AN ANALYSIS OF A ROLLED BEAM,
DECK TYPE HIGHWAY BRIDGE

Thesis for the Degree of B. S.
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T. S. Katalenich
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An Analysis of a Rolled Beam, Deck Type Highway Bridge

A Thesis Submitted to

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by

T. S. Katalenich

Candidate for the Degree of

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INTRODUCTION

The bridge selected for analysis is located 1.8 miles northwest of Reed City, Michigan, on U.S.10. This structure is a single span, rolled beam, deck type highway bridge. It was designed for H-20 loading and width of roadway to be of sufficient width to accommodate three lanes of traffic, although a two lane pavement exists at the present time.

The design of this structure was governed by, "Specifications for the Design of Highway Bridges" adopted by Michigan State Highway Department November, 1936. As in the design of most structures, there are a few instances where experience and practical application of construction methods has permitted a deviation from strict application of the specifications. Such instances will be made a note of in the analysis.

The bridge provides a crossing of Johnson Creek, a stream which drains a basin of approximately 12 sq. miles. The basin top soil is composed of a very sandy loam with a low runoff coefficient.

A_s

b

d'

d

f

e'

f_c

f'

f_s

I

j

k

K

L

M

M

N

SYMBOLS and NOTATION

As area of tensile reinforcement

b width of rectangular beam

d' distance from extreme fiber to compressive resultant

d effective depth of flexural members

e eccentricity

e' eccentricity measured from gravity axis.

f_c compressive stress in extreme fiber

f'_c ultimate compressive strength of concrete

f_s stress in tensile reinforcement

I moment of inertia

j ratio of distance (jd)

k ratio of distance between extreme fiber & N.A. to eff. d

K $\frac{1}{2}f_c jk$ (Reinforced Concrete--Sutherland & Reese, p. 520)

L length of span; length of anchorage of reforcement bars

M external moment

n ratio of modulus of elasticity of steel to concrete

N.A. neutral axis

sum of perimeters of bars

P external concentrated load

s spacing of reinforcement bars

u bond stress

v shearing stress

V total shear

w permissible soil pressure; uniformly distributed load

y distance of moment arm of a horizontal force resultant

x distance of moment arm of a vertical force resultant

ANALYSIS of CHANNEL OPENING

Specifications:

Art. 11: In general, the waterway provided shall be sufficient to insure the discharge of flood waters without undue backwater head and at a velocity which will not increase the erosive action of the stream to such an extent as to endanger the structure, or cause damages to upstream property.

Art. 13: Bridges proposed over streams where future drain projects are a probability shall have footings at a sufficient depth estimated to prevent instability or undermining when the drain project is carried out.

Art. 14: Natural obstructions, such as islands, rocks, trees and brush which retard or deflect the stream in the vicinity of a bridge shall be removed and such portions of the channel shall be cleaned out as are necessary to straighten out the stream at the bridge and prevent eddies or scour.

Art. 15: The clear width of all openings and the clear vertical distance between the superstructure and the flood water elevation shall be sufficient for the passage, without damage to the structure, of ice floes and of the largest drift or debris which may be expected. Ordinarily a minimum of one foot vertical clearance shall be provided above extreme high water.

A. Computation of the Cross-Section needed:

Area of basin = 12 sq. mi.

Coefficient taken from Highway Map .25

Talberts Formula:

$a = 127\ C \sqrt{A_m^{3}}$ where;

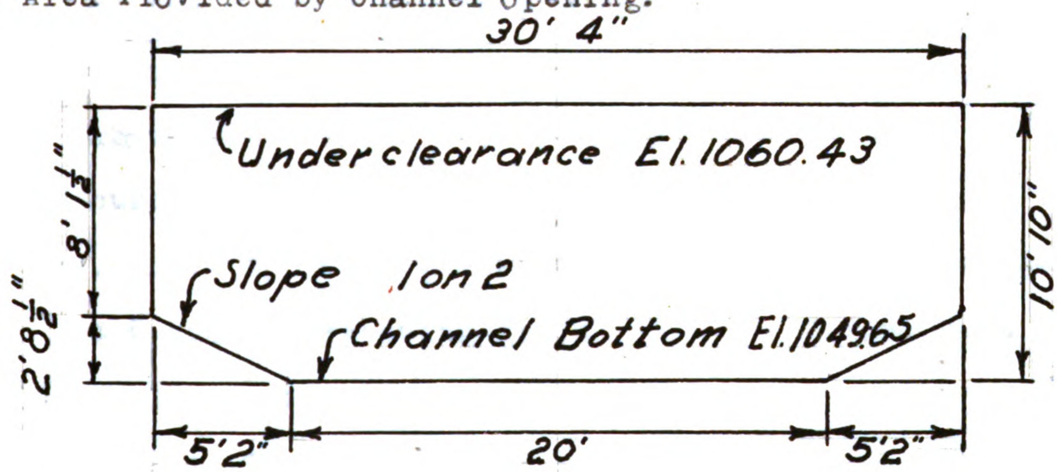
a = area in sq. ft.

C = Coefficient taken from weather map.

A_m = Drainage Basin in square miles.

$a = 127\ (.25) \sqrt{12^{3}} = 203$ sq.ft. needed

B. Area Provided by Channel Opening:



1. To find x: Fig. I

$x = \tan. 26° 30' (5.17') = .498 (5.17') = 2.67'$

2. Area provided:

$A = \frac{(20 + 30.33)}{2}(2.67) + 30.33\ (7.11) = 283.2$ sq.Ft.

It is obvious that sufficient channel area has been provided as flood areas and other drainage facilities in the area have not been discounted from the 203 sq.ft. needed. The clear width of 30' 4'' provides sufficient width for passage of largest drift or debris to be expected. Abutment footing is 4.2' below channel bottom which is sufficient.

Spec

Ar

A

A

A

SLAB ANALYSIS

Specifications:

Art. 18: In no case shall the roadway on a bridge be made less in width than that provided for traffic on the bridge approaches and preferably the width of roadway should be not less than the following:

Two lane highway (pavement)	24 ft.
Three lane highway	33 ft.

Art. 20: Substantial curbs shall be built on each side of the roadway and they shall have a width not less than 6 in. and a height of not less than 9 in. measured above the wearing surface at a point adjacent to the curb.

Art. 22: Approach grades shall preferably not exceed 4 % and intersecting grades shall be joined by a parabolic vertical curve.

Art. 24: The roadway shall be crowned to fit the approach pavement and sloped to provide effective drainage of the roadway surface.

Suitable gutters or provisions for drainage shall be constructed at the ends of the approaches adjacent to the bridge so as to prevent erosion of approach shoulders and fills by water draining off the ends of the structure.

Bridge roadway surfaces of 40 ft. width or less shall be constructed with transverse parabolic crown of height not less than given by the following formula:

$C = 0.00187\ R^2$ where:

C = Crown of roadway in inches.

R = Width of roadway in feet.

The minimum crown shall be three-fourths ($\frac{3}{4}$) inch.

Art. 25: Substantial railings shall be provided along each side of the bridge for the protection of traffic. The top of railing shall be not less then 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 eight (8) inches. Ample provision shall be made for inequality in the ratio of movement of the railing and the supporting superstructure, due to temperature or erection conditions.

Art. 26: Where no separate wearing surface is provided on concrete floor slabs, an additional slab thickness of one-half inch shall be provided. This one-half inch thickness on the top of the slab shall be disregarded in computing strength of floor slabs.

Art. 29: The dead load shall consist of the actual weight of all materials and construction comprising the completed design.

The following weights of materials may be used in estimating dead load:

Steel	490 lb./cu. ft.
Concrete, plain or reinforced	150 lb./cu. ft.
Loose sand and earth	100 lb./cu. ft.

Art. 30: 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.

Art. 31: The live load on roadways shall consist of a train of standard motor trucks, in each traffic lane, as pictured in Figure 2.

The truck trains shall be assumed to occupy traffic lanes, each having a width of 9 feet corresponding to the standard truck clearance width. Within the curb to curb width of the roadway, the traffic lanes shall be assumed to occupy any position which will produce the maximum stress, but which will not involve overlapping of adjacent lanes, nor place the center of the lane nearer than 4 feet 6 inches to the roadway face of the curb.

All trucks in the same, or adjacent, traffic lanes shall be assumed headed in the same direction.

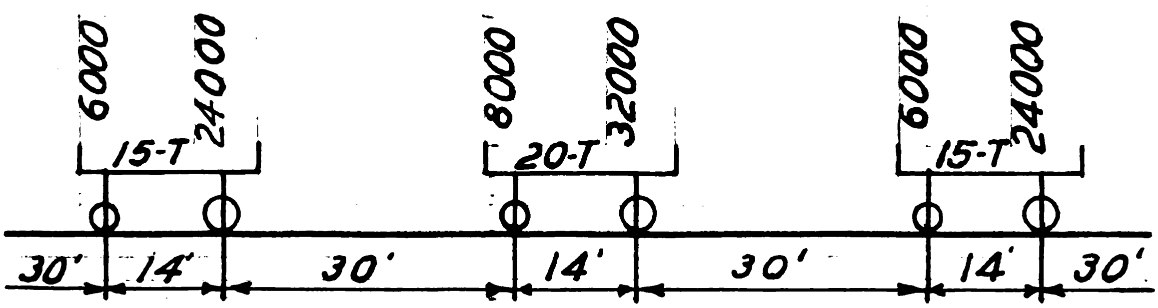


Fig. 2 Roadway Loads

Art. 32: The Standard Motor Truck, as specified above, is a motor truck with dimensions and weight distribution as shown in Fig. 3.

