

AN ANALYSIS OF THE DESIGN
OF A SINGLE SPAN HIGHWAY BRIDGE

Thesis for the Degree of B. S.
MICHIGAN STATE COLLEGE

Donald M. Tubbs
1949

**SUPPLEMENTARY
MATERIAL
IN BACK OF BOOK**

**An Analysis of the Design of a Single Span
Highway Bridge**

A Thesis Submitted to

**The Faculty of
MICHIGAN STATE COLLEGE**

OF

AGRICULTURE AND APPLIED SCIENCE

by

Donald M. Tubbs

Candidate for the Degree of

Bachelor of Science

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THESIS

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Acknowledgements

I would like to express my utmost appreciation to Mr. H. R. Puffer, of the Michigan State Highway Department, for supplying the plans for this thesis. Also to Dr. Richard Pian and Proffessor Chester Allen, for their help in the preparation of this thesis.

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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 Officials.
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 _l	ultimate compressive strength of concrete.
f _s	stress in tensile reinforcement.
I	moment of inertia.
j	ratio of distance (jd) between resultants of compressive and tensile stresses 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 shall 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 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 .302 x 10,000 = 3,020 lbs.

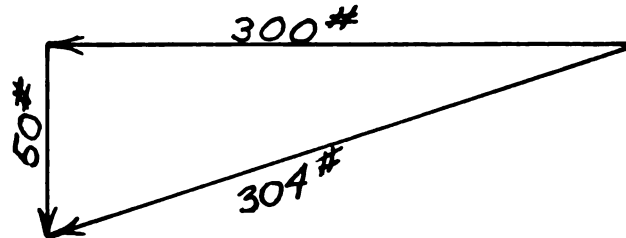
Load- 300 x 8.208/2 = 1230 lbs.

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; $304 \times 8.208/2 = 1247 \text{ lbs.}$

Bolts control as they have minimum capacity.

1(max.) $1-7/8" \# 3/8" = 2-1/4"$

$$f(\text{horizontal}) \quad \frac{1230 \times 2.25 \times 6}{.625 \times (1.75)} = 8650 \text{ psi}$$

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

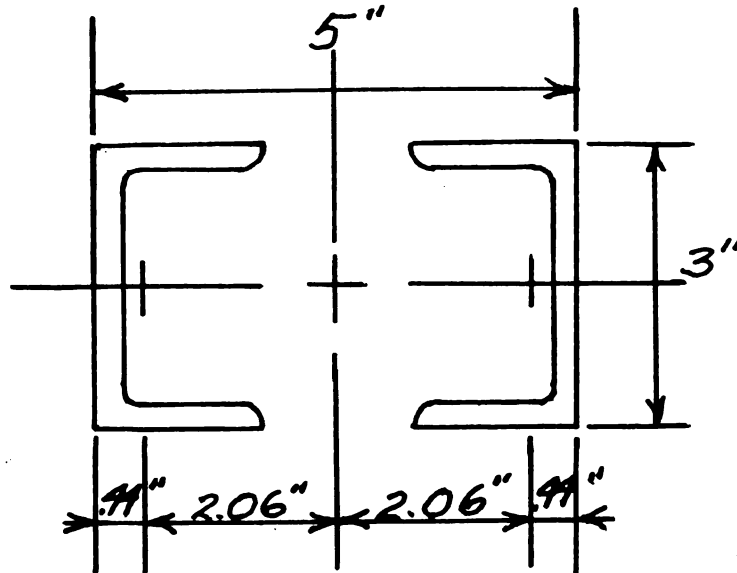
$$f(\text{total}) \quad 12700 \text{ psi}$$

$$F(\text{allowable}) \quad 18000 \text{ psi}$$

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.)



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

$$\text{Moment(M): } M = 1/8 w l^2$$

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

$$M = 34,000 \text{ in.-lbs.}$$

$$I = I_0 + A d^2 \quad d = 2.06 \text{ in.} \quad I_0 = 2 \times .25 = .50 \text{ in}^4$$

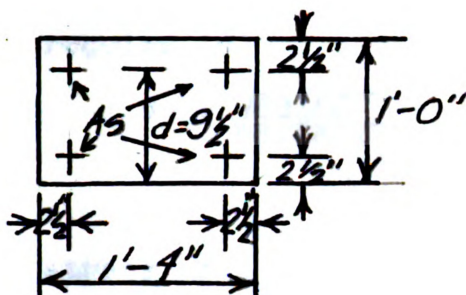
$$A = 2 \times 1.46 = 2.92 \text{ sq. in.}$$

