A CHECK OF THE DESIGN OF A STANDARD HIGHWAY BRIDGE

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Richard Porter Gillespie

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A CHECK OF THE DESIGN OF A

STANDARD HIGHWAY BRIDGE

A THESIS SUBMITTED TO

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RICHARD PORTER GILLESPIE

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INTRODUCTION

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

This bridge has been constructed in Sec. 10 and 11 T. 8 S R 3 E Fairfield township, Lenawee County. The bridge crosses the black creek 0.5 miles south of the town of Jasper, Michigan.

The existing structure has been removed because of the need for a new and larger bridge.

The span length of this bridge is 50' - 0" exact and the overall roadway width from curb to curb curb is 38' - 0" exact.

Stream Data: Black Creek is a very meandering stream and there is indications of scouring at the bends where there is no willow growth. The water has a very offensive odor due to refuse from a creamery located two miles upstream from the project.

The elevation of the water surface at the survey center line is 710.15 300 ft. upstream measured at right angles to the survey line, it is 710.82 and 300 ft. downstream it is 709.58. High water elevation was 716.9 and the normal highwater elevation is 713.9'.

Soil Conditions: The soil in this vicinity is a heavy clay loam, but there is some indication of sand and gravel in the bed of the stream.

The specifications are those of the Michigan State Highway Department published in 1944 and from the specifications for highway design published by the American Association of State Highway Officials in 1944.

The maximum allowable unit stresses used are from the Bridge Division, Michigan State Highway Department specifications.

fs = 18,000 psi
fc = 3,000 psi
fc = 1,200 psi
j = $\frac{7}{8}$ = .867
k = .400
n = 10
R = 209
V = 60 psi
v = 90 psi
U = 150 psi
u = 300 psi (special anchorage)

Loading H - 20 S16 - 44

Shear 13,500 psi power driven rivets

Shear 10,000 psi unfinished bolts

SYMBOLS

A	Area
b	Breath or width
d	Depth of beam to center of steel
	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 elasticity of steel to modulus of elasticity of concrete
R	Reaction
V	Total shear
v	Unit shear
W	Load per unit of length
Ø	Round
U	Bond

ABBREVIATIONS

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

Design of Railing:

Railings

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 the 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 (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).

Forces on Railings

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 railing adjacent to the roadway, the bottom rail shall be designed for a horizontal force of 300# lineal foot of rail, if the rail is properly reinforced the force of 300# lineal foot may be used but if the rail is not properly reinforced use 500# lineal foot. (M.S.H.D. Spec. 35, p. 14).

Design of Railing

Railings:

Bolts $\frac{3}{4}$" Ø in single shear

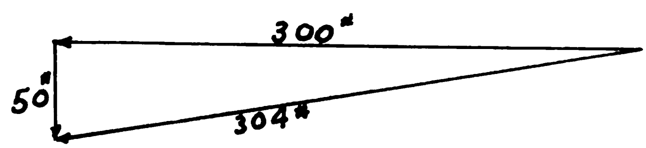
P = S .A P = 10,000 x .4418 = 4,420# Capacity

Load $\frac{300 \times 8.167}{2} = 1225\#$

Strap shear capacity $(1\frac{3}{4} - \frac{13}{16})\ \frac{3}{4} \times 10{,}000 = 9360\#$

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

Use 50# lineal foot for the dead weight of the railing.



Load is $\frac{304 \times 8.167}{2} = 1255\ \#$

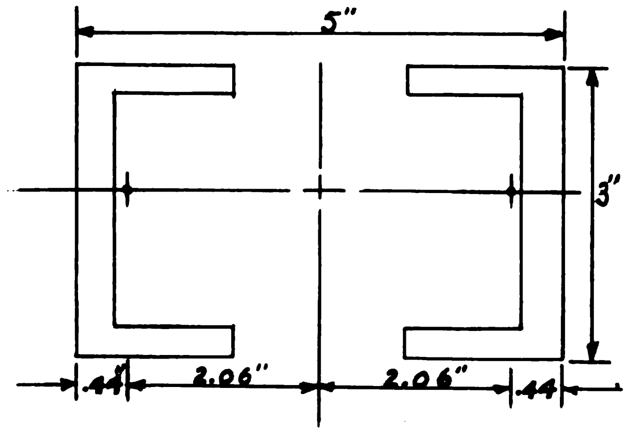
Bolts will control the design as they have the minimum

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

$$f = \frac{Mc}{I} \quad \text{where } I = \frac{bh}{6}$$

$$f\ (\text{Horizontal}) = \frac{1225 \times 2.25 \times 6}{.625 \times (1.75)} = 8650 \text{ psi}$$

$$f\ (\text{vertical}) = \frac{1225 \times 2.25 \times 6}{6 \times 1.75 \times (6.25)} = \frac{4050 \text{ psi}}{12{,}700 \text{ psi}} \text{ ok} < 18{,}000 \text{ psi}$$



Design of Railing (cont.)

Rail:

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

Horizontal bending $8\text{-}167 - (2\text{x}\ 1\frac{5}{8}) = 7.90$ ft.

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

$A = 2 \times 1.46 = 2.92$ sq. in. $\quad e = 2.06 \quad Ig = 2\text{x}\ .25 = 50.$ in.

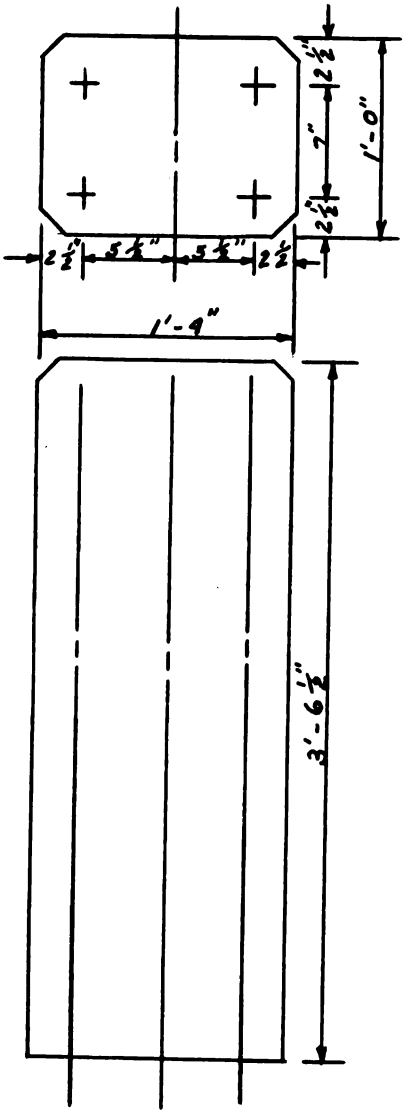
$I = Ae^2 + Ig = 2.92 \times (2.06)^2 + 50 = 12.89$ in.