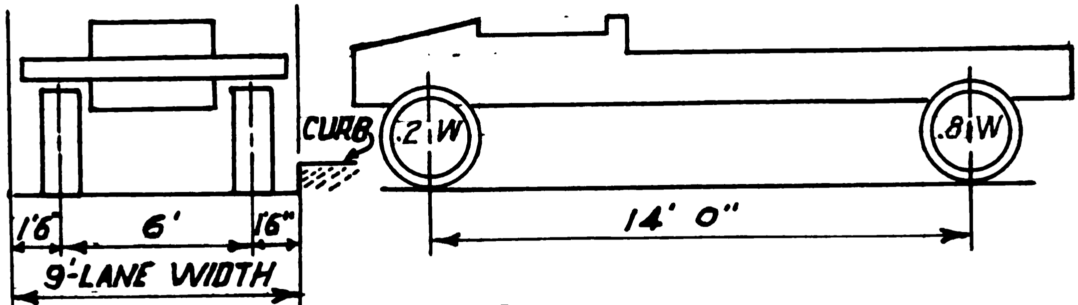


Fig. 3

Art. 33. When roadway provides for two lanes of traffic or less, the design shall provide for the maximum load that can be placed simultaneously in all traffic lanes.

When provision is made for three lanes of traffic, the design shall provide for 90% of the simultaneous maximum loading of all lanes.

Art. 35. 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 500 pounds per lineal foot of rail.

Art. 36. Curbs shall be designed to resist a force of not less than 500 pounds per lineal foot of curb applied at the rop of the curb.

Art. 37: All live load stresses shall be increased by an allowance to provide for dynamic effect. The impact allowance shall be determined by multiplying the maximum live load stress by the coefficient determined by the following formula:

$$I = \frac{L + 20}{6L + 20} \quad \text{where}$$

I = Impact Coefficient

L = Span Length

Art. 54 General Assumptions in concrete Structure Design:

a. Calculations are made with reference to unit working stresses and safe loads, as elsewhere specified herein, rather than with reference to ultimate strength and ultimate loads.

b. A section plane before bending remains plane after bending.

c. The modulus of elasticity of concrete in compressions is constant within the limits of working stresses; distribution of compressive stress in flexure is therefore rectilinear.

d. The ratio "n" of the modulus of elasticity of the steel to that of the concrete shall be taken as follows (applies also to compression members)

$$n = \frac{E_s}{1000\, f'_c} = \frac{30,000}{f'_c}$$

e. Concrete shall be assumed as offering no tensile resistance.

f. The bond between concrete and metal reinforcement is assumed to remain unbroken throughout the range of working stresses.

g. Initial stress in the reinforcement due to contraction or expansion of the concrete is neglected.

h. Nomenclature and formulae for design shall be those common usage as given in the Standard Specifications for Concrete and Reinforced Concrete of the Joint Committee.

Art. 58. The maximum spacing of principal reinforcement in walls and slabs shall not exceed two times the thickness of the slab or wall, nor more than 24 inches.

Art. 59. 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.

Art. 61. The length of bar added for anchorage may be either straight or bent. The inside radius of bend shall not be less than four diameters of the bar. A standard hook as referred to in these specifications shall consist of a semi-circular bend, whose inside radius is four bar diameters, and a final tangent length of four bar diameters.

Art. 64. Where reinforcement is used to resist tensile stresses developed by beam action, the bond stress shall be taken not less than that computed by the following formula:

$$u = \frac{8V}{7\sum o \, d}$$

Art. 66. Horizontal reinforcement for shrinkage and temperature stresses normal to the principal reinforcement shall be provided where the principal reinforcement extends in one direction only. Such reinforcement shall be placed at exposed surfaces and shall provide not less than one eighth square inch of reinforcement per foot of width of surface.

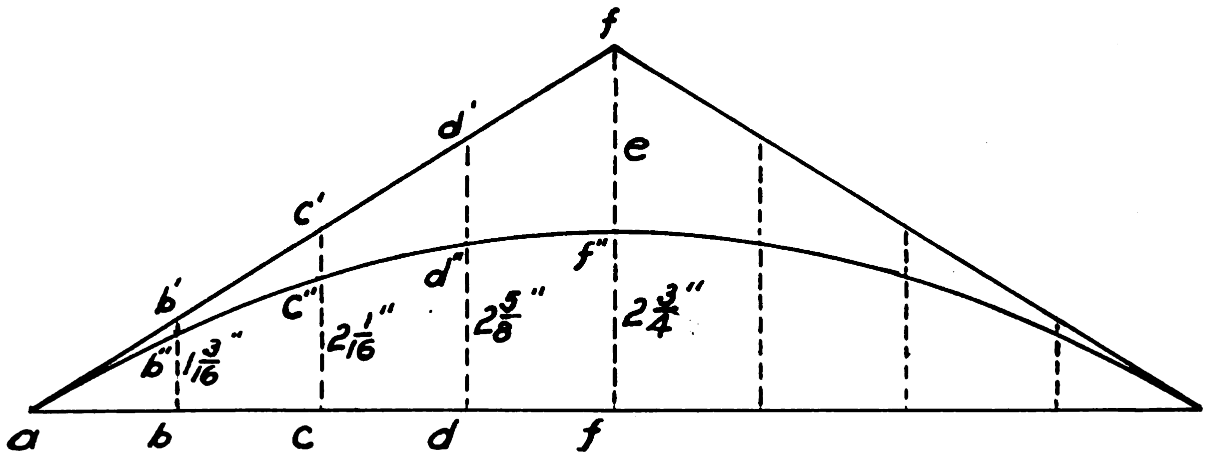
Crown Analysis: (Spec. Art. 24.)

Width of roadway 38 Ft.

$C = 0.00187 \; R^2$

$C = 0.00187 \, (38)^2 = 2.7$ in.

$C = 2\frac{3}{4}" = e$ (Parabolic Curve)



Crossection of Road Crown

Fig. 4

Computations: (quarter-sections of a right triangle)

$ff' = 2e$

$dd' = \left(\frac{3}{4}\right) 2e = \frac{3e}{2}$

$cc' = \left(\frac{1}{2}\right) 2e = e$

$bb' = \left(\frac{1}{4}\right) 2e = \frac{e}{2}$

$f'f" = e$

$d'd" = \left(\frac{3}{4}\right)^2 e = \frac{9e}{16}$

$c'c" = \left(\frac{1}{2}\right)^2 e = \frac{e}{4}$

$b'b" = \left(\frac{1}{4}\right)^2 e = \frac{e}{16}$

Crown Dimensions:

$$bb" = \frac{e}{2} - \frac{e}{16} = \frac{7e}{16} = \frac{(2.75)7}{16} = 1\frac{3}{16}"$$

$$cc" = e - \frac{e}{4} = \frac{3e}{4} = \frac{(2.75)3}{4} = 2\frac{1}{16}"$$

$$dd" = \frac{3e}{2} - \frac{9e}{16} = \frac{15}{16} e = \frac{(2.75)15}{16} = 2\frac{5}{8}"$$

Veiw Showing Roadway Width and Slab Support:

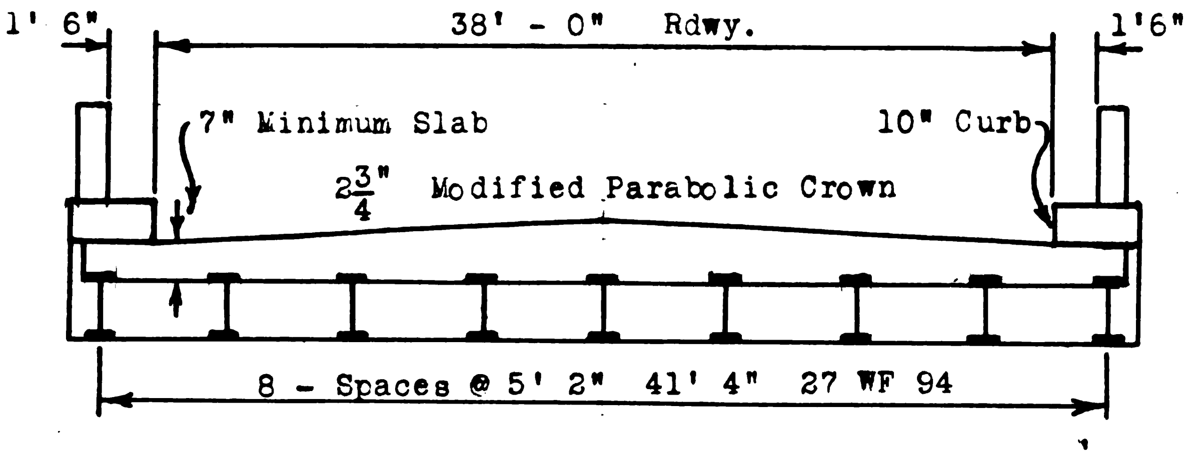


Fig. 5

Maximum Loading on Slab occurs when rear wheel is over point midway between stringers:

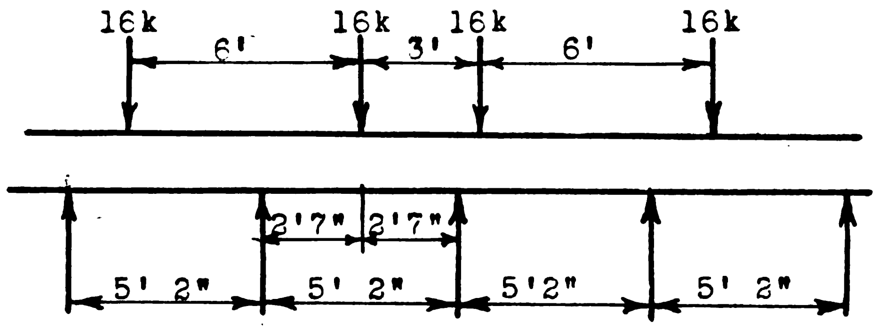


Fig. 6

A. Effective width: Spec. 41: Wheel concentration shall be assumed as uniformly distributed on a line perpendicular to the main reinforcement over an effective width not more than the value given by the formula:

$B = 0.7S + 2$ where:

B Effective width of slab in feet.

S Span of slab, center to center of supports.

$B = 0.7\ (5.17') + 2 = 5.6'$

B. Maximum Dead Load Moment:

1. Dead Load per foot of slab:

Depth of slab

design depth	7.00''
crown	2.75
camber	.19
wearing surface	.50
total depth	10.44''

$D.L. = \frac{10.44}{12}\ (1)\ (1)\ (150) =$ 131.0 lb. per ln. ft.

Extra allowance 20.0

Total D.L. 151.0 lb. per ln. ft.

