
Total shear;

S = deadload shear + live load shear + I

S = 16,200 + 35,260 = 51,460 lbs.

Stringers (cont.)

Deflection;

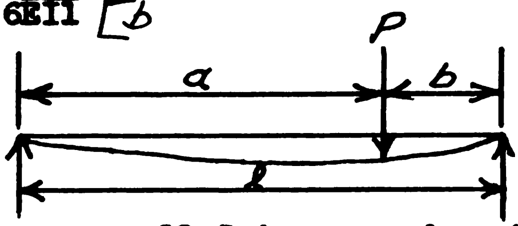


Deflection at center $x = \frac{l}{2}$ $l = 47.583$ ft.

By taking each load and finding the deflection for it and adding them all together the total live load defflection can be found.

If point (x) is to the left of the load considered we use; $\Delta = \frac{Pbx}{6EIl}(l^2 - x^2 - b^2)$

If point (x) is to the right of the load considered we use; $\Delta = \frac{Pb}{6EIl}\left[\frac{l}{b}(x-a) + (l^2-b^2)x - x^3\right]$



Load A; $a = 89.5$ in. $b = 481.5$ in.

$l = 571.0$ in. $x = 285.0$ in.

$$\Delta_A = \frac{4{,}000 \times 481.5}{6 \times 30 \times 10^6 \times 6699 \times 571}\left[\frac{571}{481.5}(286-89.5)^3 + (571^2 - 481.5^2)285 - (286)^3\right]$$

$\Delta_A = .0346$ inches

Load B; $a = 258.0$ in. $b = 313.0$ in.

$l = 571.0$ in. $x = 285.0$ in.

$$\Delta_B = \frac{16{,}000 \times 313}{6 \times 30 \times 10^6 \times 6699 \times 571}\left[\frac{571}{313}(285-258)^3 + (571^2 - 313^2)285 - (285)^3\right]$$

$\Delta_B = .304$ inches

Stringers (cont.)

Load C; b = 145.5 in. l = 571.0 in.

x = 285.0 in.

$$\Delta_c = \frac{16,000 \times 145.5 \times 285}{6 \times 30 \times 10^6 \times 6699 \times 571}\left((571^2)-(285)^2-(145.5)^2\right)$$

Δ_c = .216 inches

Total = $\Delta_A + \Delta_B + \Delta_C$ = .0346 + .304 + .216

Δ = .5546 inches

plus 22% for Impact

Δ = .5546 + (.5546 x .22) = .5546 + .122 = .6766 inches

Allowable deflection equals $\frac{1}{800}$ of the span as given as specifications on plans.

$\frac{1}{800}$ x 571 in. = $\frac{571}{800}$ = .714 inches

A 33" WF 130# beam is the smallest beam tha will give a deflection that will not exceed the specifications.

Diaphragms

Specifications:

M.S.H.D. Spec. 38;

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

M.S.H.D. Spec. 92;

(a) Size of rivets; 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 betweencenters of rivets shall be three times the diameter of the rivets but preferably shall be not less than the following:

For 3/4 inch diameter rivets -- 2-1/2 inches

M.S.H.D. Spec. 124;

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

M.S.H.D. Spec. 123;

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

Diaphragms (cont.)

M.S.H.D. Spec. 126;

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

Design:

Area of structure as seen in elevation;

7.916 x 50	=	395 sq. ft.
1.5 x 395	=	594 sq. ft.
.5 x 7 x 2.75 x 48.4	=	466 sq. ft.
total effective area		1060 sq. ft.

Moving load;

30 lbs. x 1060 sq. ft. = 31,800 lbs.

Area required (end diaphram);

$$\frac{31,800}{18,000} = 1.77 \text{ sq. inches.}$$

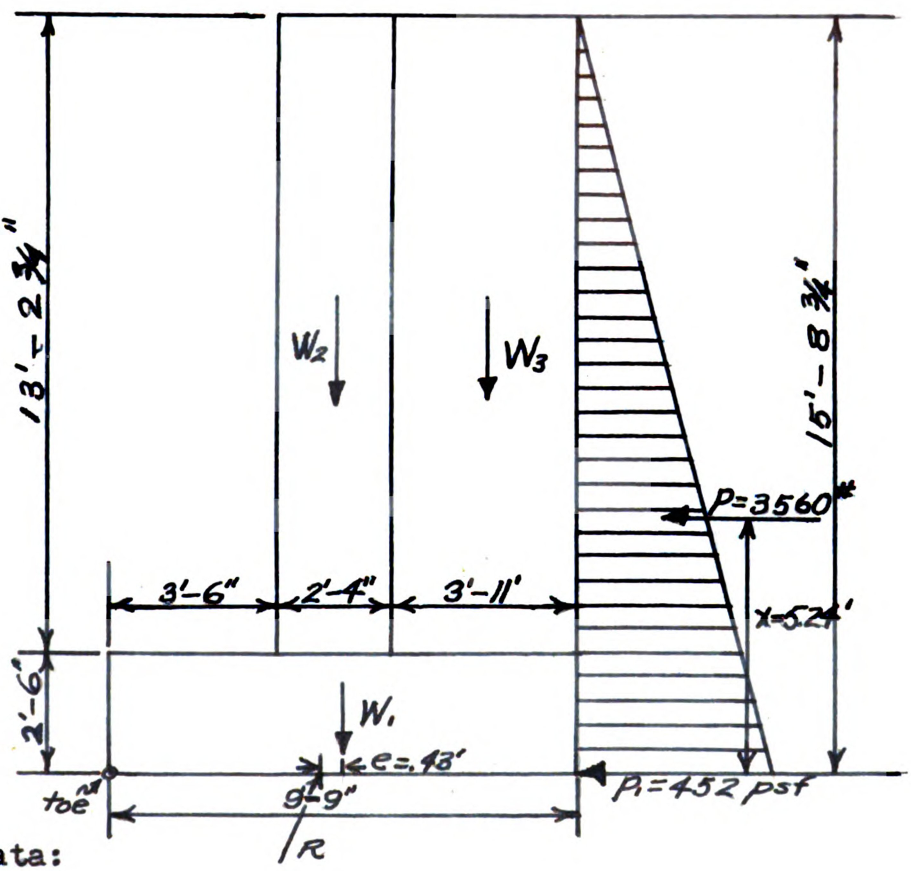
Area furnished;

Area of 2 ∠s 3" x 3" x 3/8" = 4.22 sq. inches

The intermediate diaphragms will not be checked as they will meet all of the nessary specifications provided by the M.S.H.D. for depth of web, size of angles, pitch of rivets, depth of hitch angles, and number of stiffeners, because the design is the same and the load will be less.