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

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

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

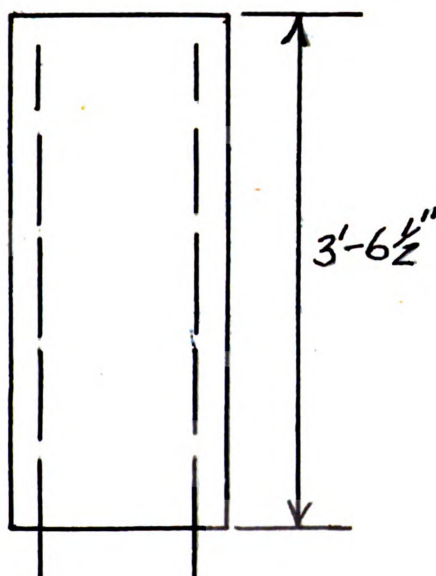
Posts



Intermediate Posts

$$b = 16 \text{ in.}$$

$$d = 9\frac{1}{2} \text{ in.}$$



$$A_s = 2 \times .44 = 0.88 \text{ sq. in.}$$

2 bars in compression

2 bars in tension

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

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

Bending moments;

$$150 \times 9.54 \times 32 = 45,800 \text{ in. lbs.}$$

$$300 \times 9.54 \times 5 = 14,300 \text{ in. lbs.}$$

$$\text{total} \quad \underline{\quad\quad\quad} 60,100 \text{ in. lbs.}$$

Posts (cont.)

required (d)

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

9.50 inches furnished

stress (f_s)

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

(18000 psi allowable)

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

The posts 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.

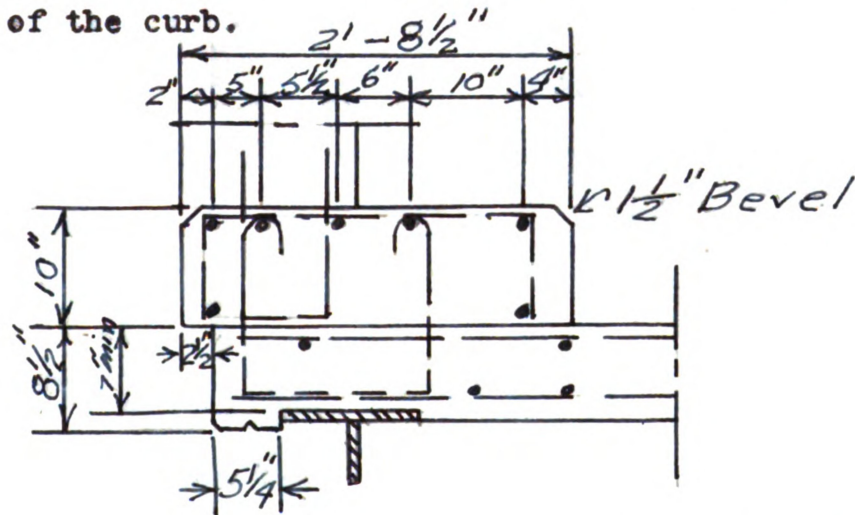
Sidewalks and Curbs

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.

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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 reinforcing perpendicular to center line of roadway.

Distribution of wheel loads;

(for spans 2-7 ft.)

$$E = 0.6s + 2.5 \text{ ft.}$$

Bending moment(M) for freely supported spans;

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

Bending moment(M) for continuous spans;

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

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, either 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:

$$E = 0.6s + 2.5 \quad s = 5' - 103/4" = 5.146 \text{ ft.}$$

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

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

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

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

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

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 min.

Amount of tension steel required laterally;

$$A_s = \frac{M}{f_s j d}$$

where;

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

f_s = allowable steel stress
(18000 psi)

j = 7/8

d = effective depth

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

$$A_s = .685 \text{ sq. in./ft. required}$$

$$A_s = .694 \text{ sq. in./ft. furnished}$$

Steel at bottom for lateral distribution;

Percent of main steel required

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

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

$$A_s = .685 \times .441 = .320 \text{ sq. in./ft. required}$$

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 beams 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;

Longitudinal 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 = width of one traffic lane (not to exceed 10')
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"WFL03#)		= 130 lbs/ft.
Future wearing surface 20 x 5.146		= 103 lbs/ft.
Total dead load per foot of span		<u>= 683 lbs/ft.</u>

Dead load moment

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

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

$$M = 2,315,000 \text{ in.-lbs.}$$

1. The first part of the document is a list of the names of the persons who have been appointed to the various offices of the city government. The names are listed in alphabetical order, and each name is followed by the name of the office to which the person has been appointed.

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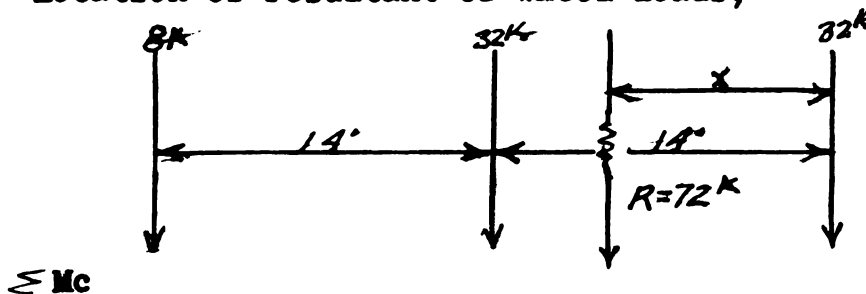
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Stringers (cont.)