2. Maximum D. L. Moment:

$Max.\ Mo. = \frac{1}{12} wl = \frac{1}{12}\ (151)\ (5.17)\ (12) = 4000$ lb. in.

C. Mamimum Live Load Moment:

H-20 loading used in analysis. The beam is regarded as a continuous loaded beam and maximum positive and negative moments are computed using formulae in Kirkham's "Highway Bridges" page 74.

1. Max. positive moment:

$$M = \frac{1}{5} Pl = \frac{1}{5} (16000)(5.17)(12) = 198,000 \text{ lb. in.}$$

2. Max. negative moment:

$$M = \frac{1}{6} Pl = \frac{1}{6} (16,000)(5.17)(12) = 165,000 \text{ lb. in.}$$

Maximum is due to positive moment. Inasmuch as the steel in the slab is the same in top and bottom, we will consider analysis for positive moment only.

3. Max. L.L. moment for 1' width of slab:

$$\text{Max. Mo.} = \frac{198,000}{5.6'} = 35,400 \text{ lb. in.}$$

4. Impact:

$$I = \frac{L + 20}{6L + 20} = \frac{32.58 + 20}{6(32.58) + 20} = .24$$

Impact = 35,4000 (0.24) = 8600 lb.in.

5. Total Moment:

Live Load	35,400 lb. in.
Impact	8,600
Dead Load	4,000
Total Moment	48,000 lb. in.

D. Steel needed:

d= 7 - 1.5 = 5.5 ''

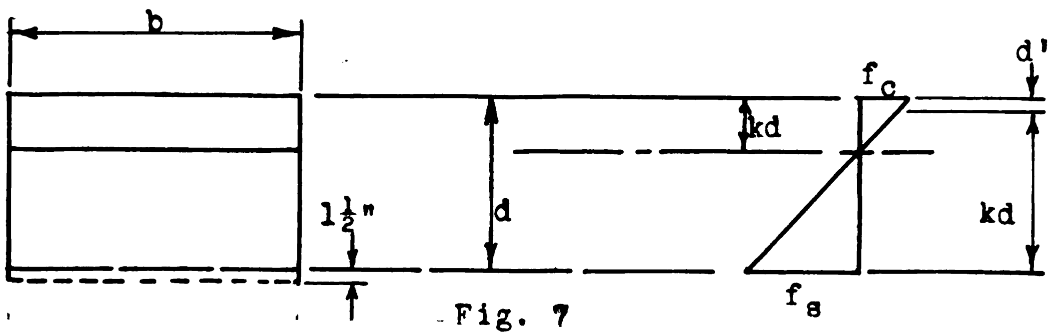


Fig. 7

1. To find jd :

$$x = \frac{1200\ (5.5)}{3000} = 2.2''$$

$$z = \frac{2.2}{3} = .73''$$

$$jd = d - z = 5.5 - .73 = 4.77''$$

2. To find A_s :

$$A_s = \frac{M}{f_s\ (jd)} = \frac{48{,}000}{18{,}000\ (4.77)} = .56 \text{ sq. in. per ft.}$$

E. Steel provided: (per foot of concrete)

$\frac{5}{8}''$ round bars spaced at 6''.

$$A_s = .307 \text{ sq. in. } (2) = .62 \text{ sq. in.}$$

F. Check stress:

$$f_c = \frac{2M}{(b)\ (jd)\ (kd)} = \frac{(2)\ 48{,}000}{12\ (4.77)\ (2.2)} = 775 \text{ lb. per sq. in.}$$
less than 1200 p.s.i.

$$f_s = \frac{M}{A_s\ (jd)} = \frac{48{,}000}{.62\ (4.77)} = 16{,}500 \text{ lb. per sq. in.}$$
less than 18,000 p.s.i.

G. Check Shear:

Formula for maximum shear from Steel Construction Manual of AISC 5th edition 1947 page 376.

$$V = \frac{5}{8}\ w\ l + \frac{11}{16}\ P = \frac{5}{8}\ (151)\ (5.17) + \frac{11}{16}\ \frac{(16{,}000)}{5.17}$$

$$V = 487.5 + 2130.0 = 2617.5 \text{ lbs.}$$

$$v = \frac{V}{b\ (jd)} = \frac{2618}{12\ (4.77)} = 46.2 \text{ p.s.i.}$$
less than 60 p.s.i.

$$u = \frac{8\ V}{7 \Sigma_o d} = \frac{8\ (2618)}{7\ (2)\ (5)\ (5.5)} = 138 \text{ p.s.i.}$$
less than 150 p.s.i.

H. Temperature steel provided normal to principal reinforcement:

½ in. circular bars equally spaced at $\frac{(5.17')}{3}$

$As = \frac{1.963}{1.72} = .114$ sq. in. per ln. ft.(top)

$As = \frac{1.963}{1.29} = .157$ Sq. in. per ln. ft.(bottom)

As required Spec. art. 66;

.125 sq. in. per ln. ft.

Temperature steel in top of slab is .011 sq. in. less than required. Since it is desired to maintain one bar directly over a stringer, it was necessary to space the bars as they are. This deficiency is so small that it is negligible. Temperature steel in bottom of slab is more than sufficient. Maximum spacing is 20 in. which is within specifications.

STRINGER ANALYSIS

Specifications:

Art. 43. 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.

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

C = Coefficient based on type of floor.

M'= Bending moment on one beam or stringer.

$$M' = C\left(\frac{M}{N}\right)$$

Value of C is one for reinforced concrete slab.

In determining the end shear on longitudinal beams or stringers, the floor slab or flooring shall be assumed to act as a simply supported beam.

Art. 47. Structural steel:

Axial tension (net section)	18,000 p.s.i.
Tension on extreme fibre in flexure	18,000 ''
Shear on power-driven rivets and pins	13,500 ''
Shear on gross area of webs of beams & girders, $\frac{V}{A}$, where the clear distance between flanges is not more than 60 times the thickness of the web	12,000 ''

Art. 79. Depth ratio of rolled beams not more than 1 /25 of the span.

Art. 80. Rolled beams shall be proportioned by the moments of inertia of their net sections.

Art. 88. The minimum thickness of structural steel shall be 5/16 in. except for gusset plates, fillers and railings. Gusset plates shall in no case be less than 3/8 in. in thickness.

Art. 92. The diameter of rivets in angles carrying calculated stress shall not exceed one-fourth of the width of the leg in which they are driven.

The minimum distance between centers of 3/4 in. rivets shall be not less than $2\frac{1}{2}$ in.

The minimum edge distance for 3/4 in. rivets shall be $1\frac{1}{4}$ in.

Art. 97. All connections shall be proportioned to develop not less than the full strength of the members connected provided that the full strength does not exceed the maximum computed stress by more than 50%, in which case the latter shall govern.

No connection, except for lacing bars and handrails, shall contain less than three rivets. Connections shall be made symmetrical about the axes of the members in so far as practicable.

Art. 110. Provision shall be made for expansion and contraction, to the extent of 1/8 in. for each 10 feet of span. Expansion ends shall be firmly secured against lifting or lateral movement.

Art. 111. Spans of less than 70 feet may be arranged to slide upon metal plates with smooth surfaces.

Art. 117. Superstructure steelwork shall be securely anchored to the substructure. Anchor Bolts shall be not less than one inch in diameter embedded not less than ten inches in the masonry and shall be swedged or threaded to secure a satisfactory grip in the masonry or otherwise suitably anchored.

Art. 124. Diaphragms shall be provided at the third points of all T-Beam spans of forty feet or more.

Art. 127 (i). Sole plates shall each be not less than 3/4 in. thick and not less than the thickness of the flange angles plus 1/8 in. Preferably, they shall not be longer than 18 in.

A. Maximum Dead Load Moment:

1. Dead Load per ln. ft. of stringer

(distance between stringers - 5' 2'')

151 lb. (5.17')	780
wt. per ft. of 27 WF 94 Beam	94
Total dead load	874 lbs. per ft. of stringer

2. $M = \frac{1}{8} w l^2 = \frac{1}{8} (874) (32.67)^2 \cong 117,000$ ft. lb.

B. Maximum Live Loading. (H-20 loading)

1. Maximum loading on stringer (A) will occur when two trucks are side by side with the wheel of one truck directly on the stringer and the respective wheel of the other truck 3' to the side of the first wheel. (shown below)

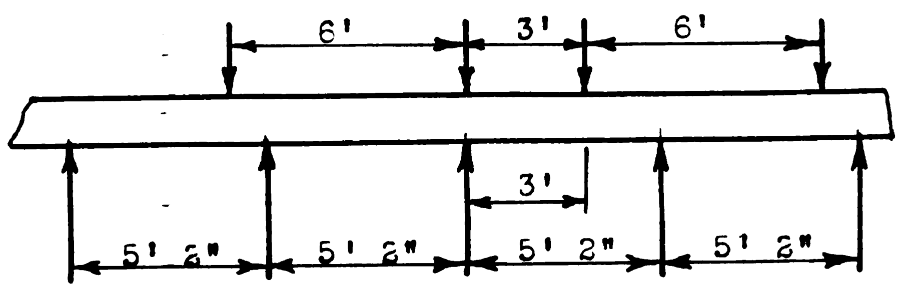


Fig. 7

Maximum load rear axle:

$$R_a = 16,000 + \frac{3\ (16,000)}{5.17} = 16,000 + 6,720 = 22,720 \text{ lbs.}$$

Maximum load front axle:

$$R_a = 4,000 + \frac{3\ (4,000)}{5.17} = 4,000 + 1680 = 5,680 \text{ lbs.}$$

C. **Maximum Live Load Moment:**

Loading to produce maximum live load moment on stringer will occur when rear wheels of the two trucks will simultaneously reach a point the distance $(\frac{a}{2})$ passed the center of the span. (absolute moment)