Abutments

Case I -- No superstructure load or surcharge;



Data:

Earth w_e = 100 lbs per cu. ft.

$\emptyset$ = 33° 40' = $\tan^{-1}$ 2/3 (earth on earth)

δ = 22° 00' = (earth on concrete)

Foundation pressure

p = 4,100 psf(max)

wt. of concrete w_c = 150 lbs. per cu. ft.

f_c' = 3000psi $\quad$ f_c = 1350psi

v_c = 40psi $\quad$ R = 248

f_s = 18000psi

Abutments (cont.)

$$p = wh \times \frac{1 - \sin \emptyset}{1 + \sin \emptyset} = 100h \times \frac{1 - 0.554}{1 + 0.554} = 28.7\ h$$

$$p_1 = 28.7 \times 15.73 = 452 \text{ psf}$$

$$P = \left(\frac{452}{2}\right) \times 15.73 = 3560 \text{ lbs.}$$

$$x = \frac{15.73}{2} = 5.24 \text{ ft.}$$

$$M_{OT} = 3560 \times 5.24 = 18680 \text{ ft.-bls.}$$

Base pressures; moments about toe;

$$W_1 = 2.5 \times 9.75 \times 150 = 3660 \text{ lbs;} \quad \times \frac{9.75}{2} = 17820 \text{ ft.-lbs.}$$

$$W_2 = 2.33 \times 13.23 \times 150 = 4630 \text{ lbs;} \quad \times \left(3.5 - \frac{2.33}{2}\right) = 20350 \text{ ft.-lbs.}$$

$$W_3 = 3.917 \times 13.23 \times 100 = \underline{5170 \text{ lbs}}; \quad \times \left(5.834 - \frac{3.917}{2}\right) = \underline{40300 \text{ ft.-lbs.}}$$

$$\Sigma W = 13460 \text{ lbs.} \qquad M_R = 78470 \text{ ft.-lbs.}$$

Location of resultant;

$$x = \frac{M_R - M_{OT}}{\Sigma W} = \frac{78470 - 18680}{13460} = 4.44 \text{ ft. from toe}$$

$$e = \frac{9.75}{2} - 4.44 = .43 \text{ to left of center}$$

Toe and heel pressures;

$$p = \frac{W}{A} \pm \frac{Wy}{I} = \frac{W}{A}\left(1 \pm \frac{6e}{d}\right) = \frac{13460}{9.75}\left(1 \pm \frac{6 \times .43}{9.75}\right)$$

$$p = \begin{cases} 1380 + 366 = 1746 \text{ psf toe} \\ 1380 - 366 = 1014 \text{ psf heel} \end{cases}$$

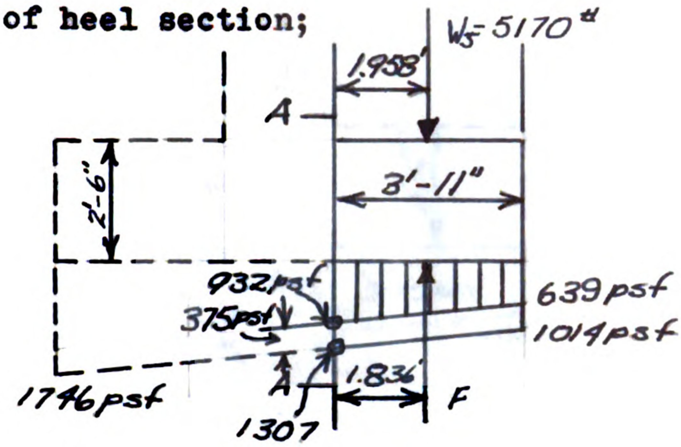
Saftey factors;

$$\text{overturning} \quad s.f. = \frac{M_R}{M_{OT}} = \frac{78470}{18680} = 4.2 \quad \text{o.k.}$$

$$\text{sliding} \quad s.f. = \frac{W \tan \delta}{P_h} = \frac{13460 \times 0.40}{3560} = 1.52 \quad \text{o.k.}$$

Abutments (cont.)

Check design of heel section;



$$\frac{732}{9.75} = \frac{x}{3.916}$$

$x = 293$ psf

$1014 + 293 = 1307$ psf

$$F = 3.916\left(\frac{932 + 639}{2}\right) = 3090 \text{ lbs.}$$

$$y = \frac{3.916}{3}\left(\frac{932 + 2 \times 639}{932 + 639}\right) = 1.835 \text{ ft.}$$

Max Shear and Moment on Section A A;

W_3	=	-5170 lbs;	x 1.958	=	-10,100 ft.-lbs.
earth P	=	+3090 lbs;	x 1.835	=	+5,850 ft.-lbs.
V	=	2080 lbs	M	=	4,250 ft.-lbs.