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 causes maximum center moment when the loads are so placed that the center of the beam lies midway between the load and the resultant of all the loads on the span.

Location of resultant of wheel loads;



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

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

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

$$x = 9.333 \text{ ft.}$$

Live load moment;

End reaction

$$\Sigma MR_2 = 0$$

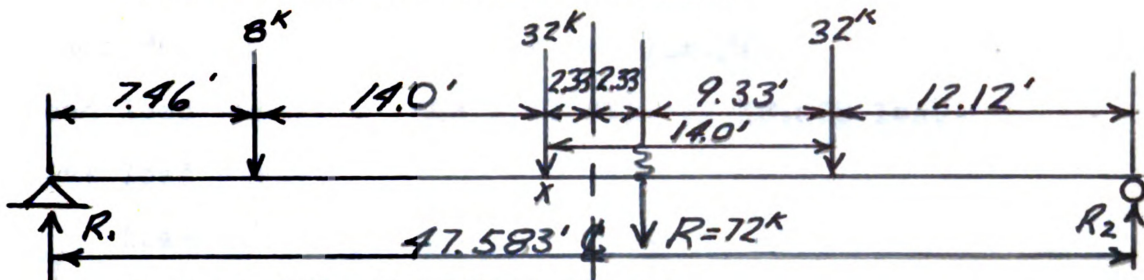
$$R_1 \times 47.583 - 72000 \times (9.33 + 12.12)$$

$$R_1 = \frac{72000 \times 21.45}{47.583}$$

$$R_1 = 32,450 \text{ lbs.}$$

Stringers (cont.)

Beam loading for maximum moment;



x = point of maximum moment

bending moment(M) for one traffic lane

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

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

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

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

bending moment for each stringer

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

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

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

Total moment equals deadload moment plus live load moment.

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

Section modulus required;

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

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

1. The first part of the document is a letter from the President of the United States to the Congress, dated January 3, 1862. It is a very important document, as it contains the President's annual message to Congress. The letter is written in a formal, dignified style, and it is one of the most important documents in the history of the United States. It is a document that has been read and studied by many generations of Americans, and it is a document that has shaped the course of our nation's history.

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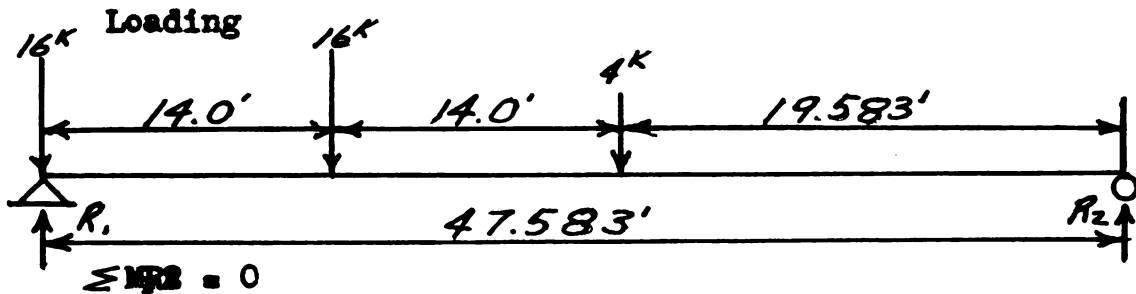
Stringers (cont.)

End shear;

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

Dead load shear = $683 \times 47.583 = 16,200$ lbs.

Live load shear;



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

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

$$R_1 = 28.9 \text{ kips}$$

Live load shear = $R_1 + \text{Impact}$

$$I = .22(28.9) = 6.36 \text{ kips}$$

Total live load shear

$$= 28,000 + 6,360 = 35,260 \text{ lbs.}$$

Total shear;

$$S = \text{deadload shear} + \text{live load shear} + I$$

$$S = 16,200 + 35,260 = 51,460 \text{ lbs.}$$

1. The first part of the document discusses the importance of maintaining accurate records of all transactions. It emphasizes that proper record-keeping is essential for the transparency and accountability of the organization. This section also outlines the specific procedures for recording and verifying financial data.

2. The second part of the document addresses the role of the audit committee in overseeing the financial reporting process. It details the committee's responsibilities, including reviewing the financial statements, assessing the effectiveness of internal controls, and ensuring compliance with applicable laws and regulations. The committee is also responsible for reporting its findings to the board of directors.