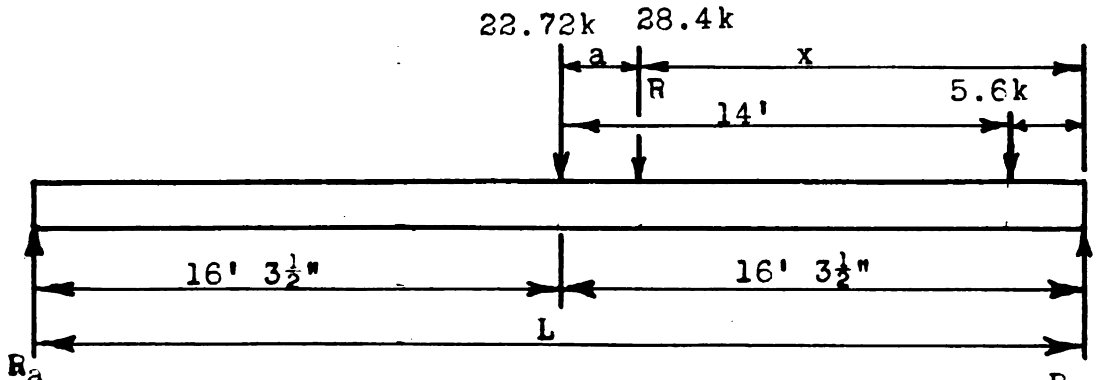


Fig. 8

1. To find R_a and R_b:

$$R_a = \frac{22{,}720\ (16.29') + 5{,}680\ (2.29')}{32.58'} = 11{,}750 \text{ lbs.}$$

$$R_b = 28{,}400 - 11{,}750 = 16{,}650 \text{ lbs.}$$

2. To find x:

$$x = \frac{11{,}750\ (32.58')}{28{,}400} = 13.5'$$

3. To find a:

$$a = 16.29' - 13.5' = 2.79'$$

4. To find Absolute Maximum Moment: (Formula from "Structural Theory" Sutherland & Bowman, 3rd. edition 1944, page 116)

$$M = \frac{R_1}{L}\left[\left(\frac{L}{2}\right)^2 - \frac{L\ (a)}{2} + \left(\frac{a}{2}\right)^2\right] - R_L\ b$$

$$M = \frac{28{,}400}{32.58'}\left[\left(\frac{32.58}{2}\right)^2 - \frac{32.58'\ (2.79')}{2} + \left(\frac{2.79}{2}\right)^2\right] - 0$$

$$M \simeq 196{,}000 \text{ lb. ft.}$$

5. Spec. Art. 43:

$$M' = C \left(\frac{M}{N}\right) = \frac{1}{9}\frac{(135,000 \text{ lb.ft.})}{5.17} = 156,000 \text{ lb.ft.}$$

M' above is much less than the moment computed in 4 above. Therefore I will use the larger moment in this analysis.

D. Beam size:

1. Total moment on stringer:

Dead Load moment	117,000 lb.ft.
Live Load moment	196,000 "
Impact (196,000 x .24)	47,000 "
Total moment	360,000 lb.ft.

2. To find $\frac{I}{C}$ (section modulus)

$$\frac{I}{C} = \frac{M}{s} = \frac{360,000\ (12)}{18,000 \text{ psi}} = 240 \text{ in.}^3$$

27 WF 94 beam has $\frac{I}{C}$ of 242.8 in.^{3} Therefore the beam is suitable to carry the maximum loading on the bridge.

3. Depth Ratio: Spec. Art. 79.

$$\text{Ratio} = \frac{1}{32.58\ (12) \div 27} = \frac{1}{14}$$

The ratio is greater than specifications call for $(\frac{1}{25})$. Therefore the beam is satisfactory for depth.

4: Check Shear: Spec. Art. 47.

27 ÷ .49 = 58 (less than 60 times web thickness)

$$V = 16,650\ (1.24) + \frac{874}{2}\ (32.58) = 34,900 \text{ lbs.}$$

$$s = \frac{V}{A} = \frac{34,900}{.49\ (26.9)} = 2,650 \text{ p.s.i.}$$ (less than 12,000 therefore O.K.)

E. Diaphragms: Provided at 1/3 distance of span.

Spec. Art. 124

Connection angles 4 x 4 x 3/8

Horizontal angles 3 x 3 x 3/8

Diaphragm plate 18 x 3/8

Force resisted by Diaphragms: Spec. Art. 38

Area of substructure $= \frac{9(27)\ (30.33)}{2\ (12)} = 307$ sq.ft.

Area of side elevation $= 5(30.33)(1.5) = \underline{228}$ "

Total area 535 sq.ft.

Lateral force, wind on structure-535(30) = 16,050#

Lateral force, wind on Live Load-150(30.33) = 4,600#

Total force 20,650#

Area steel required:

$$A_s = \frac{20,650}{18,000} = 1.15 \text{ sq. in.}$$

Area steel provided: (2 plates)

$$A_s = 2\ (18)\ (\frac{3}{8}) = 13.5 \text{ sq.in.}$$

This is much more steel area than is necessary.

Stress in rivets:

$$s = \frac{20,650}{2\ (5)\ (.442)} = 4,700 \text{ p.s.i. (allowed is 13,500)}$$

F. Bearing Plates:

Plate size: 12" x ½" x 19½"

Bearing on concrete:

$$f_c = \frac{34,900 \text{ lb.}}{12\ (19.5)} = 149 \text{ p.s.i. (this is very low)}$$

G. The ends of the stringers are not connected but are embedded in concrete a depth of one foot, with two steel bars passing through the web of the stringers. This gives the necessary stability.

ABUTMENT ANALYSIS

Specifications:

Art: 39. 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 non-cohesive granular material, provided that no structure shall be designed for an equivalent fluid pressure of less than 30 pounds per square foot.

To provide for live load an equivalent earth surcharge of four feet shall be applied over all surfaces subject to highway traffic.

Art: 59. In footings and other principal structural members where the concrete is in direct contact with soil, the reinforcement shall have a minimum covering of 3 in. of concrete measured to the center of bar.

Art. 63. (b) In cantilever footings all bars shall be anchored by means of standard hooks at the outer ends of the bar.

In analysis of the Abutment, four types of loadings will be considered as follows:

Case 1: Abutment before stringers are placed and only pressure is due to retained soil without allowance for live loading over fill.

Case 2: Pressure due to retained soil <u>With</u> allowance for live loading over fill and stabilizing moment due to dead load of bridge.

Case 3: Pressure due to retained soil not allowing for live loading surcharge and stabilizing moment due to dead and live loading of bridge.

Case 4: Pressure due to retained soil <u>with</u> allowance for live loading surcharge and stabilizing moment due to dead and live loading of bridge.

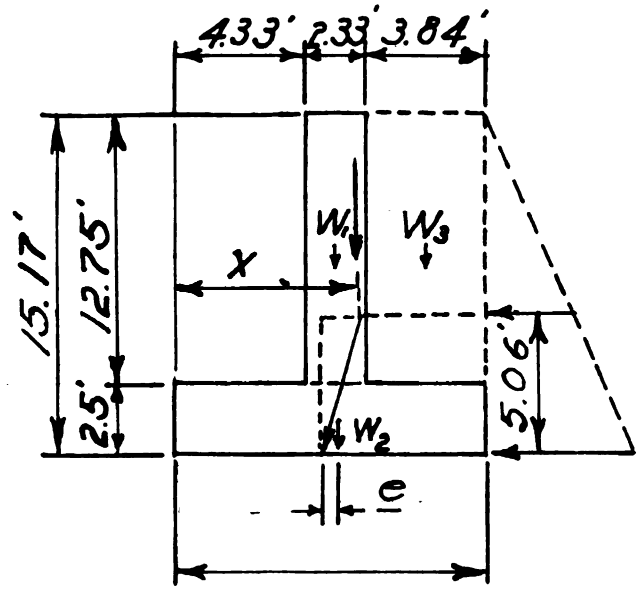
Total Height of Fill:

Surcharge live load	4' 0"		
Surcharge (12" pavement)	1' 6"		
Distance, pavement & abutment	2' 6 $\frac{5}{8}$"		
Total surcharge	8' $\frac{5}{8}$"	(Call it 8')	8' 0"
Height retaining			15' 1$\frac{7}{8}$"
Total height of fill			23' 1$\frac{7}{8}$"

Weight of reinforced concrete	150 lb. per cu.ft.
Weight of fill	100 lb. per cu.ft.
Angle of repose of fill	30 degrees

Case 1

Abutment before stringers are placed and only pressure is due to retained soil without live load surcharge:



$p' = Cwh = .33\ (100)\ 15.17 = 504$ lb.

$P = \frac{504}{2}\ (15.17) = 3820$ lb.

	wt.	arm	moment
$W_1 = 150\ (2.33)\ (12.75') =$	4460 lbs.	5.5 =	24,800 lb.ft.
$W_2 = 150\ (2.50)\ (10.5) =$	3940	5.25 =	20,660 "
$W_3 = 100\ (3.84)\ (12.75) =$	4900	8.58 =	42,000 "
Total wt.	13300 lbs.	Total Mo.	87,460 lb.ft.

To find "x":

$$x = \frac{87,460}{13,300} = 6.57'$$

To find "e"

$$e' = \frac{5.06\ (3820)}{13,300} = 1.46' \qquad 6.57 - 1.46 = 5.11'$$

$$e = 5.25 - 5.11 = .14'$$

Earth Pressure:

$$P = \frac{13,300}{10.5'} \left[1 \pm \frac{6(.14)}{10.5}\right] = 1270\ (1 \pm .08) \doteq 1370\ \#/\text{sq.ft.}$$
$$1170\#/\text{sq.ft.}$$

Overturning Factor:

$$\text{Ratio} = \frac{87,460 \text{ lb.ft.}}{3,820(5.06)} = 4.4 \quad \text{(very safe)}$$

Sliding Factor:

$$\text{Ratio} = \frac{13,300\ (.50)}{3,820} = 1.74 \quad \text{(Just on the borderline of being safe)}$$

It is obvious that the resultant intersects the base within the middle third since "e" is only .14'.

Stem Moment:

$$p = C^{w}h = .33\ (12.75)(100) = 421 \text{ lbs.}$$

$$M = \left(\frac{421}{2}\right)(12.75)\left(\frac{12.75}{3}\right) = 11,400 \text{ lb. ft.}$$

Toe Moment:

$$p = \frac{6.17\ (355)}{10.5} + 1090 = 1299 \text{ lbs. (earth pressure at edge of stem)}$$

Moment upward due to earth pressure:

$$1299\ (4.33)\left(\frac{4.33}{2}\right) = 12,050 \text{ ft. lb.}$$

$$\frac{146}{2}(4.33)\left(\frac{2}{3}\right)(4.33) = 896 \text{ "}$$

Total Moment = 12,946 ft. lb.