$$d = \frac{V}{vjb} = \frac{2080}{40 \times 7/8 \times 12} = 4.95 \text{ inches.}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{4250}{248}} = 4.14 \text{ inches}$$

d furnished = 27.0 inches

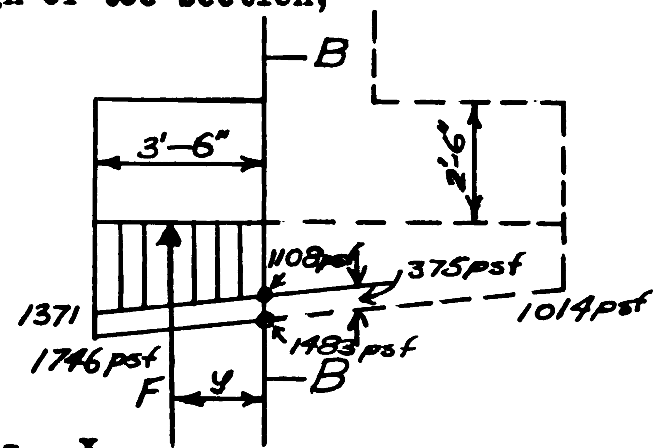
$$R = \frac{M}{bd^2} = \frac{4250 \times 12}{1 \times (27)^2} = 5.9 < 248 \therefore f_c < 1350 \text{ psi}$$

$$A_s = \frac{M}{f_s jd} = \frac{4250 \times 12}{18000 \times 7/8 \times 27} = .12 \text{ sq. inl/ft.}$$

A_s = furnished = .44 sq. in./ft.

Abutments (cont.)

Check design of toe section;



$$\frac{732}{9.75} = \frac{x}{3.5}$$

$$x = 263 \text{ psf}$$

$$F = 3.5 \left(\frac{1108 + 1371}{2}\right) = 4330 \text{ lbs.}$$

$$y = \frac{3.5}{3} \left(\frac{1108 + 2 \times 1371}{1108 + 1371}\right) = 1.855 \text{ ft.}$$

Max Shear and Moment on Section B B;

$$V = 4330 \text{ lbs.}$$

$$M = 4330 \times 18.55 = 8050 \text{ ft.-lbs.}$$

$$d = \frac{V}{vjd} = \frac{4330}{40 \times 7/8 \times 12} = 10.3 \text{ inches}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{8050}{248}} = 5.92 \text{ inches}$$

d furnished = 27.0 inches

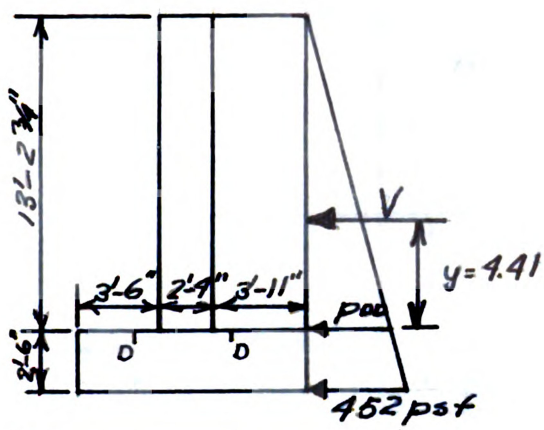
$$R = \frac{M}{bd^2} = \frac{8050}{1 \times (27)^2} = 11.9 < 248 \therefore f_c < 1350 \text{ psi}$$

$$A_s = \frac{8050 \times 12}{18000 \times 7/8 \times 27} = .245 \text{ sq. in./ft.}$$

A_s furnished = .60 sq. in./ft.

Abutments (cont.)

Check design of stem;



$$\frac{452}{15.728} = \frac{p_{DD}}{13.24}$$

$p_{DD} =$ 381 psf

$$y = \frac{13.24}{3} = 4.41 \text{ ft.}$$

Max Shear and Moment on Section D D;

$$V = \frac{13.24(381)}{2} = 2530 \text{ lbs.}$$

$$d = \frac{2530}{12 \times 7/8 \times 40} = 8.7 \text{ inches}$$

$$M = 2530 \times 4.41 = 11{,}200 \text{ ft.-lbs.}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{11200}{248}} = 6.72 \text{ inches}$$

d furnished = 25.0 inches

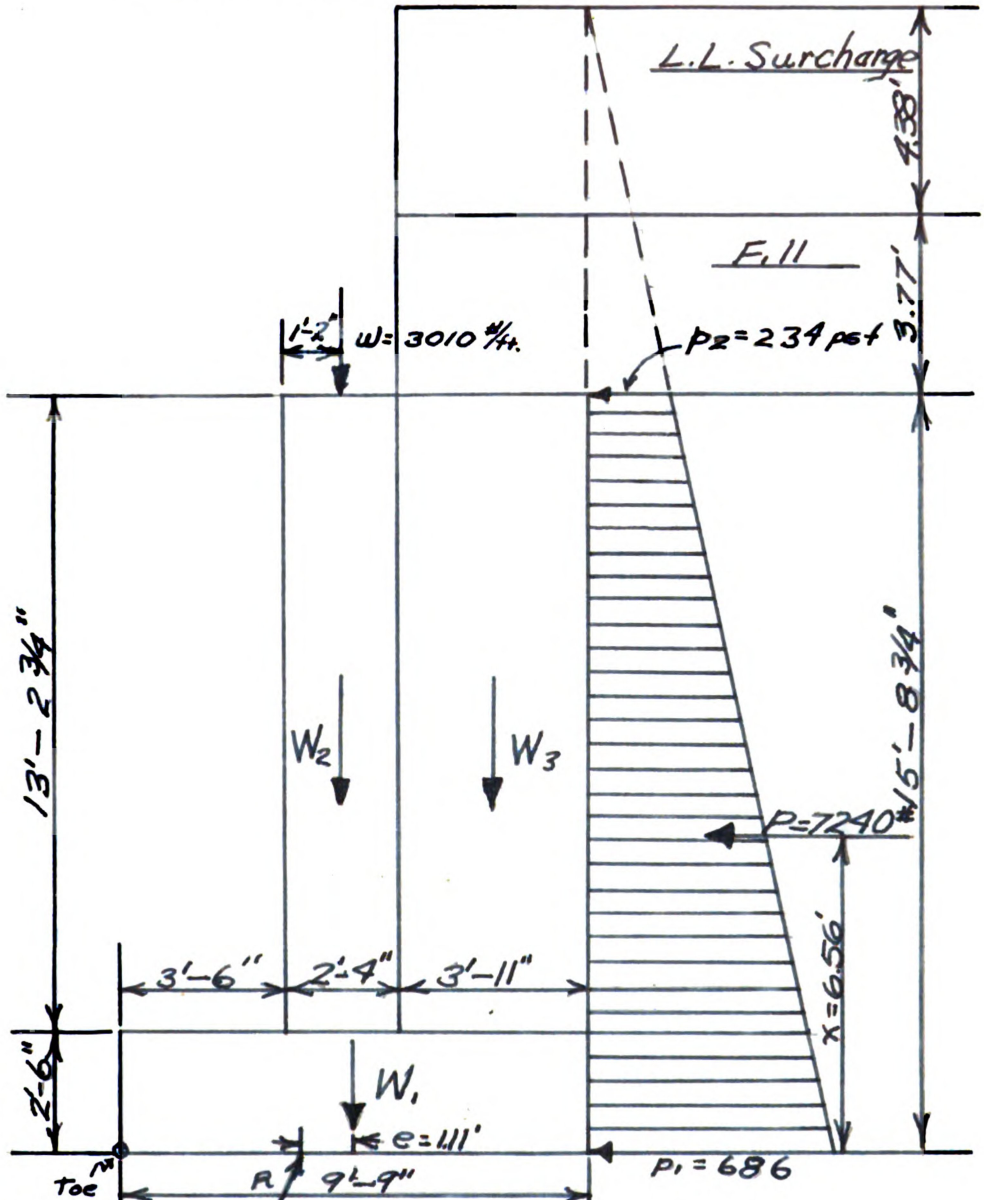
$$R = \frac{M}{bd^2} = \frac{11200}{1 \times (25)^2} = 17.9 < 248 \therefore f_c < 1350 \text{ psi}$$