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

18,000 psi allowable

$\frac{18,000}{5450} = 331\%$ over designed we may neglect the design of the top channel of the lower railing in the design because the stresses are low. This top channel would increase the strength at the railing which is overdesigned already.

Posts



Rc 3 Intermediate Posts standard

M.S.H.D.

b = 1' - 4" = 16"

d = 9½"

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

(2 bars in compression)

(2 bars in tension)

$$k = \frac{1}{fs/nfc \quad 1} = \frac{1}{18000/10 \times 1200 + 1}$$

$k = .400$

$$j = (1 - \frac{k}{3}) = (1 - \frac{.400}{3}) = .867$$

Bending moments:

150 x 9.5 x 32 =	45,300"#
300 x 9.5 x 5 =	14,200"#
Totals	59,500"#

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{59,500}{(209)(16)}} = 4.2 \text{ inches required}$$

$$fs = \frac{M}{As\,jd} = 59,500 / .88 \times .867 \times 9.5 = 8230 \text{ psi.}$$

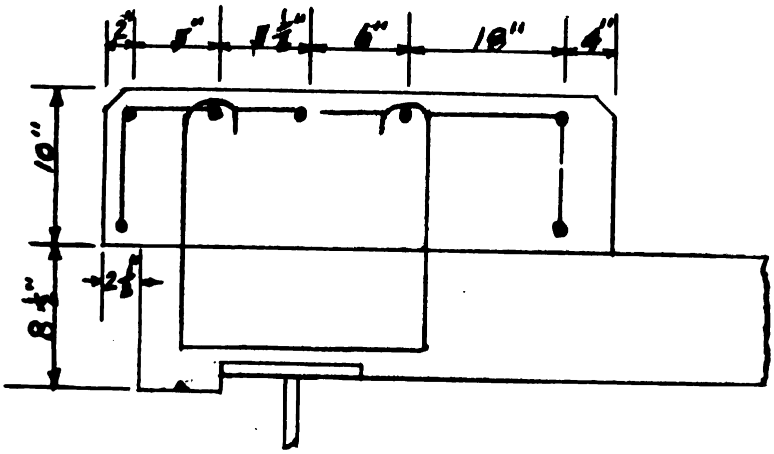
18,000 allowable

$$\frac{18,000}{8,230} = 219\% \text{ over designed.}$$

Posts:

The railing and posts are greatly over designed but part of this is due to the fact that large posts and railings produce a more balanced looking bridge in appearance.

Design of Sidewalk and Curb:



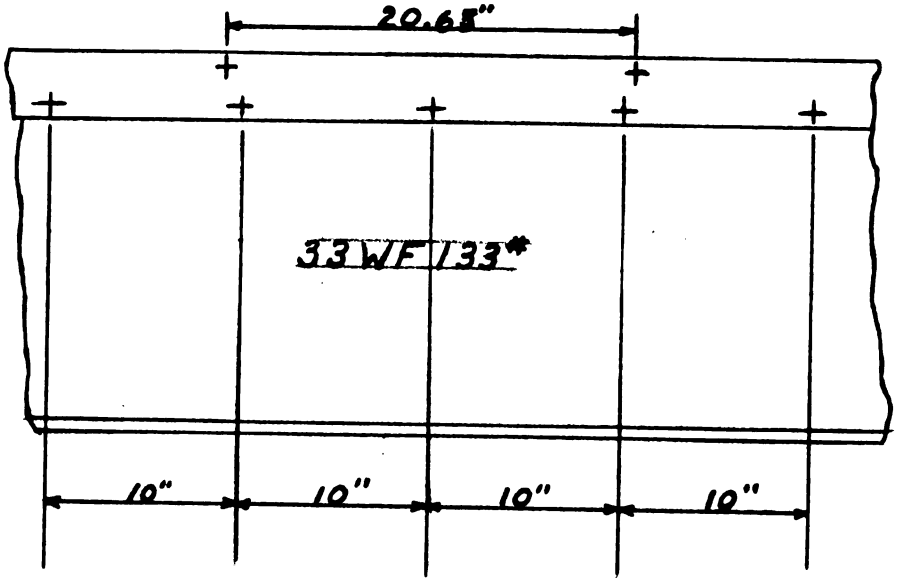
Curbs shall be designed to resist a force of not less than five hundred (500) pounds per lineal foot of curb applied at the top of the curb.

(M.S.H.D. Spec. 3b, 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. The only steel required is temperature steel, the rest is put in to bind the structure together.

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 a span of 2 to 7 ft.

$E = 0.65 + 2.5$

Bending moment for continuous span

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

Bending moment for freely supported span.

$M = 0.25 \times P/E \times S \times (100 + I + 10\%$ 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 ½ width of girder flanges.

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

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.

$$I = \frac{L + 20}{6L + 20} = .22$$

Design

$E = 0.6 \times 5.14 + 2.5 = 5.59$ ft.

$M = 0.25 \times 16{,}000/5.59 \times 5.14 \times 1.32 = 4860$ ft lbs.

$d = \sqrt{\frac{M}{Rb}} = \sqrt{4860 \times 12/209 \times 12} = 4.82$ 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: 4.82 1.5" = 6.32 in

Actual 7.00 in.

$As = \frac{M}{fsjd} = (4860 \times 12)/18000 \times \frac{7}{8} \times 5.59 = .675$ in req.

Actual 1.000 sq. in.

Steel at bottom of slab for lateral distribution

Percent of main steel required:

$100/\sqrt{s}$ $100/5.4 = 44.1\%$

(A.A.S.H.O. Art. 3. 2.2 p. 140)

$As = .675 \times .441 = .298$ 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 1½ 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. 38, p. 15)

(a) Size of Rivets shall be of the size specified but generally shall be 3/4" or 7/8" 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" diameter rivets - 2½". (M.S.H.D. Spec. 92, p.39-40).