Moment downward due to concrete in toe:

$$M = 150\ (4.33)(2.5)\left(\frac{4.33}{2}\right) = 3,470 \text{ ft. lb.}$$

Toe Moment = 12,946 - 3,470 = 9,476 ft. lb.

Heel Moment:

$$p = \frac{3.84\ (355)}{10.5} = 130 \text{ lb./sq.ft. (earth pressure at edge of stem.)}$$

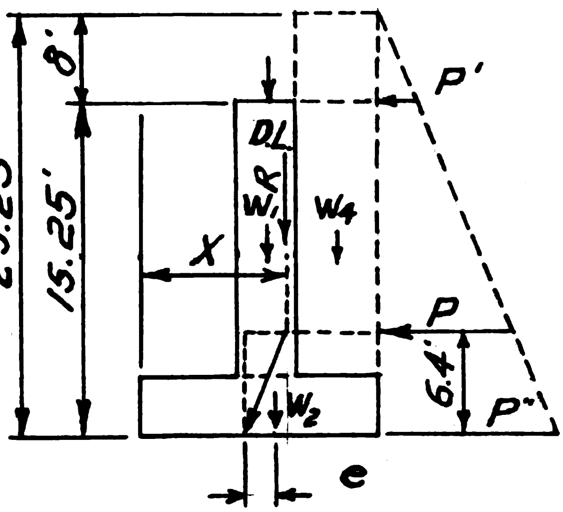
$$\text{Upward } M = 1090(3.84)\left(\frac{3.84}{2}\right) + \frac{130}{2}(3.84)\left(\frac{3.84}{3}\right)2 = 8369 \text{ ft.lb.}$$

$$\text{Downward } M = 3.84(2.5)(150) + 4900 \times \frac{3.84}{2} = 12,180 \text{ ft.lb.}$$

Heel Moment = 12,180 - 8,369 = 3,811 ft. lb.

Case 2

Pressure due to retained soil with allowance for live load surcharge and stabilizing moment due to dead load of bridge.



To find horizontal force:

$P' = Cwh = .33(100)(8) = 264$ lb.

$P'' = CWh = .33(100)23.25 = 768$ lb.

$$264\ (15.25)\frac{(15.25)}{2} = 30{,}700 \text{ ft.lb.}$$

$$\left(\frac{768 - 264}{2}\right)(15.25)\ \left(\frac{15.25}{3}\right) = \underline{19{,}550} \text{ ft.lb.}$$

Total Moment = 50,250 ft.lb.

$$P = 264\ (15.25) + \frac{(768 - 264)}{2}(15.25) = 7870 \text{ lb.}$$

$$y = \frac{50{,}250 \text{ ft.lb.}}{7870 \text{ lb.}} = 6.4'$$

Dead Load of deck per foot of Abutment:

$$D.L. = \frac{874 \text{ lb. } (33.87')}{2\ (5.17')} = 2860 \text{ lbs.}$$

To find Resultant Vertical Force:

D.L.		= 2860 lbs.	5.46'	=	15,600 lb.ft.
W_1	150 (2.33)(12.75)	= 4460 "	5.5 '	=	24,800 "
W_2		= 3940 "	5.25'	=	20,660 "
W_4	20.75 (3.83)(100)	= 7950 "	8.58'	=	68,250 "
	Total weight	19210 lbs:	Total M		129,310 lb.ft.

$$x = \frac{129,310 \text{ lb.ft.}}{19,210 \text{ lbs.}} = 6.72'$$

$$e' = \frac{6.4' \ (7870)}{19,210} = 2.62' \ ; \ 6.72' - 2.62' = 4.10'$$

$$e = 5.25' - 4.10' = 1.15'$$

Earth Pressure: Pressure greater at toe.

$$P = \frac{19,210}{10.5'} (1 \pm .66) = \begin{matrix} 3020 \text{ lb.per sq.ft.} \\ 620 \text{ lb.per sq.ft.} \end{matrix}$$

Overturning Factor:

$$\text{Ratio} = \frac{129,310 \text{ lb.ft.}}{7870 \text{ lb.}(6.4)} = 2.6$$ (this is satisfactory)

Sliding Factor:

$$\text{Ratio} = \frac{19,210 \text{ lb. } (.50)}{7870 \text{ lb.}} = 1.2$$ (this is unsatisfactory)

Check if Resultant intersects base within middle third:

$$\frac{10.5}{3} = 3.5' \ ; \quad \frac{3.5}{2} = 1.7'$$ (greater than 1.15' therefore within middle third)

Stem Moment: (tending to overturn)

$$P = C^w h = 20.75(100)(.33) = 685 \text{ lb.}$$

264 (12.75)	= 3370 lb.	$\frac{12.75'}{2}$	=	21,500 lb.ft.
$\frac{(685-264)}{2}(12.75)$	= 2680 lb.	$\frac{12.75'}{3}$	=	11,400 lb. ft.
Total wt.	= 6050 lb.	Totsl Mo.	=	32,900 lb. ft.

Stem Moment: (tending to stablize)

2860 (1.125') = 3210 lb.ft.

4460 (1.165') = 5200 lb.ft.

Total Mo.= 8410 lb.ft.

Total Stem Moment:

M = 32,900 lb.ft. - 8410 lb.ft.= 24,490 lb.ft.

Total Toe Moment:

$p = \frac{6.17\ (2400)}{10.5} + 620 = 2040$ lb.per sq.ft.(earth pressure at edge of stem).

Moment due to Earth Pressure:

		arm	Moment
2040 (4.33)	= 8780 lbs.	$\frac{4.33}{2}$	= 18,900 ft.lb.
$\left(\frac{2400 - 2040}{2}\right)(4.33)$	= 795 lbs.	$\frac{2(4.33)}{3}$	= 2,260 "
Total weight	= 9575 lbs:	Total Mo.	= 21,160 ft.lb.

Moment due to toe concrete:

150 (2.5) (4.33) = 1615 lbs. $\frac{4.33}{2}$ = 3,470 ft.lb.

Total Moment = 21,160 - 3,470 = 17,690 ft.lb.

Total Shear = 9,575 - 1615 = 7,960 lbs.

Total Heel Moment:

$p = \frac{3.83\ (2400)}{10.5} + 620 = 1500$ lbs.per sq. ft. (earth pressure at edge of stem)

Earth Pressure Moment upward:

620 (3.83)	= 2370 lb.	$\frac{3.83}{2}$	= 4,550 ft.lb.
$\left(\frac{880}{2}\right)(3.83)$	= 1685 lb.	$\frac{3.83}{3}$	= 2,150 ft.lb.
Total wt	= 4055 lb. :	Total Mo.	= 6,700 ft.lb.

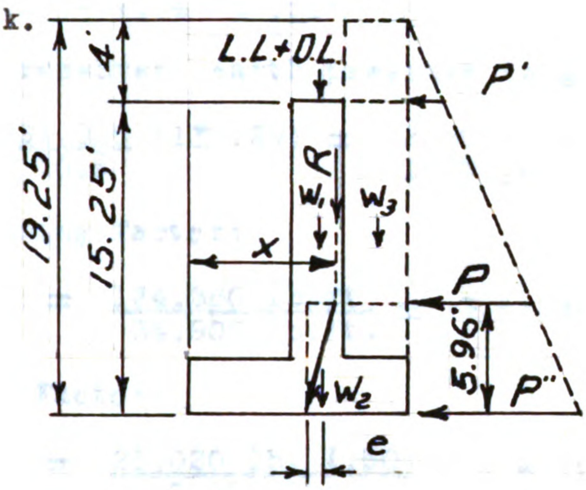
Earth and Concrete Moment Downward:

7950 + 1435 = 9,385 lbs. $\frac{3.83}{2}$ = 18,000 ft.lb.

Total Moment = 18,000 - 6,7000 = 11,300 ft. lbs.

Case 3

Pressure to retained soil without allowance for live load surcharge. Stabilizing moment due to dead and live loading of deck.



$P' = .33\ (4)(100) = 132$ lb.

$P'' = .33\ (19.25)(100) = 636$ lb.

To find "P": horizontal force due to earth pressure.

132 (15.25)	= 2,015 lb.	$\frac{15.25}{2}$	= 15,400 lb.ft.
$\frac{(636 - 132)}{2}$ (15.25)	= 3,840 lb.	$\frac{15.25}{3}$	= 19,500 "
Total P	= 5,855 lb.	Total Mo.	= 34,900 lb.ft.

$y = \frac{34,900 \text{ lb.ft.}}{5,855 \text{ lb.}} = 5.96'$

To find Resultant of vertical forces:

D.L.+ L.L. of Deck $= \left[16,650 + \frac{874}{2}(33.58)\right] \div 5.17 = 6200\#/\text{ft.}$

			arm	moment
D.L. L.L. of Deck		= 6200 lb.	5.46'	= 33,900 lb.ft.
W_1		= 4460 "	5.5	= 24,800 "
W_3	100(3.83)(16.75)	= 6420 "	8.58	= 55,000 "
W_2		= 3940 "	5.25	= 20,660 "
	Total wt.	= 21020 lbs.	Total Mo.	= 134,360 lb.ft.

$$x = \frac{134{,}360 \text{ lb.ft.}}{21{,}020 \text{ lbs.}} = 6.4'$$

$$e' = \frac{5.96\ (5{,}855)}{21{,}020} = 1.66' \quad : \quad 6.4 - 1.66 = 4.74'$$

$$e = 5.25 - 4.74 = .51'$$

Earth Pressure: earth pressure is greater at toe

$$P = \frac{21{,}020}{10.5}(1 \pm .29) = \begin{matrix} 2580 \text{ lb.per sq.ft.} \\ 1420 \text{ lb. per sq.ft.} \end{matrix}$$

Overturning Factor:

$$\text{Ratio} = \frac{134{,}360 \text{ lb.ft.}}{34{,}900 \text{ lb.ft.}} = 3.8 \text{ (satisfactory)}$$

Sliding Factor:

$$\text{Ratio} = \frac{21{,}020 \text{ lb. } (.50)}{5{,}855 \text{ lb.}} = 1.8 \text{ (Satisfactory)}$$

It is obvious that the Resultant intersects the base within the middle third as "e" is very small.