$$A_s = \frac{M}{f_s jd} = \frac{11200 \times 12}{18000 \times 7/8 \times 25} = .317 \text{ sq in./ft.}$$

A_s = furnished = .79 sq. in./ft.

Abutments (cont.)

Case II -- Superstructure dead load and live load surcharge;



Superstructure dead load

steel = 63,380 lbs slab = 188,000 lbs

railing and post = 12,990 lbs total = 264, 370 lbs.

per abutment = 132,185 lbs per ft. = 3,010 lbs/ft.

$$\text{L.L. surcharge} = \frac{32{,}000}{14 \times 6 \times 100} + .15\left(\frac{32{,}000}{14 \times 6 \times 100}\right) = 4.38 \text{ ft.}$$

Abutments(cont.)

$p = 28.7\ h$

$p_1 = 28.7 \times 23.88 = 686$ psf

$p_2 = 28.7 \times 8.15 = 234$ psf

$P = \left(\frac{686 + 234}{2}\right) 15.73 = 7240$ lbs.

$x = \frac{15.73}{3}\left(\frac{686 + 2 \times 234}{686 + 234}\right) = 6.56$ ft.

$M_{OT} = 7240 \times 6.56 = 47{,}500$ ft.-lbs.

Base pressures; moments about toe;

$W_1 = 2.5 \times 9.75 \times 150 = 3660$ lbs; $\times\ 9.75 = 17820$ ft.-lbs.

$W_2 = 2.33 \times 13.23 \times 150 = 4630$ lbs; $\times\ (3.5 + \frac{2.33}{2}) = 20350$ ft.-lbs.

$W_3 = 3.92 \times 23.88 \times 100 = 9360$ lbs; $\times (5.834 + \frac{3.917}{2}) = 72900$ ft.-lbs.

$W = 3010$ lbs; $\times (3.5 + 1.17) = 14050$ ft.-lbs.

$\Sigma W = 20660$ lbs; $M_R = 125120$ ft.-lbs.

Location of Resultant;

$x = \frac{M_R - M_{OT}}{\Sigma W} = \frac{125120 - 47500}{20660} = 3.76$ ft. from toe

$e = 4.87 - 3.76 = 1.11$ to left of center

toe and hell pressures;

$p = \frac{W}{A}\left(1 \pm \frac{6e}{d}\right) = \frac{20660}{9.75}\left(1 \pm \frac{6 \times 1.11}{9.75}\right)$

$p = \begin{cases} 2120 + 1450 = 3570 \text{ psf toe} \\ 2120 - 1450 = 670 \text{ psf heel} \end{cases}$

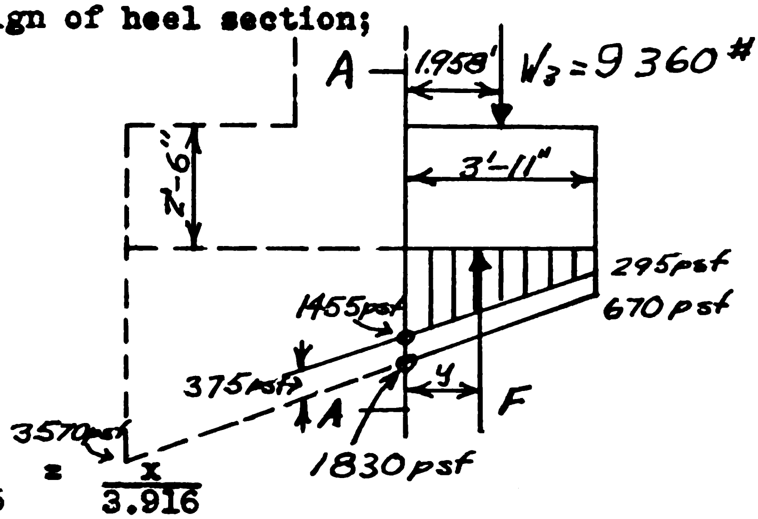
Saftey fadtors;

overturning s.f. $= \frac{M_R}{M_{OT}} = \frac{125120}{47500} = 2.64$

sliding s.f. $= \frac{W \tan\phi}{P_h} = \frac{20660 \times 0.40}{7240} = 1.14$

Abutments (cont.)