The end connections angles of floor beams and stringers shall be not less than 3/8 inch in finished thickness. (M.S.H.D. Spec. 120, p. 47).

(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)

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).

Area of structure as seen in elevation:

7.89 x 50	400 sq. feet
400 x 1.5	600 sq. feet
.5 x 6 x 3 x 47.5	427 sq. feet
Total effective area	1027 sq. feet

Moving load 30 x 1027 = 30,800#

Area required for diaphragm

$$\frac{30,800}{18,000} = 1.71 \text{ sq. inches}$$

Area furnished

4" x 4" x 3/8" furnished 2.86 sq. inches.

The diaphragms placed at 1/3 the width meet all of the necessary M. S. Highway standards they are used principally to keep the beams from turning and also to keep the I beams on the same level. The specifications provided by the M.S.H.D. for depth of web, size of angles, pitch of rivets, depth of tie angles and number of stiffeners place the use of these within the proper limit of design.

Girder Design:

For structures with concrete slab floors with out 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).

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. 76, p. 36).

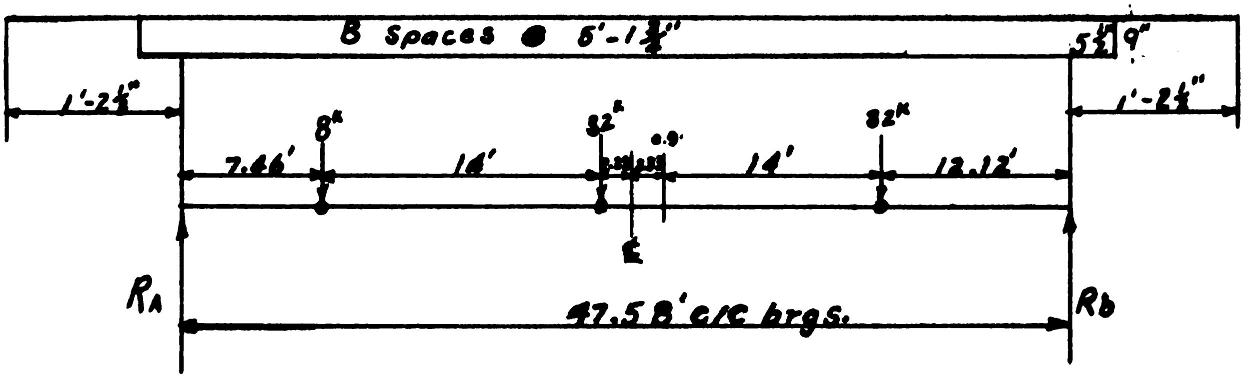
Calculations of stresses, span lengths shall be assumed as follows:

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)

Using H 20 S 16-44 Loading from appendix A. (A.A.S.H.O. p. 229).

Total Span 50'



Check final design of Beams using H20S16-44 loading:

Assumptions 38' Rdwy, 2 walks @ 1'-6" 7"min slab.

One span 50' between R. P. 90° ∠ of crossing

Try 33" WF 132# stringers $\frac{L}{800}$

C.G. Take moments about small wheel 8,000# load

```
32000 x 14  = 448,000
32000 x 28  = 896,000
 8000 x  0  =       0
-------------------------
72000       = 1,344,000' #
      ÷72   =  18.66 to c. g. from 8,000# load
```

Max moment occurs when ℄ of Span is midway between center of gravity of loads and nearest load.

18.66 - 14.00 = 4.66' c. g. to near load

$\frac{4.66}{2}$ = 2.33' distance c.g. placed off centers

<u>L. L. moments</u> to determine L. L reactions RA and RB

```
RA  32,000 x 12.12  =  387,840
    32,000 x 26.12  =  835,840
     8,000 x 40.12  =  320,960
                     1,544,640 ÷ 47.58 = 32,460  R.A.
```

R_B

8,000 x 7.46 = 59,680
32,000 x 21.46 = 686,720
32,000 x 35.46 = 1,134,720
1,881,120 ÷ 47.58 = 39,540# R_B

72,000# total O.K.

Check final design $\frac{L}{800}$

Max moment occurs under middle wheel.

RA 32,460 x 21.46 = 696,590'#
- 8,000 x 14 = - 112,000

Max mom L.L = 584,590'# lane

Try 33' WF 132# = 8 spaces @ 5' - 1 3/4" = 41'- 2"

Distribution L.L. Mom. C.M. ÷ M = $\frac{584,590 \times 5.15}{10}$

= 301,060'#/ beam

Dead load

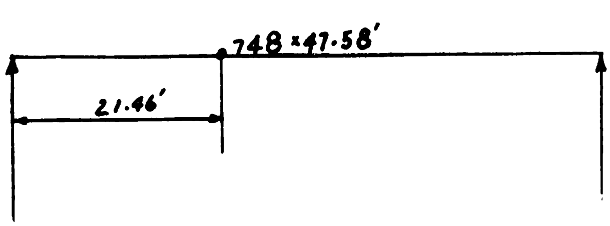
Conc. 150 x .651 x 5.15 = 503#

Future wearing surface 20 x 5.02 = 100

Beam 10% for Diaphs. and = 145

748#/ ft of beam

D. L. Mom (under same wheel)



$$R = \frac{748 \times 47.58}{2} = 17{,}900\#$$

$$\begin{aligned} M_{DL} &= 17{,}900 \times 21.46 &&= 384{,}000 \\ &- 748 \times 21.46 \times 10.73 &&= \underline{-173{,}000} \\ & && 211{,}000 \ \#^{1}/ \text{ beam} \end{aligned}$$

$$\text{Impact Factor } \frac{L + 20}{6L + 20} = \frac{47.58 + 20}{6 \times 47.58 + 20} = \frac{67.58}{305.48} = 0.221$$