Stem Moment:

$$p = .33\ (100)(16.75) = 552 \text{ lb.}$$

Overturn Moment: (Due to earth pressure)

$$132\ (12.75) = 1{,}685 \text{ lb.} \quad \frac{12.75}{2} = 10{,}700 \text{ ft.lb.}$$

$$\frac{420}{2}\ (12.75) = 2{,}680 \text{ lb.} \quad \frac{12.75}{3} = 11{,}400 \text{ "}$$

Total wt. = 4,365 lb. Total Mo. = 22,100 ft.lb.

Stablizing Moment: (Due to loaded deck and wt. of stem)

$$6200\ (1.125) = 7{,}000 \text{ ft.lb.}$$

$$4460\ (1.165) = 5{,}200 \text{ "}$$

Total Mo. = 12,200 ft.lb.

Stem Moment = 22,100 - 12,200 = 9,900 ft.lb.

Toe Moment:

$$p = \frac{6.17\ (1160)}{10.5} + 1420 = 2100 \text{ lb.per sq.ft.}$$ (earth pressure next to stem edge)

Moment Upward:

$2100\ (4.33) = 9{,}100 \text{ lb.} \quad \frac{4.33}{2} = 19{,}700 \text{ ft.lb.}$

$\frac{480}{2}\ (4.33) = \underline{1{,}030} \text{ " } \quad \frac{(4.33)2}{3} = \underline{2{,}980} \text{ "}$

Total wt. = 10,130 lb. Total Mo. = 22,680 ft.lb.

Total Toe Moment:

$M = 22{,}680 - 3{,}470 = 19{,}210 \text{ ft.lb.}$

Heel Moment:

$P = \frac{3.83(1160)}{10.5} + 1420 = 1843 \text{ lb./sq.ft.}$ (pressure next to stem edge)

Moment Upward:

$1420\ (3.83) = 5440 \text{ lb.} \quad \frac{3.83}{2} = 10{,}400 \text{ ft.lb.}$

$\frac{423}{2}\ (3.83) = \underline{811} \text{ " } \quad \frac{3.83}{3} = \underline{1{,}035} \text{ "}$

Total wt. = 6251 lb. Total Mo. = 11,435 ft.lb.

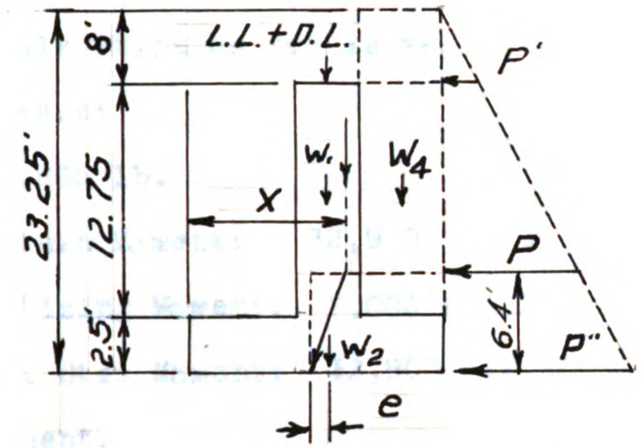
Moment Downward:

$M = 6420 + 1435 = 7855 \text{ lb.} \quad \frac{(3.83)}{2} = 15{,}030 \text{ ft.lb.}$

Total Moment = 15,030 - 11,435 = 3,595 ft.lb.

Case 4

Pressure due to retained soil with live load surcharge and stabilizing moment due to dead and live load on deck.



Resultant Horizontal Force = 7870 lb. : y = 6.4'

To find Resultant Vertical Force:

		arm	moment	
D. & L. load on deck	--6200	5.46'	33,900	ft.lb.
W_1	= 4460	5.5	24,800	"
W_2	= 3940	5.25	20,660	"
W_4	= 7950	8.58	68,250	"
Total weight	= 22550#	.Total Mo. =	147,610	ft. lb.

$$x = \frac{147{,}610 \text{ ft.lb.}}{22{,}550 \text{ lb.}} = 6.55'$$

$$e' = \frac{6.4\ (7870)}{22{,}550} = 2.23' \quad : \quad 6.55' - 2.23' = 4.32'$$

$$e = 5.25 - 4.32 = .93'$$

Earth Pressure: (greater at the toe)

$$P = \frac{22{,}550}{10.5}\ (1 \pm .55) = \begin{matrix} 3320 \text{ lb. per sq.ft.} \\ 965 \text{ lb. per sq.ft.} \end{matrix}$$

Overturning Factor:

$$\text{Ratio} = \frac{147{,}610 \text{ ft.lb.}}{7870\ (6.4')} = 2.9 \quad \text{(satisfactory)}$$

Sliding Factor:

$$\text{Ratio} = \frac{22{,}550 \text{ lb. } (.50)}{7870 \text{ lb.}} = 1.4 \text{ (this is unsatisfactory)}$$

It is obvious that the Resultant intersects the base within the middle third as "e" is very small.

Stem Moment:

p = 685 lb.

Overturn Moment: 32,900 ft.lb.

Stablizing Moment: 7,000 + 5,200 = 12,200 ft.lb.

Total Stem Moment: 32,900 -12,200 = 20,700 ft.lb.

Toe Moment:

$$p = \frac{2355\ (6.17)}{10.5} + 965 = 2345 \text{ lb.per sq.ft.}$$

Upward Moment:

$$2345\ (4.33) = 10{,}150 \text{ lb. } \frac{4.33}{2} = 21{,}800 \text{ ft.lb.}$$

$$\frac{975}{2}\ (4.33) = \underline{2{,}110} \text{ lb. } \left(\frac{4.33}{3}\right)2 = \underline{6{,}100} \text{ "}$$

total wt.= 12,260 lb. Total.M.= 27,900 ft.lb.

Total Toe Moment: 27,900 - 3,470 = 24,430 ft.lb.

Heel Moment:

$$p = \frac{2355\ (3.83)}{10.5} + 965 = 1823 \text{ lb.per sq.ft.(earth pressure)}$$

Upward Moment:

$$965\ (3.83) = 3700 \text{ lb. } \frac{3.83}{2} = 7{,}070 \text{ ft.lb.}$$

$$\frac{858}{2}\ (3.83) = \underline{1645} \text{ lb. } \frac{3.83}{3} = \underline{2{,}100} \text{ ft.lb.}$$

Total wt = 5345 lb. Total Mo.= 9,170 ft.lb.

Total Moment = 18,000 - 9,170 = 8,830 ft. lb.

Total Shear = 9,385 - 5,345 = 4,040 lbs.

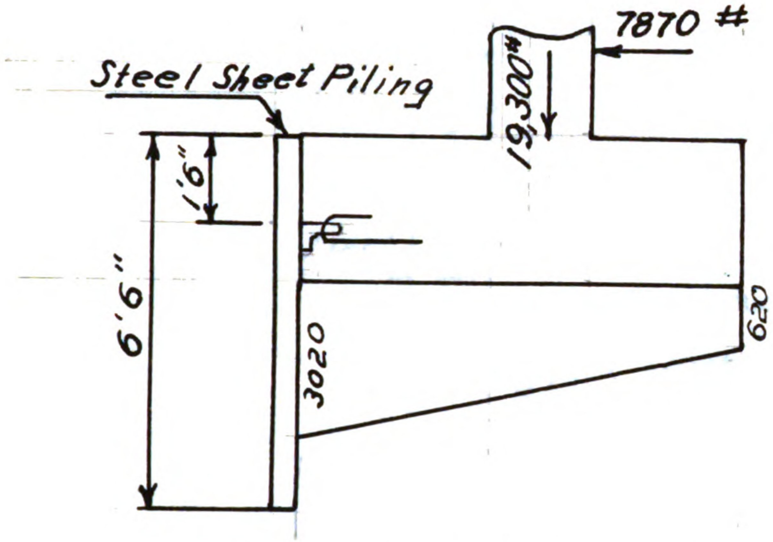
Results of the Invertigations of the Four Cases:

ITEM		CASE 1	CASE 2	CASE 3	CASE 4
Eccentricity		.14	1.15	.51	.93
Earth Pressure lb./sqft.	Toe	1,370	3,020	2,580	3,320
	Heel	1,170	620	1420	965
Safety Factor	Overturning	4.4	2.6	3.8	2.9
	Sliding	1.7	1.2	1.8	1.4
Moments ft.lb.	Toe	9,476	17,690	19,210	24,430
	Heel	3,811	11,300	3,595	8,830
	Stem	11,400	24,490	9,900	20,700
Shear lbs.	Toe	4,291	7,960	8,505	10,635
	Heel	1,892	5,330	1,604	4,040
	Stem	2,690	6,050	4,365	6,050

Conclusions:

1. Maximum Stem moment occurs at Case 2 loading.
2. Maximum Toe moment occurs at Case 4 loading:
3. Maximum Heel moment occurs at Case 2 loading.
4. Greatest Excentricity occurs at Case 2 loading, which is within the middle third.
5. Lowest Overturning Factor , 2.6, is satisfactory.
6. Lowest Sliding Factor, 1.2, is unsatisfactory.

Resistance to Sliding:



Lowest Sliding Factor occurs with case 2 loading:

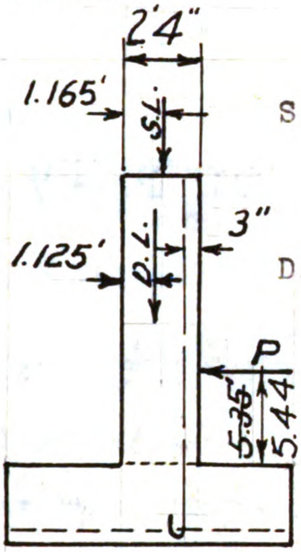
To secure against sliding there is provided steel sheet piling driven to a depth of 6' 6" and a secure bond between footing and piling (fig. above) exists by means of an angle iron welded to piling and a length of reinforcement bar hooked through angle and embedded in the footing.

$$\text{Sliding Factor} = \frac{19{,}310\ (.50) + 3020(2.5)}{7870} = 2.2$$

This factor is satisfactory.