Check design of heel section;



$$\frac{2900}{9.75} = \frac{x}{3.916}$$

$x = 1160$ psf

$670 + 1160 = 1830$ psf

$$F = 3.916 \left(\frac{1455 + 295}{2}\right) = 3420 \text{ lbs.}$$

$$y = \frac{3.916}{3} \left(\frac{1455 + 2 \times 295}{1455 + 295}\right) = 1.525 \text{ ft.}$$

Max Shear and Moment on section A A;

W_3	=	-9360 lbs;	x	1.958	=	-18310 ft.-lbs.
earth P	=	+3420 lbs;	x	1.525	=	+ 5210 ft.-lbs.
V	=	5940 lbs;		M	=	13100 ft.-lbs.

$$d = \frac{V}{vjb} = \frac{5940}{40 \times 7/8 \times 12} = 14.1 \text{ inches}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{13100}{248}} = 7.26 \text{ inches}$$

d furnished = 27.0 inches

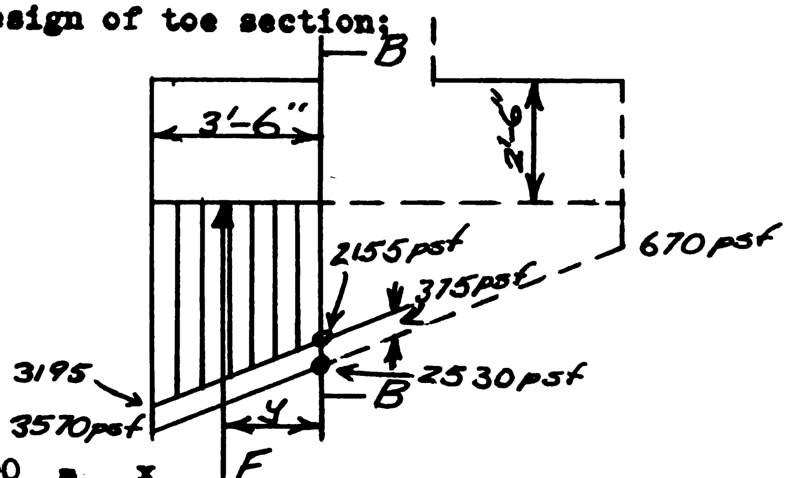
$$R = \frac{M}{bd^2} = \frac{13100}{1 \times (27)^2} = 18 < 248 \therefore f_c < 1350 \text{ psi}$$

$$A_s = \frac{M}{f_s jd} = \frac{13100 \times 12}{19000 \times 7/8 \times 27} = .37 \text{ sq. in./ft.}$$

A_s furnished = .44 sq. in./ft.

Abutments (cont.)

Check design of toe section:



$$\frac{2900}{9.75} = \frac{x}{3.5}$$

$x = 1040$ psf

$3570 - 1040 = 2530$ psf

$$F = 3.5\left(\frac{2155 + 3195}{2}\right) = 9380 \text{ lbs.}$$

$$y = \frac{3.5}{3}\left(\frac{2155 + 2 \times 3195}{2155 + 3195}\right) = 2.065 \text{ ft.}$$

Max Shear and moment on Section B B;

$V = 9380$ lbs.

$M = 9380 \times 2.065 = 19350$ ft.-lbs.

$$d = \frac{V}{vjb} = \frac{9380}{40 \times 7/8 \times 12} = 22.3 \text{ inches}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{19350}{248}} = 8.84 \text{ inches}$$

d furnished $= 27.0$ inches

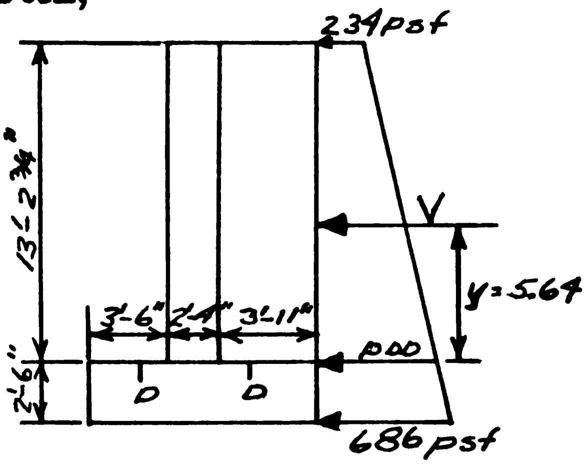
$$R = \frac{M}{bd^2} = \frac{19350}{1 \times (27)^2}$$

$$A_s = \frac{M}{f_s jd} = \frac{19350 \times 12}{18000 \times 7/8 \times 27} = .547 \text{ sq. in./ft.}$$

A_s furnished $= .60$ sq. in./ft.

Abutments (cont.)

Check design of stem;



$$\frac{686}{23.88} = \frac{p_{DD}}{21.44}$$

$p_{DD} = 614$ psf

$$y = \frac{13.24}{3}\left(\frac{614 + 2 \times 234}{614 + 234}\right) = 5.64 \text{ ft.}$$