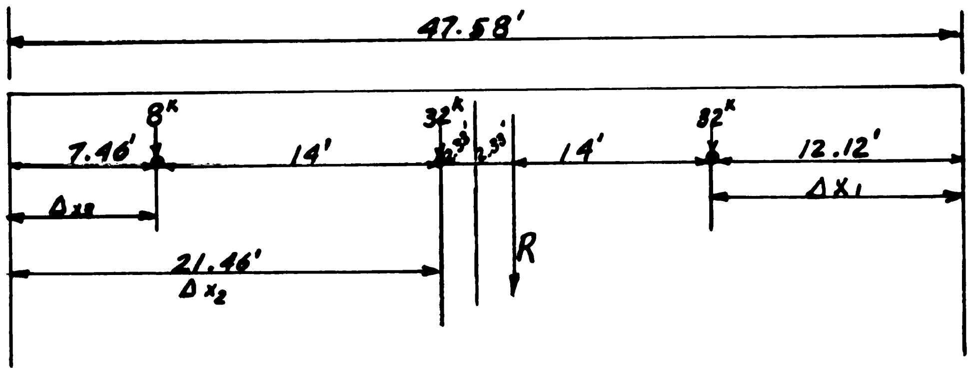
Total Moment

$$\begin{aligned} &\text{L. L. + Impact} = 301{,}060 \times 1.221 &&= 367{,}590 \\ &\text{Max. D. L.} &&= \underline{211{,}000} \\ & && 578{,}590 \end{aligned}$$

$$S = \frac{578{,}590 \times 12}{18{,}000} = 390'' \text{ req'd.}$$

A 33' WF 132# gives S = 404.8#^{3} provided

Check Deflection



Test Deflection by Lane Loading as per A.A.S.H.O. - 44.

By Maxwell's Theorem $\Delta x = \frac{WX}{48\ EI}\ (3l^2 - 4\ x\ ^2)$

$D = .713''$ allowed $\frac{L}{800} = 0.713''$

max $\Delta x_1 + \Delta x_2 + \Delta x_3$

$\Delta x_1 = \frac{32{,}000 \times (12 \cdot 12) \times 12}{48 \times 29{,}000{,}000 \times 6856.8} \left[(3 \times (47.58)^2 - 4\ (12.12)^2 \times (12)^2\right]$

$\Delta x_1 = \frac{4{,}157{,}838{,}720{,}000}{48 \times 29 \times 10^6 \times 6856.8} = .436''$

$\Delta x_2 = \frac{32{,}000 \times 21.46 \times 12}{48\ EI} \left[3 \times (47.58)^2 \times (12)^2 - 4(21.46)^2 \times (12)^2\right]$

$\Delta x_2 = \frac{5{,}873{,}268{,}940{,}800}{9{,}544{,}665{,}600{,}000} = .615$

$\Delta x_3 = \frac{8{,}000 \times 7.46 \times 12}{48\ EI} \left[3 \times (47.58)^2 \times (12)^2 - 4(7.46)^2 \times (12)^2\right]$

$= \frac{677{,}437{,}944{,}960}{9{,}544{,}665{,}600{,}000} = .071$

$I = \frac{L+20}{6L+20} = \frac{47.58' + 20}{6 \times 47.58' + 20} = .221$

.436
.615
.071

Δ Total = 1.122/lane

D = 1.121 x 1.221 x .515 = .705"/Beam Load

Allowable = .713"

Use a 33" WF @ 132# this is the smallest beam which does not exceed the allowable deflection.

Abutment Design:

"Rankines" Theory of pressure distribution in non-cohesive granular material, provides that no structure shall be designed for an equivalent fluid pressure of less than 30 pounds per square foot.

Retaining walls, abutments and structures built to retain fills shall be designed to resist pressure determined in accordance with the Rankin theory.

Abutment Design

45' - 0"   O-O sholders @ bridge from prelim.

℄ to sholder   22' - 6"

Dist.from bridge seat to top of curb .

½" Masonry Plate

2" Bearing Plate

33" Girder

7" Slab

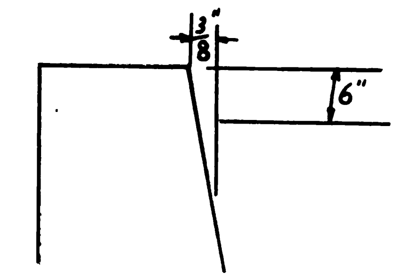
10" Curb

52½" = 4' - 4½"   seat to top curb.

4' - 4½
   6

4' - 10½ (hgt.from top of curb to inter. of grd. and back of wing).

(4'-10½) x 2   9' - 9" (from sholder to back of wing)



Wing ht. = 13' - 2 3/4" = 158 3/4" (prelim plan)

Total batter = (2' - 4") -1'-6") = 10"

$$\frac{10'' \times 6}{158.75} = .378'' = 3/8''$$

Dist ℄ to out of wing-

22' - 6"
9' - 9"
3/8"
1' - 6"
33' - 9 3/8"   use 33' - 9½"

O - O Wing walls   (33' - 9½") x 2 = 67' - 7")

Dead Load Superstructure Reaction on Each Abutment.

Sidewalk = 2.71 x .83 x 50 = 112 cu.ft.

Slab = 43 x .65 x 46.75 x .5 = 654 cu. ft.

Backwall:

Length = 21' - 11½ x 2 = 43' - 11"

Av. hgt. = 3' - 7 7/8 = 3.66'

(43.92 x 3.66 x 1.62) - (.5 x 38 x 1.16) = 238

1004 cu.ft.

1004 x 150 = 150,600# wgt. conc.