STEM ANALYSIS

Forces acting on Stem:



S.L. (Stem Load)	4460 lb.
D.L. (Dead Load of structure)	2860 lb.
P (Earth Pressure)	6050 lb.

Fig. 9

To find "jd":

f_c = 1200 psi.

f_s = 3000 psi.

n = 10

$kd = \frac{1200\ (25)}{3000} = 10$ in.

$d' = \frac{10}{3}$ 3.33in.

jd = 25 - 3.33 = 21.67 in.

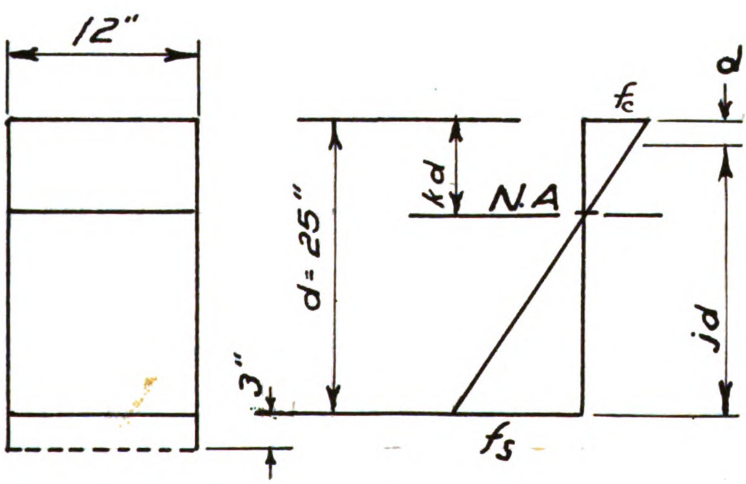


Fig. 10

Stem Analysis: (moment Case "2")

$M = 24,490$ ft.lb.

$V = 6,050$ lb.

Depth required:

$$d = \sqrt{\frac{M}{K\ b}} = \sqrt{\frac{24,490\ (12)}{208\ (12)}} = 11" \quad : \text{(stem is 28")}$$

Area of steel required:

$$A_s = \frac{M}{f_s\ (jd)} = \frac{24,490\ (12)}{18,000\ (21.67)} = .75 \text{ sq. in.}$$

Area of steel provided:

$\frac{3}{4}$" circular bars, spaced at 8" and 1'4" alternately.

$$A_s = .442\ \left(\frac{12}{8}\right) = .663 \text{ sq. in.}$$

Check f_s:

$$f_s = \frac{M}{A_s\ (jd)} = \frac{24,490\ (12)}{.662\ (21.67)} = 20,500 \text{ lb./sq.in.}$$

The steel above seems to be overstressed; however Mr. Puffer of Michigan State Highway explained to me that it is their policy of allowing a 30% over design stress because they feel that the bridge will probably never be fully stressed. Also, they feel that the rigidity of the structure resists the force caused by the loading sufficiently to warrant this policy. The problem above is beyond the scope of this paper, therefore I will only mention that the stress above is well within the 30% allowed. This same condition occurs in the steel placed in the toe.

Check f_c:

$$f_c = \frac{2M}{jd\,(b)(kd)} = \frac{24,490\ (12)2}{21.67(12)(10)} = 226 \text{ lb./sq.in.}$$

(1200 psi.allowable)

Check shear:

$$v = \frac{V}{b\,(jd)} = \frac{6050}{12\ (21.67)} = 23.2 \text{ psi.}$$ (60 psi. allowable)

Check bond:

$$u = \frac{8V}{7\,\Sigma_o\,d} = \frac{8\ (6050)}{7\left(\frac{12}{8}\right)\frac{3}{4}\ (77)\ 25} = 104 \text{ psi.}$$ (150 psi. allowable)

Anchorage is sufficient as a standard hook is provided which is hooked over another bar and embedded 27".

Stem analysis $\frac{1}{3}$h:

M = 13,780 - 8410 = 5,370 ft.lb.

Thickness of stem is constant throughout.

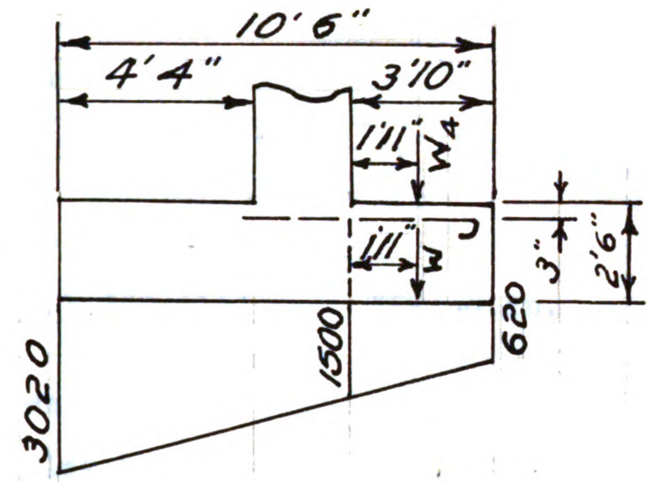
Check f_s:

$$f_s = \frac{M}{A_s\,(jd)} = \frac{5370\ (12)}{.331\ (21.67)} = 9,000 \text{ psi.}$$ (18,000 psi allowable)

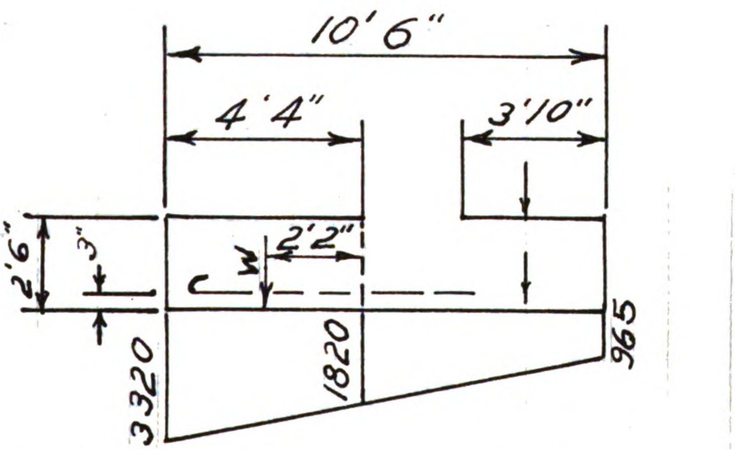
The amount of steel used above is just half of the steel provided in bottom of stem. Every other bar is cut off 1'8" above the $\frac{1}{3}$ h point taken above, therefore sufficient steel is present.

Forces acting on base of Abutment:

A. Forces acting on Heel (Case 2):



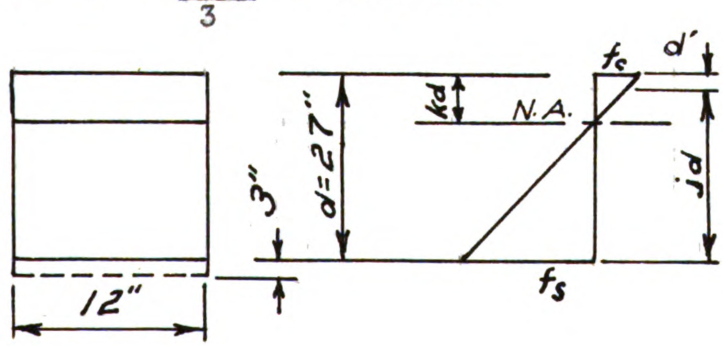
B. Forces acting on Toe (Case 4)



C. To find "jd":

$$kd = \frac{27(1200)}{3000} = 10.8 \text{ in.}$$

$$jd = 27 - \frac{10.8}{3} = 23.4 \text{ in.}$$



Heel analysis: (moment Case "2")

$M = 11,300$ ft. lb.

$V = 5,330$ lb.

Depth required:

$$d = \sqrt{\frac{11,300\ (12)}{208\ (12)}} = 7.4"$$ (30" is provided)

Area of steel required:

$$A_s = \frac{11,300\ (12)}{18,000\ (23.4)} = .322 \text{ sq.in.}$$

Area of steel provided:

$\frac{3}{4}$" circular bars, spaced at 1'4"

$$A_s = \frac{.44\ (12)}{16} = .33 \text{ sq.in.}$$

Check f_s:

$$f_s = \frac{11,300\ (12)}{.33\ (23.4)} = 17,500 \text{ psi.}$$ (18,000 psi. allowable)

Check f_c:

$$f_c = \frac{11,300\ (12)}{23.4\ (12)(10.8)} = 45 \text{ psi.}$$ (very low)

Check bond:

$$u = \frac{8\ (5,330)\ 16}{7\ (9)(\pi)25} = 43 \text{ psi.}$$ (150 psi. allowable)

Check shear:

$$v = \frac{5,330}{12\ (23.4)} = 19 \text{ psi.}$$ (very low)

Anchorage:

$$L = \frac{17,500}{4(150)}\left(\frac{3}{4}\right) = 22"$$ required (33" provided)

Toe analysis: (moment Case "4")

$M = 24,430$ ft.lb.

$V = 10,635$ lb.

Depth required:

$$d = \sqrt{\frac{24,430\ (12)}{208\ (12)}} = 10.8" \quad \text{(30" provided)}$$

Area of steel required:

$$A_s = \frac{24,430\ (12)}{18,000\ (23.4)} = .695 \text{ sq.in.}$$

Area of steel provided:

$\frac{3}{4}$" circular bars, spaced at 8" and 1'4" alternately

$$A_s = \frac{.442\ (12)}{8} = .664 \text{ sq. in.}$$

Check f_s:

$$f_s = \frac{24,430\ (12)}{.663\ (23.4)} = 18,900 \text{ psi.} \quad \text{(18,00 psi. design stress allowable)}$$

The overstress here is almost negligible.