Mas Shear and Moment on Section D D;

$$V = \frac{13.24}{2}(234 + 614) = 5610 \text{ lbs.}$$

$$d = \frac{5610}{12 \times 7/8 \times 40} = 13.4 \text{ inches}$$

$$M = 5610 \times 5.64 = 31600 \text{ ft.-lbs.}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{31600}{248}} = 11.3 \text{ inches}$$

d furnished = 25.0 inches

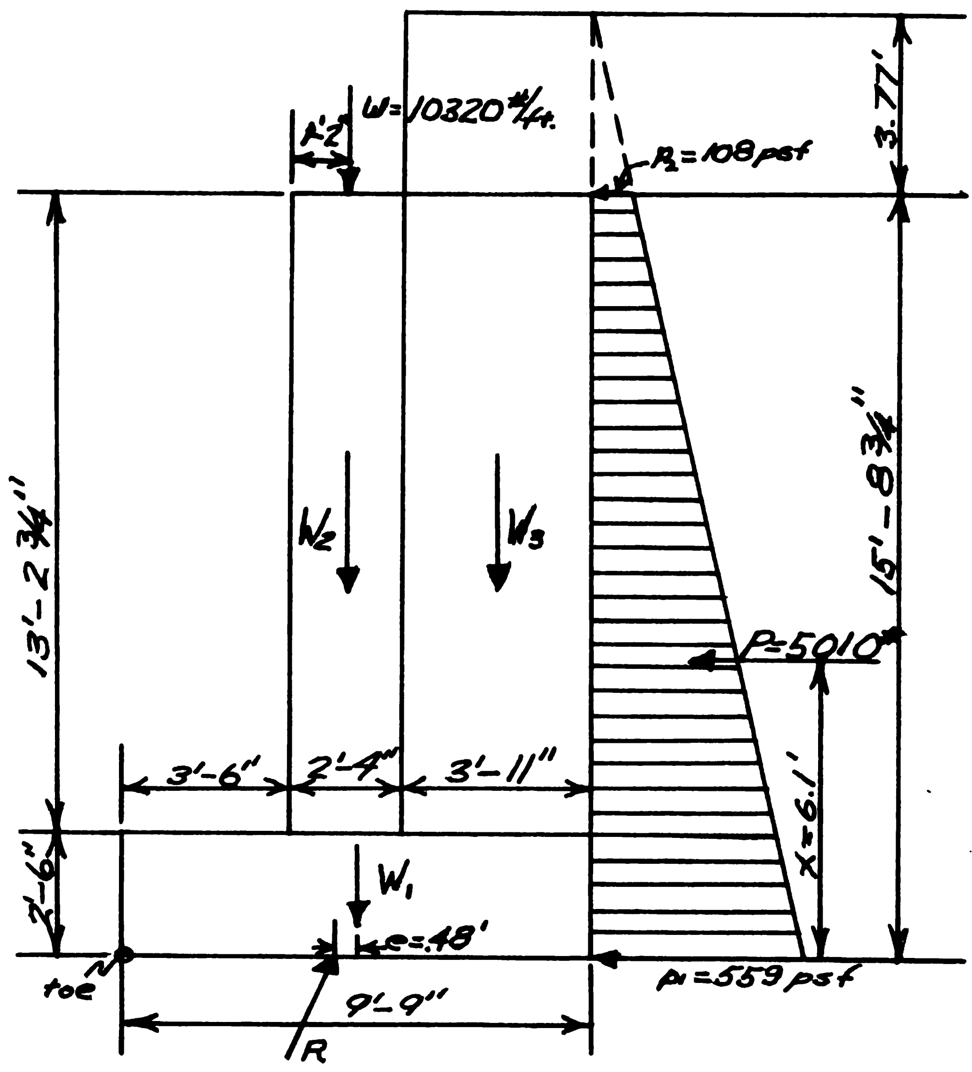
$$R = \frac{31600}{1 \times (25)^2} = 50.5 < 248 \therefore f_c < 1350 \text{ psi}$$

$$A_s = \frac{M}{f_s j d} = \frac{31600 \times 12}{18000 \times 7/8 \times 25} = .89 \text{ sq. in./ft.}$$

A_s furnished = .79 sq. in./ft.

Abutments (cont.)

Case III -- Superstructure and live load - No surcharge;



Superstructure dead load

D.L. = 3010 lbs./ft.

Max L.L. per stringer = 29200 lbs

29200 x 9 stringers = 263000 lbs

L.L. = $\frac{263000}{44}$ = 5990 lbs./ft. I (22%) = 1,320 lbs/ft.

Abutments (cont.)

$p = 28.7\ h$

$p_1 = 28.7 \times 19.5 = 559$ psf

$p_2 = 28.7 \times 3.77 = 108$ psf

$P = \left(\frac{559 + 108}{2}\right) 15.73 = 5010$ lbs.

$x = \frac{15.73}{3} \left(\frac{559 + 2 \times 108}{559 + 108}\right) = 6.1$ ft.

$M = 5010 \times 6.1 = 30550$ ft.-lbs.

Base pressures; Moments about toe;

$W_1 = 2.5 \times 9.75 \times 150$	$= 3660$ lbs;	$\times \frac{9.75}{2}$	$= 17820$ ft.-lbs.
$W_2 = 2.33 \times 3.23 \times 150$	$= 4630$ lbs;	$\times \left(3.5 - \frac{2133}{2}\right)$	$= 20350$ ft.-lbs.
$W_3 = 3.92 \times 19.50 \times 100$	$= 7650$ lbs;	$\times (5.834 - 3.917)$	$= 59500$ ft.-lbs.
W	$= 10320$ lbs;	$\times 4.67$	$= 48100$ ft.-lbs.
ΣW	$= 26{,}260$ lbs	$M_R =$	145770 ft.-lbs.