Steel Rail 50 x 150 = 7,500#

F.W.S. 38 x 20 x 24.5 = 18,600#

Str. Steel and Diaph

48.5 x 9 x 125 x 1.05 x .5 = 28,650#

Fillet Walls

(1.75 x 10 x 5 x ½) x 2 x 150 = 13,100#

Total D. L. 218,450#

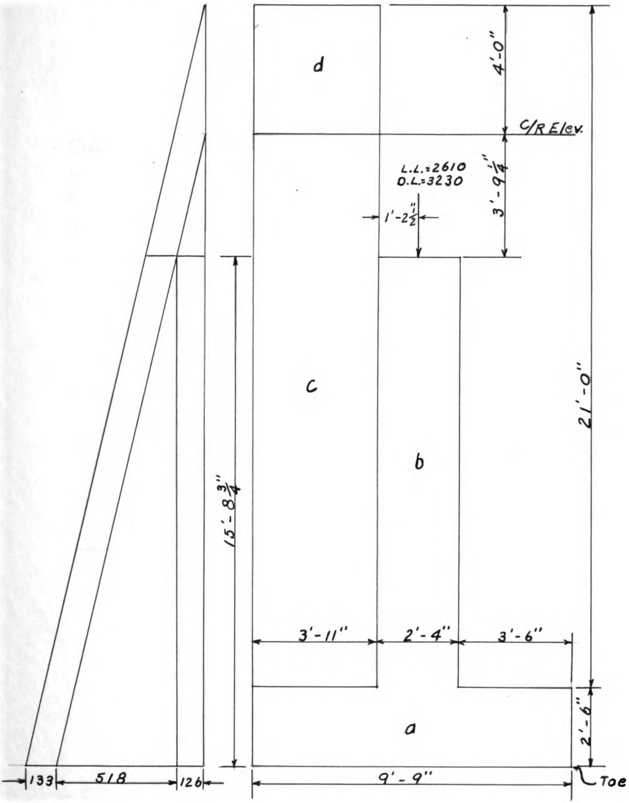
$\frac{218,450}{67.58}$ = 3230# / Lin. Ft. D.L.

Live Load Superstr. Reaction on Abut.

640 x $\frac{47.58}{2}$ + 40,000) 4 x .8 = 176,000#

176,000 x $\frac{1}{67.58}$ = 2610 #/1

Abutment Design:



Load	Area		Unit Wt.	Total wt.	Arm	Mon Ft# about toe
a	9.75 x 2.5	24.4	150	3670	4.88	17,850
b	2.33 x 13.23	30.8	150	4620	4.67	21,600
c	17 x 3.92	66.6	100	6660	7.79	51,900
	Totals Case I		W 1	14950	M1	91,350
	Deck D. L.			3230	4.62	14,900
	Totals Case IV		W IV	18180	MIV	106,250
d	4 x 3.92	15.7	100	1570	7.79	12,200
	Totals Case II		W II	19750	M II	118,450
	Totals Case IV		W IV	18180	M IV	106,250
	Deck L. L.			2610	4.62	12,100
	Totals Case III		W III	20790	M III	118,350

Overturning Moments:

Case I and III

$126 \times 15.73 \times 7.87 = 15{,}600\#$

$518 \times (15.73 \times .5) \times \frac{15.73}{3} = \frac{21{,}400\#}{37{,}000\#}$

Mo_1 and Mo_{III}

Case II

$133 \times 15.73 \times 7.87 = \underline{16{,}500\#}$

Mo II 53,500#

Earth Thrust

$P \quad \frac{259\ 777}{2} \times 15.73 = 8150\#$

Case I No superstr. Load or L.L. Surcharge.

$M\ I = 91{,}350$

$M\ I = \underline{37{,}000}$

$54{,}350\#$

$e = 4.87 - 3.63 = 1.24$

$s = \frac{P}{A} = \frac{14{,}950}{9.75} = 1535$

$Arm = \frac{54{,}350}{14{,}950} = 3.63$

$\frac{Mc}{I} = \frac{6We}{l^2} = \frac{6 \times 14950 \times 1.24}{9.75 \times 9.75} = 1170$

Min
$Arm = \frac{9.75}{3} = 3.25$

Toe pressure 2700#

Heel 365

Case II Superstr. D.L. and L. L. Surch.

$M \quad II = 118,450$

$M \quad II = \underline{53,500}$
$\quad 64,950$

$\text{Arm} = \frac{64,950}{19,750} = 3.28$

Min. Arm = 3.25

$e = 4.87 - 3.28 = 1.59$

$\frac{P}{A} = \frac{19750}{9.75} = 2120\#$

$\frac{Mc}{I} = \frac{6 \times 19750 \times 1.59}{9.75 \times 9.75} = 1980\#$

Toe Pressure = 4100#

Heel " = 140#

Sliding $\frac{8150}{19750}$ = .41

Case III Superstr. D. L. and L. L. No surcharge:

$M \; s \; III = 118,350$

$M \quad III = \underline{37,000}$
$\quad 81,350$

$\text{Arm} = \frac{81,350}{20,790} = 3.92$

Min Arm = 3.25

$e = 4.87 - 392 = 0.95$

$\frac{P}{A} = \frac{20,790}{9.75} = 2130\#$

$\frac{Mc}{I} = \frac{6 \times 20790 \times 0.95}{9.75 \times 9.75} = 1245\#$

Toe Pressure 3375#

Heel 885#

2) 4260

2130# ave. ft.

Case IV Superstr. D. L. (*NO* L.L. or Horiz. thrust)

MsIV $\frac{106,250}{18,180} = 5.85$

Max arm 2/3 x 9.75 = 6.5'

$e = 5.85 - 4.87 = 0.98$

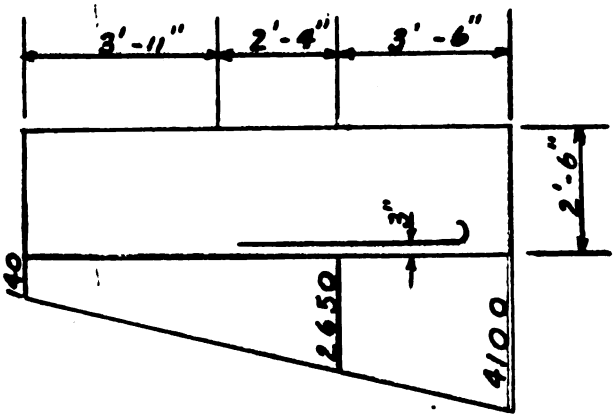
$\frac{P}{A} = \frac{18,180}{9.75} = 1860\#$

$\frac{Mc}{I} = \frac{6 \times 18\ 180 \times 0.98}{9.75 \times 9.75} = 1,125$

Heel pressure 2985# /□' Toe pressure 735# /□'.

Case IV assumes that the earth exerts no horizontal force against the abutment wall.