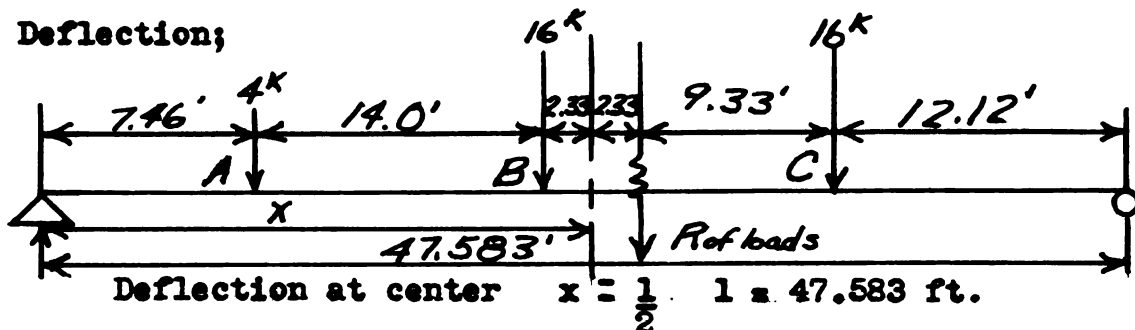
3. The third part of the document focuses on the internal control system, which is designed to prevent and detect errors and fraud. It describes the various components of the system, such as the segregation of duties, the authorization of transactions, and the reconciliation of accounts. The document also provides guidance on how to identify and address weaknesses in the internal control system.

4. The fourth part of the document discusses the importance of communication and collaboration between different departments and individuals within the organization. It stresses that effective communication is crucial for ensuring that everyone is aware of their responsibilities and the overall goals of the organization. This section also provides tips for improving communication and resolving conflicts.

5. The fifth part of the document provides a summary of the key points discussed in the previous sections. It reiterates the importance of accurate record-keeping, the role of the audit committee, the internal control system, and the need for effective communication and collaboration. The document concludes by expressing the organization's commitment to transparency and accountability.

Stringers (cont.)

Deflection;

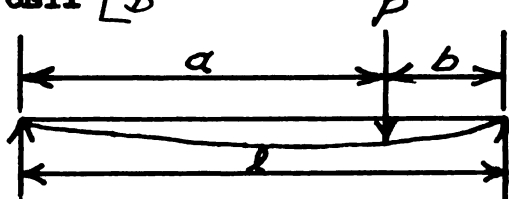


By taking each load and finding the deflection for it and adding them all together the total live load deflection can be found.

If point (x) is to the left of the load considered we use;

$$\Delta = \frac{Pbx}{6EI} (l^2 - x^2 - b^2)$$

If point (x) is to the right of the load considered we use;

$$\Delta = \frac{Pb}{6EI} \left[\frac{1}{b} (x-a) + (l^2 - b^2)x - x^3 \right]$$


Load A; $a = 89.5$ in. $b = 481.5$ in.
 $l = 571.0$ in. $x = 285.0$ in.

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

$$\Delta_A = .0346 \text{ inches}$$

Load B; $a = 258.0$ in. $b = 313.0$ in.
 $l = 571.0$ in. $x = 285.0$ in.

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

$$\Delta_B = .304 \text{ inches}$$

Stringers (cont.)

Load C; $b = 145.5 \text{ in.}$ $l = 571.0 \text{ in.}$

$x = 285.0 \text{ in.}$

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

$$\Delta_c = .216 \text{ inches}$$

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

$$\Delta = .5546 \text{ inches}$$

plus 22% for Impact

$$\Delta = .5546 + (.5546 \times .22) = .5546 + .122 = .6766 \text{ inches}$$

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

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

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 between centers 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.

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100
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 80
 70
 60
 50
 40
 30
 20
 10
 0

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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;

$$7.916 \times 50 = 395 \text{ sq. ft.}$$

$$1.5 \times 395 = 594 \text{ sq. ft.}$$

$$.5 \times 7 \times 2.75 \times 48.4 = 466 \text{ sq.ft.}$$

$$\text{total effective area} = 1060 \text{ sq. ft.}$$

Moving load;

$$30 \text{ lbs.} \times 1060 \text{ sq. ft.} = 31,800 \text{ lbs.}$$

Area required (end diaphragm);

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

Area furnished;

$$\text{Area of 2 } \angle 3" \times 3" \times 3/8" = 4.22 \text{ sq. inches}$$

The intermediate diaphragms will not be checked as they will meet all of the necessary 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.

• $\frac{1}{2} \times 100 = 50\%$

• $\frac{1}{2} \times 100 = 50\%$

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• $\frac{1}{2} \times 100 = 50\%$