Location of Resultant;

$x = \frac{M_R - M_{OT}}{\Sigma W} = \frac{145770 - 30550}{26260} = 4.39$ ft. from toe

$e = 4.87 - 4.39 = .48$ ft. to left of center

Toe and hell pressures;

$p = \frac{W}{A}\left(1 \pm \frac{6e}{d}\right) = \frac{26260}{9.75}\left(1 \pm \frac{6 \times .48}{9.75}\right)$

$p = \begin{matrix} 2700 + 796 = 3496 \text{ psf toe} \\ 2700 - 796 = 1904 \text{ psf heel} \end{matrix}$

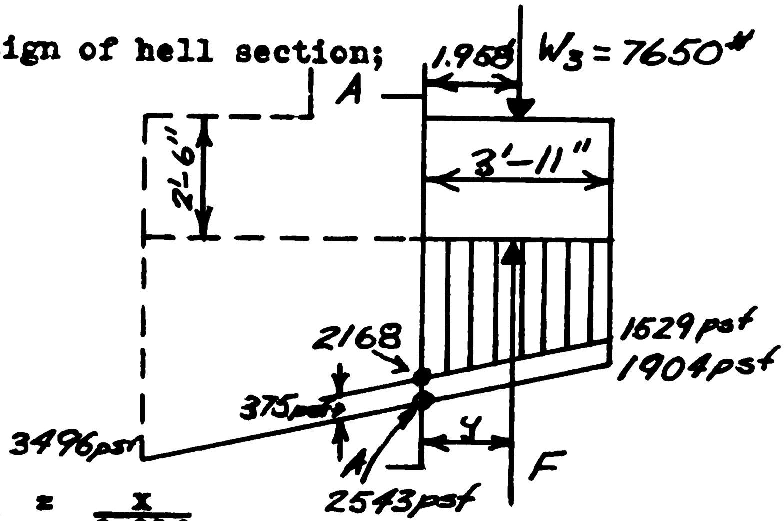
Saftey factors;

overturning $s.f. = \frac{M_R}{M_{OT}} = \frac{145770}{30550} = 4.77$ o.k.

sliding $s.f. = \frac{W \tan\phi}{P_h} = \frac{26260 \times 0.40}{5010} = 2.09$ o.k.

Abutments (cont.)

Check design of hell section;



$$\frac{1592}{9.75} = \frac{x}{3.916}$$

$x = 639$ psf

$1904 + 639 = 2543$ psf

$$F = 3.916 \left(\frac{2168 + 1529}{2}\right) = 7420 \text{ lbs.}$$

$$y = \frac{3.916}{3} \left(\frac{2168 + 2 \times 1529}{2168 + 1529}\right) = 1.845 \text{ ft.}$$

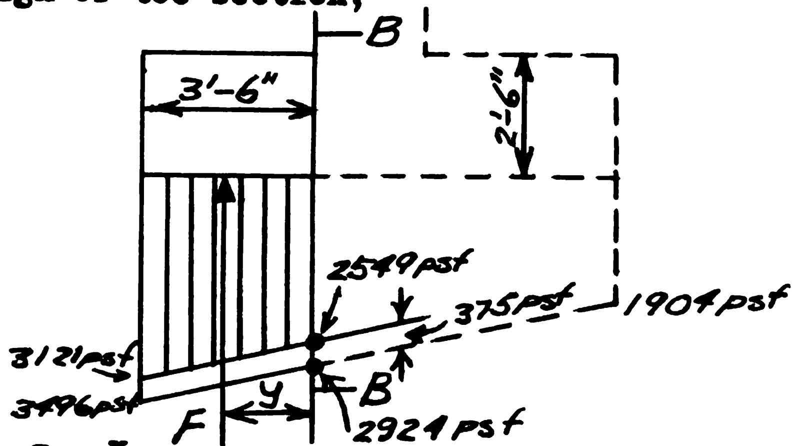
Max Shear and Moment on Section A A;

W_3	=	-7650 lbs;	x	1.958	=	-15000 ft.-lbs.
earth P	=	+7420 lbs;	x	1.845	=	+13700 ft.-lbs.
V	=	230 lbs;		M	=	1300 ft.-lbs.

The shear and moment is so small there will be no need to check for (d) and A_s in this case for the heel section.

Abutments (cont.)

Check design of toe section;



$$\frac{1952}{9.75} = \frac{x}{3.5}$$

$$x = 572 \text{ psf}$$

$$3496 + 572 = 2924 \text{ psf}$$

$$F = 3.5 \left(\frac{3121 + 2549}{2}\right) = 10100 \text{ lbs.}$$

$$y = \frac{3.5}{3} \left(\frac{2549 + 2 \times 3121}{2549 + 3121}\right) = 1.840 \text{ ft.}$$

Max Shear and Moment on Section B B;

$$V = 10100 \text{ lbs}$$

$$M = 10100 \times 1.840 = 18600 \text{ ft.-lbs.}$$

$$d = \frac{V}{vjb} = \frac{10100}{40 \times 7/8 \times 12} = 24.1 \text{ inches}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{18600}{248}} = 8.67 \text{ inches}$$

d furnished = 27.0 inches

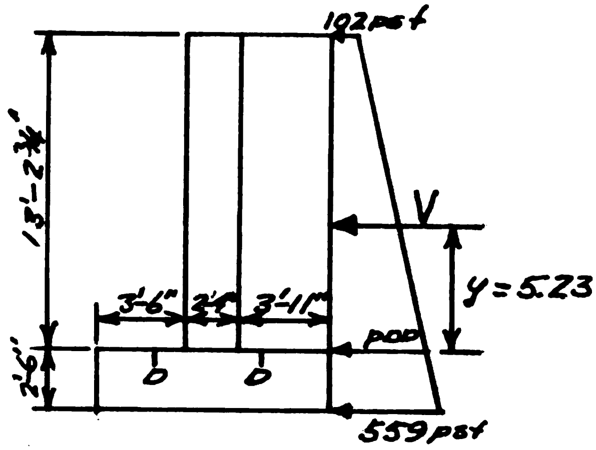
$$R = \frac{18600}{1 \times (27)^2} = .526 \text{ sq. in./ft}$$

$$A_s = \frac{M}{f_s jd} = \frac{18600 \times 12}{18000 \times 7/8 \times 27} = .526 \text{ sq. in./ft.}$$

A_s furnished = .60 sq. in./ft.

Abutments (cont.)