Footing Steel Case 11



Moments

Earth up $= 2650 \times 3.5 \times \frac{3.5}{2}$

$+1420 \times \frac{3.5}{2} \times 3.5 \times \frac{2}{3}$

$= 16{,}200 + 5{,}800 = 22{,}000'\#$

Conc. down

$2.5 \times 3.5 \times 150 \times \frac{3.5}{2} = \frac{2{,}300}{19{,}700}$

Net Up

$\frac{7}{8}"$ @ 12" : .60 □" $\Sigma_o = 2.75$

$As = \frac{19{,}700 \times 12}{18{,}000 \times .875 \times 27} = .560$ □" req'd.

$P = \frac{As}{bd} = \frac{0.60}{12 \times 27} = .00185$

$k = \sqrt{2Pn\ (Pn)^2} - Pn = \sqrt{.037\ (.0185)^2} - .0185$

$k = 0.174 \qquad j = 1 - \frac{.174}{3} = 0.942$

$fs = \frac{M}{As\,jd} = \frac{19{,}700 \times 12}{.60 \times .942 \times 27} = 15{,}500\# /$□"

Use $\frac{7}{8}"\ \phi$ at 12" run bond length

beyond face of wall.

$fc = \frac{2M}{Kb_jd^2} = \frac{19{,}700 \times 12 \times 2}{0.174 \times 12 \times .942 \times 27^2} = 330$ /□"

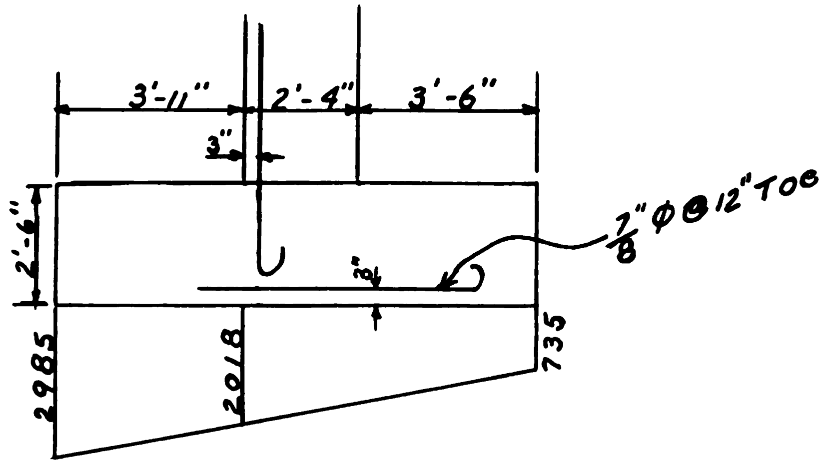
$V = \frac{2650 + 4100}{2} \times 3.5 = 11{,}760 \qquad v = \frac{10{,}450}{12 \times .942 \times 27} = 34.2\#/$□"

$-3.5 \times 2.5 \times 150 = \quad \underline{1{,}310}$

$V = 10{,}450$

$\mu = \frac{V}{jd} = \frac{10{,}450}{2.75 \times .942 \times 27} = 150\#/$□" o.k. (provide use special anchorage)

Heel Steel Case IV



Σ M about dowel steel

Earth up $(2018 \times 4.17 \times \frac{4.17}{2}) + (962 \times \frac{4.17}{2} \times \frac{4.17}{3} \times 2)$

$= 17,680 + 5570 = 23,250$

Earth Down

$3.92 \times 17.0 \times 100 \times 2.08 = 13,850\#$

Conc. Down

$4.17 \times 2.5 \times 150 \times 2.08 = \underline{3250\#}$

Mom. Down 17100#

Net Mon up = 6,150#

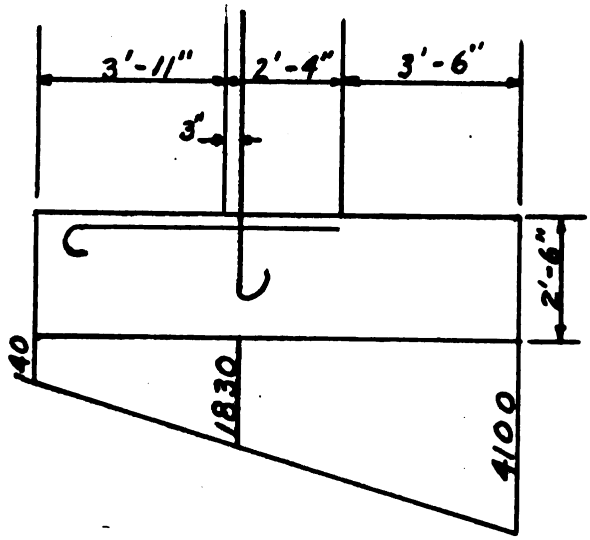
$As = \frac{M}{fsjd} = \frac{6150 \times 12}{18,000 \times .875 \times 27} = .174$ " req'd

By running every other toe far into heel.

.30 " furnished.

Heel Steel Top

Case II



ΣM about dowels

Earth down = (17x3.92x100x2.08) + (9 x 3.92 x 100 x 2.08)

= 17,160

Conc. down 2.5 x 4.17 x 150 x 2.08 = 3,260

20,420#

Earth up $(140 \times 4.17 \times \frac{4.17}{2}) + (1690 \times \frac{4.17}{2} \times \frac{4.17}{3}) =$ 6030

Net Mom down. = 14,390

$A_s = \frac{M}{f_s jd} = \frac{14390 \times 12}{18000 \times .875 \times 27} = .41$ " req'd

$\frac{3}{4}$" ϕ @ 12" $A_s = .44$ $\Sigma o = 2.36$ $p = \frac{.44}{12 \times 27} = .00136$

$K_s = \sqrt{2pn \quad (pn)^2} - pn = \sqrt{.0272 \; (.000185)} - .0136 = .152$

$j = 1 - \frac{K}{3} = 1 - \frac{.152}{3} = .949$

$f_s = \frac{M}{A_s jd} = \frac{14390 \times 12}{.44 \times .949 \times 27} = 15300\#/\square$"

$f_c = \frac{2M}{Kjbd^2} = \frac{2 \times 14390 \times 12}{.152 \times .949 \times 12 \times 27 \times 27} =$ 274# /□"

$$V = (3.92 \times 21 \times 100) + (4.17 \times 2.5 \times 150) = 9815\#$$

$$-(140 \times 4.18) + \left(1690 \times \frac{4.18}{2}\right) = \underline{4065\#}$$