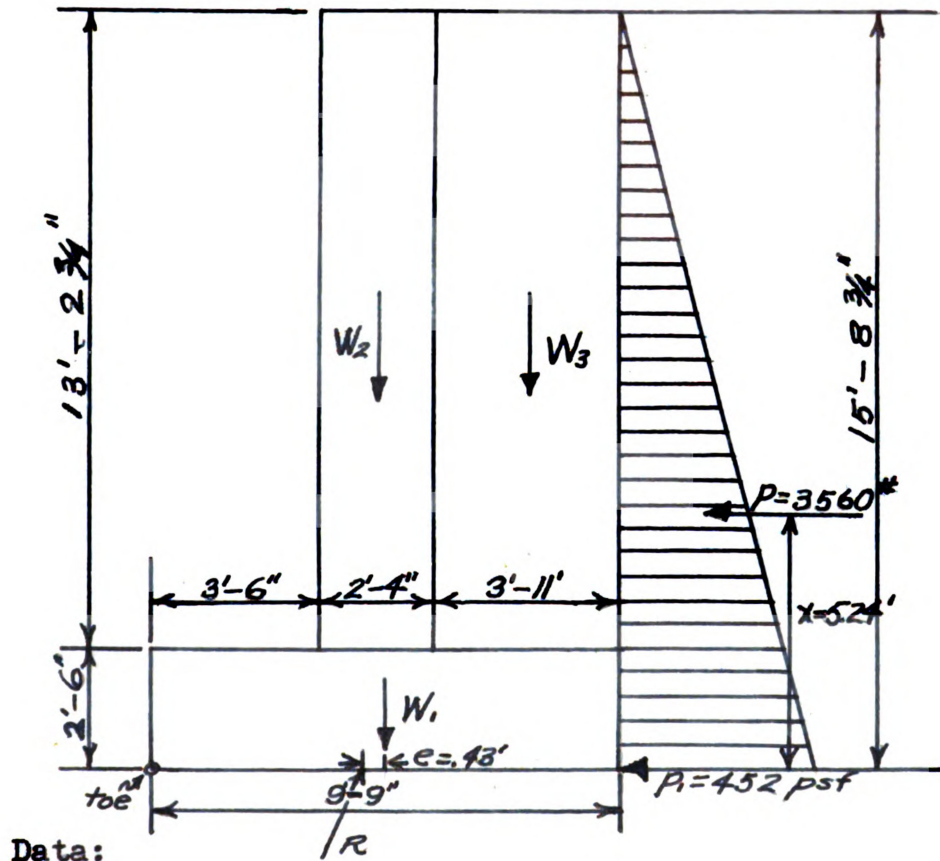
• $\frac{1}{2} \times 100 = 50\%$

• $\frac{1}{2} \times 100 = 50\%$

• $\frac{1}{2} \times 100 = 50\%$

Abutments

Case I -- No superstructure load or surcharge;



Earth $w_e = 100 \text{ lbs per cu. ft.}$

$\phi = 33^\circ 40' = \tan^{-1} 2/3 \text{ (earth on earth)}$

$\delta = 22^\circ 00' = \text{(earth on concrete)}$

Foundation pressure

$p = .4,100 \text{ psf(max)}$

wt. of concrete $w_c = 150 \text{ lbs. per cu. ft.}$

$f_c' = 3000 \text{ psi} \quad f_c = 1350 \text{ psi}$

$v_c = 40 \text{ psi} \quad R = 248$

$f_s = 18000 \text{ psi}$

1890. 1891. 1892. 1893. 1894. 1895. 1896. 1897. 1898. 1899. 1900.

1901. 1902. 1903. 1904. 1905. 1906. 1907. 1908. 1909. 1910. 1911.

1912. 1913. 1914. 1915. 1916. 1917. 1918. 1919. 1920. 1921. 1922.

1923. 1924. 1925. 1926. 1927. 1928. 1929. 1930. 1931. 1932. 1933.

1934. 1935. 1936. 1937. 1938. 1939. 1940. 1941. 1942. 1943. 1944.

1945. 1946. 1947. 1948. 1949. 1950. 1951. 1952. 1953. 1954. 1955.

1956. 1957. 1958. 1959. 1960. 1961. 1962. 1963. 1964. 1965. 1966.

1967. 1968. 1969. 1970. 1971. 1972. 1973. 1974. 1975. 1976. 1977.

1978. 1979. 1980. 1981. 1982. 1983. 1984. 1985. 1986. 1987. 1988.

1989. 1990. 1991. 1992. 1993. 1994. 1995. 1996. 1997. 1998. 1999.

2000. 2001. 2002. 2003. 2004. 2005. 2006. 2007. 2008. 2009. 2010.

2011. 2012. 2013. 2014. 2015. 2016. 2017. 2018. 2019. 2020. 2021.

2022. 2023. 2024. 2025. 2026. 2027. 2028. 2029. 2030. 2031. 2032.

2033. 2034. 2035. 2036. 2037. 2038. 2039. 2040. 2041. 2042. 2043.

2044. 2045. 2046. 2047. 2048. 2049. 2050. 2051. 2052. 2053. 2054.

2055. 2056. 2057. 2058. 2059. 2060. 2061. 2062. 2063. 2064. 2065.

2066. 2067. 2068. 2069. 2070. 2071. 2072. 2073. 2074. 2075. 2076.

2077. 2078. 2079. 2080. 2081. 2082. 2083. 2084. 2085. 2086. 2087.

2088. 2089. 2090. 2091. 2092. 2093. 2094. 2095. 2096. 2097. 2098.