Check f_c:

$$f_c = \frac{2\ (24,430)\ 12}{23.4\ (12) 10.8} = 193 \text{ psi. (very low)}$$

Check bond:

$$u = \frac{8\ (10,635)\ 8}{7\ (9)(\pi)\ 25} = 137 \text{ psi. (150 allowable)}$$

Check shear:

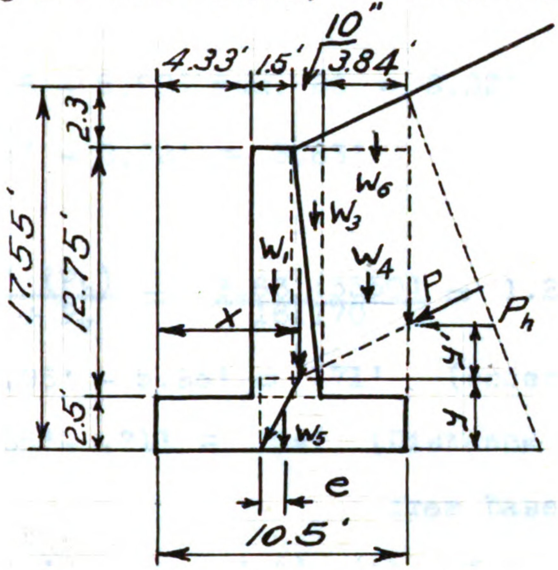
$$v = \frac{10,635}{12\ (23.4)} = 37.8 \quad \text{(very low)}$$

Anchorage:

$$L = \frac{18,900}{4\ (150)} \left(\frac{3}{4}\right) = 24" \quad \text{(34" provided)}$$

WINGWALL ANALYSIS

The wingwall serves only to retain soil with a 26 degree surcharge.



Angle of repose taken as 30 degrees.

$$C = .946 \left[\frac{.946 - (.9 - .755)^{1/2}}{.946 + (.9 - .755)^{1/2}} \right] = .4$$

Soil pressure against footing:

Resultant Vertical Force:

		arm	moment
$W_1 = (12.75)(1.5)(150)$	= 2870	5.1'	= 14,600 ft.lb.
$W_2 = (12.75)\frac{(.83)}{2}(150)$	= 793	6.1	= 4,840 "
$W_3 = (12.75)\frac{(.83)}{2}(100)$	= 530	6.33	= 3,370 "
$W_4 = (12.75)(3.83)(100)$	= 4880	8.58	= 42,000 "
$W_5 = (10.5)(2.5)(150)$	= 3840	5.25	= 20,700 "
$W_6 = (4.67)\frac{(2.3)}{2}(100)$	= 537	8.94	= 4,800 "
	Total wt = 13450		Total M = 80,310 ft.lb.

$$x = \frac{80,310}{13,450} = 5.96'$$

Resultant Horizontal Force:

$$p = WCh = 100\ (.4)\ (17.55) = 703 \text{ lb.}$$

$$P = \frac{703}{2}(17.55) = 6,170 \text{ lb.}$$

$P_h = 6,170 \ (.9) = 5,550$ lb.

$P_v = 6,170 \ (.44) = 2,720$ lb.

To find Y'

$y = (10.5 - 5.96) \tan. 26 = 2.22'$

$y' = 5.85' - 2.22' = 3.63'$

To find e:

$$e' = \frac{y' \ (P_h)}{R + P_v} = \frac{3.63 \ (5550)}{16,170} = 1.25'$$

$5.96' - 5.25' = .71'$ (eccentricity of R_h)

$e = 1.25' - .71' = .54'$ (Distance R intersects base from base center.)

$.54'$ less than $\frac{3.5'}{2}$; (therefore R intersects base within middle third)

Earth Pressure on Base:

$$P = \frac{16,170}{10.5} (1 \pm .309) = \begin{cases} 2020 \text{ lb./sq.ft. (at toe)} \\ 1065 \text{ lb./sq.ft. (at hee)} \end{cases}$$

Moment against overturning:

$$F = \frac{16,170 \ (5.96)}{5,550 \ (3.63)} = 4.8 \text{ (good safety factor)}$$

Moment against sliding:

$$F = \frac{16,170 \ (.50)}{5,550} = 1.46 \text{ (This is a little low)}$$

Sliding factor is a little low but there is sufficient fill over toe to provide passive resistance sufficient to keep wall from sliding.

Stem analysis:

$p = 100\ (.4)(15.05) = 602$ lb.

$P = \frac{602}{2}(15.05) = 4{,}503$ lb.

$P_h = 4{,}503(\tan 26^\circ) = 4070$ lb.

Overturn moment: $4{,}070\ (5') = 20{,}350$ ft.lb.

Stabilizing moment:

$M. = 12.75(1.5)(150)\frac{(1.5)}{2} + \frac{.83}{2}(12.75)(150)(1.92)$

$M = 3{,}670$ ft.lb.

Total Moment: $20{,}350 - 3{,}670 = 16{,}680$ ft.lb.

$d = \sqrt{\frac{16{,}680\ (12)}{208\ (12)}} = 10''$ (30" provided)

$A_s = \frac{16{,}680\ (12)}{18{,}000(21.67)} = .512$ sq. in. (required)

Steel Area provided:

$\frac{3}{4}''$ circular bars, spaced at 8"

$A_s = \frac{.442\ (12)}{8} = .663$ sq. in.(provided)

Check f_s:

$f_s = \frac{16{,}680\ (12)}{.663\ (21.67)} = 13{,}900$ psi. (18,000 psi. allowable)

Check f_c:

$f_c = \frac{2\ (16{,}680)}{10.8\ (12)\ 21.67} = 119$ psi. (very low)

Check shear and bond:

$v = \frac{4{,}070}{12\ (21.67)} = 15.6$ psi. (60 psi. allowable)

$u = \frac{8\ (4070)8}{7(9)(\pi)25} = 52.7$ psi. (150psi. allowable)

Check steel 4' above base:

$p = 11.05 (100) (.4) = 442$ lb.

$P = \frac{442}{2} (11.05) = 2,450$ lb.

Overturning Moment:

$$M = 2,450 \frac{(11.05)}{3} = 9,030 \text{ ft.lb.}$$

Total Moment:

$M = 9,030 - 2150 = 6,880$ ft.lb. (allowing for stabilizing moment)

To find "d":

$d = \sqrt{\frac{6,880 (12)}{208 (12)}} = 6"$ (narrowest portion of stem 18"; therefore depth is O.K)

Steel provided: half of the steel in bottom of stem is cut off 1' 8" above $\frac{1}{3}h$, therefore we will use one-half the steel provided in bottom of stem in determing the steel stress.

Check f_s:

$f_s = \frac{6,880 (12)}{.331 (.87)(19.1)} = 15,000$ psi. (18,000 psi. allowed)

Sufficient steel is present in the stem.

Heel Analysis:

$$P = \frac{(2020 - 1065)(3.83)}{10.5} + 1065 = 1415 \text{ lb/sq.ft. (pressure next to stem)}$$

Moment upward:

$$1065\,(3.83)\left(\frac{3.83}{2}\right) = 7{,}840 \text{ lb.ft.}$$

$$\frac{350}{2}\,(3.83)\left(\frac{3.83}{3}\right) = \underline{633} \text{ "}$$

$$\text{Total Moment} = 8{,}473 \text{ lb.ft.}$$

Moment downward:

$$M = (4880 + 1440)(1.92) = 11{,}360 \text{ lb.ft.}$$

Total Moment:

$$M = 11{,}360 - 8{,}473 = 2{,}887 \text{ lb.ft.}$$

Total Shear:

$$V = 6320 - 4575 = 1{,}745 \text{ lb.}$$

Steel Provided:

$\frac{3}{4}$" circular bars, spaced at 2' 0".

$$A_s = \frac{.442}{24}(12) = .22 \text{ sq.in. (provided)}$$

Check f_s:

$$f_s = \frac{12(2887)}{.22\,(23.4)} = 675 \text{ psi. (very low)}$$

Obvious that shear is satisfactory.

Check bond:

$$u = \frac{8\,(1745)6}{7\,(3)(\pi)\,27} = 62.8 \text{ psi. (150 psi. allowed)}$$

Toe Analysis:

$$P = \frac{955\ (6.17)}{10.5} + 1065 = 1625 \text{ lb./sq.ft. (earth presure edge of stem)}$$

Moment upward:

$1625\ (4.33) = 7040 \qquad \left(\frac{4.33}{2}\right) = 15{,}100$ ft.lb.

$\frac{560}{2}\ (4.33) = \underline{1210} \qquad 2\left(\frac{4.33}{3}\right) = \underline{3{,}490}$ "

Total wt. = 8250 Total Mo = 18,590 ft.lb.

Total Moment:

$$M. = 18{,}590 - 3{,}470 = 15{,}120 \text{ ft. lb.}$$

Total Shear:

$$V = 8{,}250 - 1{,}615 = 6635 \text{ lb.}$$

Depth is obviously satisfactory.

Area Steel Required:

$$A_s = \frac{15{,}120\ (12)}{18{,}000\ (23.4)} = .43 \text{ sq.in.}$$

Area Steel Provided:

$\frac{3}{4}$" circular bars, spaced at 1' 0".

$$A_s = .442 \text{ sq.in.}$$

Check f_s: and f_c :

$$f_s = \frac{15{,}120\ (12)}{.442\ (23.4)} = 17{,}500 \text{ psi. (18,000 allowable)}$$

$$f_c = \frac{2\ (15{,}120)\ (12)}{23.4\ (12)\ 10.8} = 240 \text{ psi. (very low)}$$

Check v: and u:

$$v = \frac{6{,}635}{12\ (23.4)} = 23.6 \text{ psi. (very low)}$$

$$u = \frac{8\ (6{,}635)\ 8}{7\ (9)\ (\pi) 25} = 85.6 \text{ psi. (150 allowable)}$$

Anchorage is sufficient (same as abutment)

CONCLUSION

The analysis of this structure has shown that it is adequately designed and meets all specifications with the exception of the overstressed steel in the stem of the abutment while undergoing maximum bending moment.

The policy of allowing a 30% overstress, if justified, places the stress of the stem steel well within the limit.

Thus the structure can be expected to serve safely and adequately all the traffic that may be passing over it and at the same time provide free passage of all water that may be expected to flow in Johnson Creek.

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