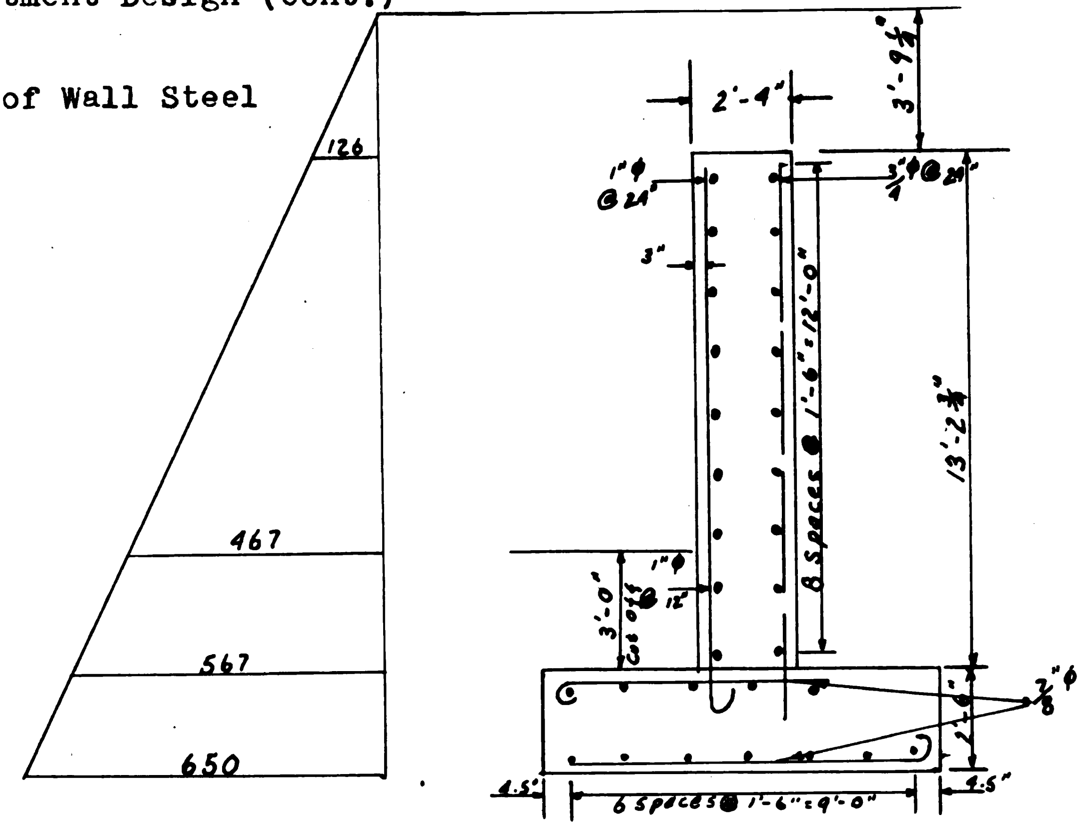
$$5750\#$$

$$v = \frac{V}{bjd} = \frac{5750}{12 \times .949 \times 27} = 18.7\# /\square''$$

$$u = \frac{V}{\Sigma_o jd} = \frac{5750}{2.36 \times .949 \times 27} = 95 \#/\square''$$

Abutment Design (cont.)

Design of Wall Steel



Moment

$$126 \times 13.23 \times \frac{13.23}{2} + 441 \times \frac{13.23}{2} \times \frac{13.23}{3}$$

$$11{,}000 + 12{,}900 = 23{,}900 \ \#'$$

$$A_s = \frac{23{,}900 \times 12}{18{,}000 \times .875 \times 25} = 0.73 \text{ " required}$$

Use 1" ϕ @ 12" area = 0.78

$\Sigma o = 3.14$

$$P = \frac{0.78}{12 \times 25} = .0026$$

$$k = \sqrt{.52\,(.026)^2} - .026 = .228 - .026 = 0.202$$

$$j = 1 - \frac{.202}{3} = 0.933$$

$$f_s = \frac{23{,}900 \times 12}{0.78 \times 0.933 \times 25} = 15{,}700 \#/\square\text{"} \quad \text{o.k.}$$

Abutment Design (cont.)

Design of Wall Steel:

$$fc = \frac{23,900 \times 12 \times 2}{.202 \times 0.933 \times 12} \quad \frac{2}{x25} = 406\#/\square'' \text{ o.k.}$$

$$V = \frac{126 + 567}{2} \times 13.123 = 4580\#$$

$$V = \frac{V}{bjd} = \frac{4580}{12x.933x25} = 16.4 \#/\square'' \text{ o.k.}$$

$$U = \frac{4580}{3.14 \times .0 \; .933x25} = 63 \#/\square'' \quad \text{o.k.}$$

Cut-off of Wall Steel

Trial # 1 6' above top of Ftg.

$$P = 567 - 6 \times 33\ 1/3 = 367$$

$$M = 126 \times 7.23 \times \frac{7.23}{2} + 241 \times \frac{7.23}{2} \times \frac{7.23}{3}$$

$$3300 + 2100 = 5400\#'$$

Trial # 2 @ 3' above top of ftg.

$$P = 567 - 3 \times 33\ 1/3 = 467$$

$$M = 126 \times 10.23 \times \frac{10.23}{2} + 341 \times \frac{10.23}{2} \times \frac{10.23}{3}$$