2099. 2100. 2101. 2102. 2103. 2104. 2105. 2106. 2107. 2108. 2109.

Abutments (cont.)

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

$$P_1 = 28.7 \times 15.73 = 452 \text{ psf}$$

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

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

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

Base pressures; moments about toe;

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

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

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

$$\Sigma W = 13460 \text{ lbs.}$$

$$M_R = 78470 \text{ ft.-lbs.}$$

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;

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

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

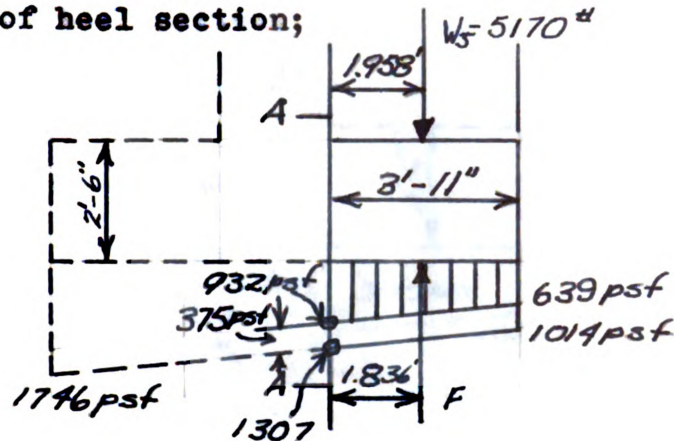
Safety factors;

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

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

Abutments (cont.)

Check design of heel section;



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

$$x = 293 \text{ psf}$$

$$1014 + 293 = 1307 \text{ psf}$$

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

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

Max Shear and Moment on Section A A;

$$W_3 = -5170 \text{ lbs; } x \ 1.958 = -10,100 \text{ ft.-lbs.}$$

$$\text{earth } P = \frac{+3090 \text{ lbs; } x \ 1.835 = +5,850 \text{ ft.-lbs.}}{V = \frac{2080 \text{ lbs}}{M = \frac{4,250 \text{ ft.-lbs.}}{2080 \text{ lbs}}}}$$

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

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

$$d \text{ furnished} = 27.0 \text{ inches}$$

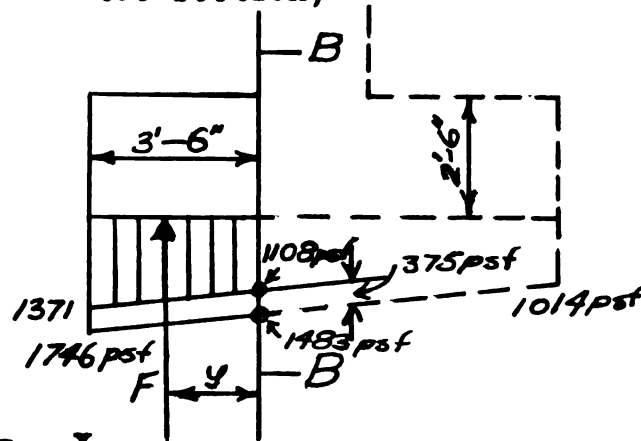
$$R = \frac{M}{b d^2} = \frac{4250 \times 12}{1 \times (27)^2} = 5.9 < 248 \therefore f_c < 1350 \text{ psi}$$

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

$$A_s = \text{furnished} = .44 \text{ sq. in./ft.}$$

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;

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

$$M = 4330 \times 1.855 = 8050 \text{ ft.-lbs.}$$

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

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

$$d \text{ furnished} = 27.0 \text{ inches}$$

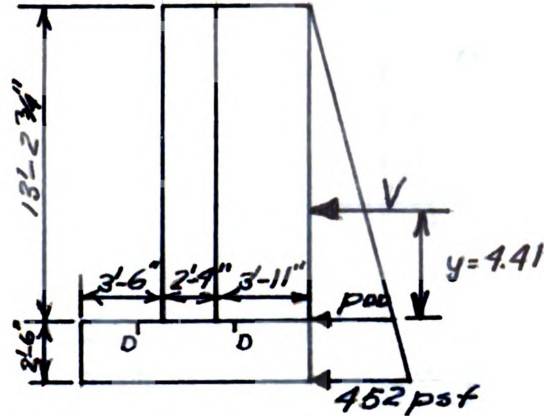
$$R = \frac{M}{bd^2} = \frac{8050}{1 \times (27)^2} = 11.2 < 248 \therefore f_c < 1350 \text{ psi}$$

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

$$A_s \text{ furnished} = .60 \text{ sq. in./ft.}$$

Abutments (cont.)