Check design of stem;



$$\frac{559}{19.5} = \frac{p_{DD}}{17.06}$$

$p_{DD} =$ 489 psf

$$y = \frac{13.24}{3}\left(\frac{489 + 2 \times 108}{489 + 108}\right) = 5.23 \text{ ft.}$$

Max Shear and Moment on Section D D;

$$V = \frac{13.24}{2}(108 + 489) = 3930 \text{ lbs.}$$

$$d = \frac{3930}{12 \times 7/8 \times 40} = 9.36 \text{ inches}$$

$$M = 3930 \times 5.23 = 20{,}500 \text{ ft.-lbs1}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{20500}{248}} = 9.1 \text{ inches}$$

d furnished = 25.0 inches

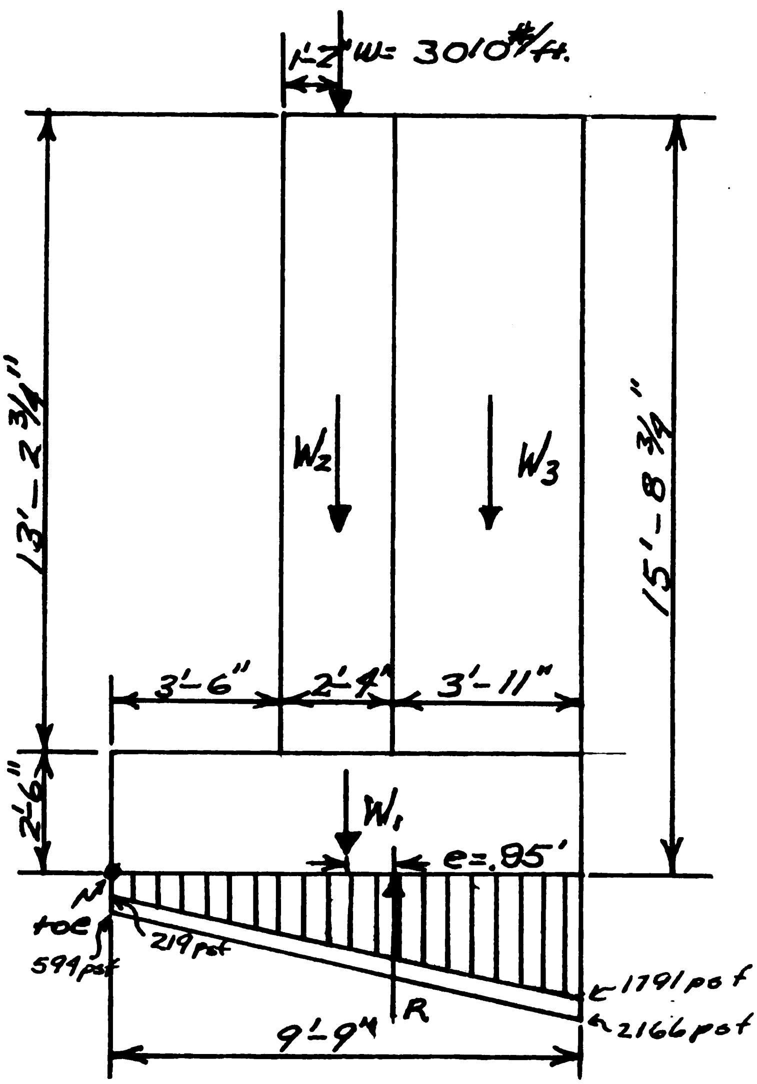
$$R = \frac{M}{bd^2} = \frac{20500}{1 \times (25)^2} = 32.8 < 248 \therefore f_c < 1350 \text{ psi}$$

$$A_s = \frac{M}{f_s jd} = \frac{20500 \times 12}{18000 \times 7/8 \times 25} = .626 \text{ sq. in./ft.}$$

A_s furnished = .79 sq. in./ft.

Abutments (cont.)

Case IV -- Superstructure dead load - No surcharge L.L. or backwall friction.



Superstructure dead load;

3,010 lbs./ft. of wall

With no backwall friction will assume no horizontal force therefore there will be no moment cause overturning.

Abutments (cont.)

Base pressures; moments about toe;

W_1 = 2.5x9.75x150	= 3660 lbs;	x 9.75	=17820 ft.-lbs.
W_2 = 2.33x13.23x150	= 4630 lbs;	x $(3.5-\frac{2.33}{2})$	=20350 ft.-lbs.
W_3 = 3.92x 13.23x 100	= 5170 lbs;	x(5.834-3.917)	=40300 ft.-lbs.
W	= 3010 lbs;	x(3.5 - 1.17)	=14050 ft.-lbs.
	W =16470 lbs.		M =92520 ft.-lbs.

Location of Resultant;

$$x = \frac{M - M}{W} = \frac{78470 - 0}{13460} = 5.82 \text{ ft from toe}$$

$$e = 5.82 - 4.87 = .95 \text{ ft to right of center}$$

Toe and heel pressure;

$$p = \frac{W}{A}\left(1 \pm \frac{6e}{d}\right) = \frac{13460}{9.75}\left(1 \pm \frac{6 \times .95}{9.75}\right)$$

$$p = \begin{array}{ccccl} 1380 & - & 786 & = & 2166 \text{ psf heel} \\ 1380 & - & 786 & = & 594 \text{ psf toe} \end{array}$$

As there is no horizional force there will be no sliding or overturning, therefore there will be no safety factors to considered.

The toe and heel section will not be checked because this is not a maximum case as can be seen. Also as there is no horizonal force there will be no action on the vertical stem.

Conclusion

It has been found that the design of the bridge in this thesis will meet all the specifications and standards that are required by the Michigan State Highway Department.

The analysis of this bridge shows that in all parts the bridge is overdesigned. This is true in some parts more than in others. The bridge railings and posts show this, as they are designed by standard methods, which are more for looks than for any true loading. This is to present a more appealing appearance to the public using the bridge. Also, thesavings in theuse of the standard design, rather than separate designs for each bridge, are far greater than the savings in material and labor. In the other parts the tendency is to go to a larger section to a certain percent , rather than to a smaller section, which would only be underdesigned by one or two percent. This is because with the advancement in machines, the bridge may be called upon to sometimes withstand greater loads than for which they are designed.