$$6600 + 6,000 = 12,600\#$$

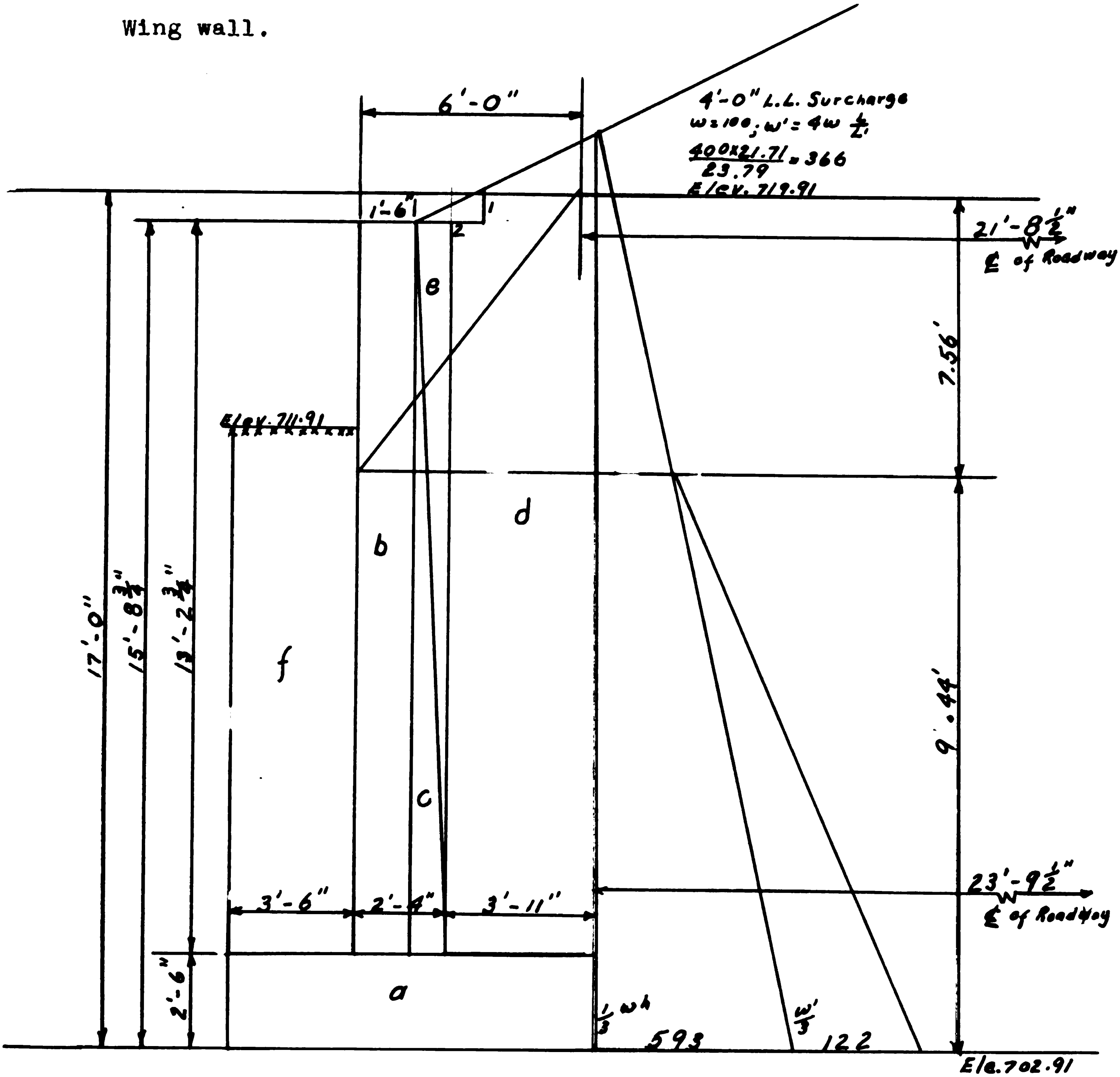
$$As = \frac{12,600 \times 12}{18,000 \; x.875x25} = 0.384 \square''$$

Furnished

0.390 if every 2nd bar is cut off @ 3' ½ bond or @ 4' -6" above footing.

Abutment design:

Wing wall.



Abutment Design:

Stabilization:

Load		Total Wt.		Mom. ent.
(a)	2.5 x 9.75 x 150	3670#	x 4.88	= 17,850' #
(b)	1.5 x 13.23 x 150	2980#	x 4.25	= 12,680' #
(c)	13.23 x .833 x 150 x $\frac{1}{2}$	826#	x 5.28	= 4,360' #
(d)	14.23 x 3.92 x 100	5590#	x 7.83	= 43,500' #
(e)	13.23 x .833 x 100 x $\frac{1}{2}$	552#	x 5.55	= 3,060' #
(f)	3.5 x 6.5 x 100	2275#	x 1.75	= 3,980' #
				85,430

W = 15,893# M(SI,II') M = 85,430'#

Overturning:

593 x 17 x $\frac{1}{2}$ 4950# x 5.67' = 28,000'# M(OI)

122 x 9.44 x $\frac{1}{2}$ 576# x 3.15' = 18,130'# M(OII)

Abutment Design:

Wing Wall Stability

Case I' No live load surcharge or superstr. L.

M (SI') = 85,430'#

M (OI') = 28,000'#

MI' = 57,430'# $R(I') = \frac{57,430'\#}{15,893\#} = 3.62'$ o.k.

Min arm $= \frac{9.75'}{3} = 3.25'$

Case II' Superstructure D. L. L.L. Surcharge.

$(M_s\ II)$ = 85,430' #

M_o II = 18,130' #

M II' = 67,300' #

$(R\ II') = \frac{67,300'\#}{15,893\#} = 4.24'$ o.k.

Min arm $= \frac{9.75'}{3} = 3.25'$

Case IV Superstructure D. L. (No L.L. or Horiz. thrust)

M = 85,430'#

W = 15,893 #

$R\ IV' = \frac{85,430'\#}{15,893\#} = 5.38'$ o.k.

Min arm $= \frac{9.75'}{3} = 3.25'$

Wing Wall Reinforcing Steel:

The main wall takes more loads on it than the wing wall does and the loads on the main wall are heavier than the loads on the wing wall. By inspecting the steel erection diagram it shows that the same steel is used for both the main wall and the wing wall, therefore, the stresses in the wing wall are below the allowable. It is therefore, not necessary to check the steel in the wing wall.

CONCLUSION

Remarks have been placed in their related sections as the checks have been performed on the design of this bridge. The Bridge Division of the Michigan State Highway accounts for the fact that such items as curbs, posts, railings and the concrete slab are overdesigned because it is cheaper to overdesign such items than it is to draw up plans for each individual bridge. The diaphragms are used to keep the bridge girders level and to keep them from twisting.

On the whole the structure is adequately designed and the standard items such as posts, etc., mentioned above are overdesigned.

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[redacted]