Check design of stem;



$$\frac{452}{15,728} = \frac{p_{oo}}{13.24}$$

$$p_{oo} = 381 \text{ psf}$$

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

Max Shear and Moment on Section D D;

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

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

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

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

$$d \text{ furnished} = 25.0 \text{ inches}$$

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

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

$$A_s \text{ furnished} = .79 \text{ sq. in./ft.}$$

Abutments(cont.)

$$p = 28.7 \text{ h}$$

$$P_1 = 28.7 \times 23.88 = 686 \text{ psf}$$

$$P_2 = 28.7 \times 8.15 = 234 \text{ psf}$$

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

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

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

Base pressures; moments about toe;

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

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

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

$$\begin{aligned} W &= 3010 \text{ lbs; } \times (3.5 + 1.17) = 14050 \text{ ft.-lbs.} \\ \Sigma W &= 20660 \text{ lbs; } M_R = 125120 \text{ ft.-lbs.} \end{aligned}$$

Location of Resultant;

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

$$e = 4.87 - 3.76 = 1.11 \text{ to left of center}$$

toe and heel pressures;

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

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

Safety factors;

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

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

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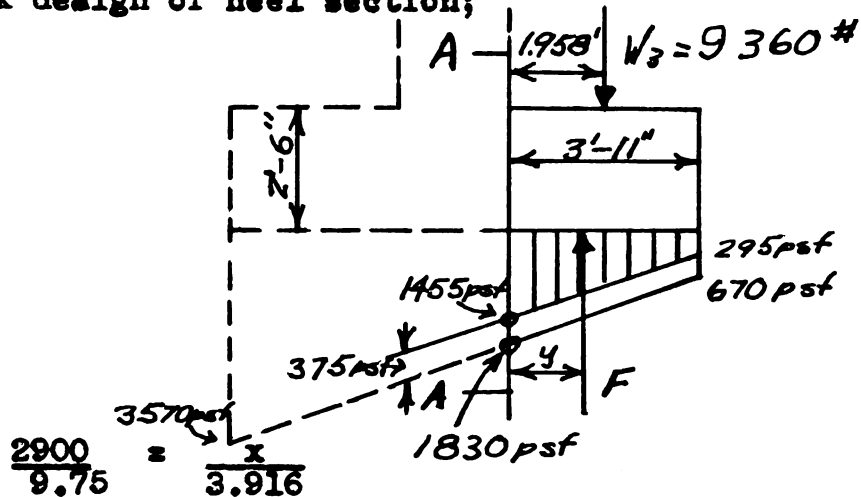
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Abutments (cont.)

Check design of heel section;



$$x = 1160 \text{ psf}$$

$$670 + 1160 = 1830 \text{ psf}$$

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

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

Max Shear and Moment on section A A;

$$W_3 = -9360 \text{ lbs; } x \ 1.958 = -18310 \text{ ft.-lbs.}$$

$$\text{earth } P = +3420 \text{ lbs; } x \ 1.525 = +5210 \text{ ft.-lbs.}$$

$$V = 5940 \text{ lbs; } M = 13100 \text{ ft.-lbs.}$$

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

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

d furnished - 27.0 inches

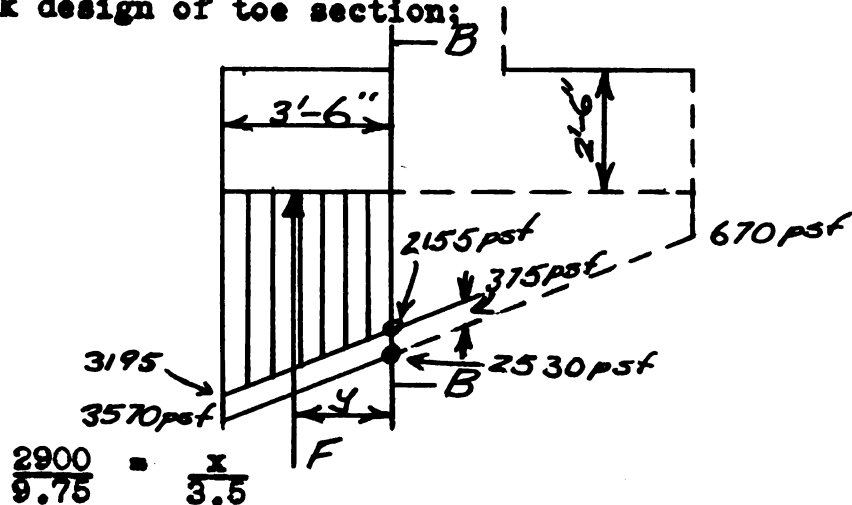
$$R = \frac{M}{bd^2} = \frac{13100}{1 \times (27)^2} = 18 < 248 \therefore f_c < 1350 \text{ psi}$$

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

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

Abutments (cont.)

Check design of toe section;



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

$$x = 1040 \text{ psf}$$

$$3570 - 1040 = 2530 \text{ psf}$$

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

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

Max Shear and moment on Section B B;

$$V = 9380 \text{ lbs.}$$

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

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

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

$$d \text{ furnished} = 27.0 \text{ inches}$$

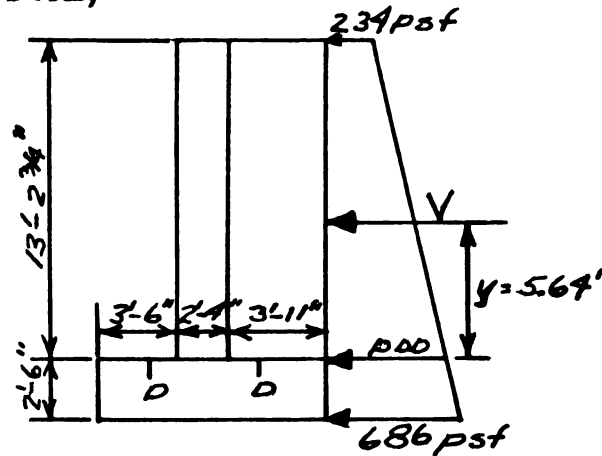
$$R = \frac{M}{b d^2} = \frac{19350}{1 \times (27)^2}$$

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

$$A_s \text{ furnished} = .60 \text{ sq. in./ft.}$$

Abutments (cont.)

Check design of stem;



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

$$p_{DD} = 614 \text{ psf}$$

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

Max Shear and Moment on Section D D;

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

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

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

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

$$d \text{ furnished} = 25.0 \text{ inches}$$

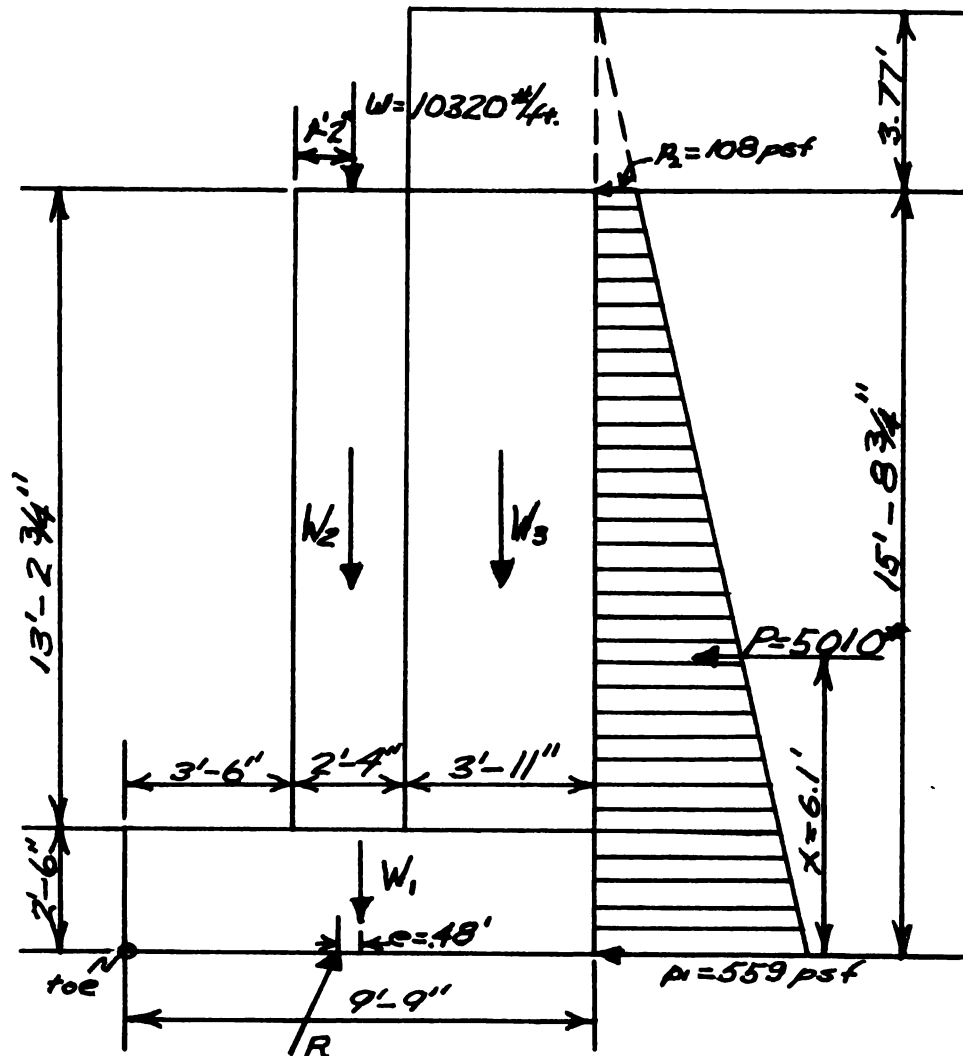
$$R = \frac{31600}{1 \times (25)^2} = 50.5 < 248 \therefore f_c < 1350 \text{ psi}$$

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

$$A_s \text{ furnished} = .79 \text{ sq. in./ft.}$$

Abutments (cont.)

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



Superstructure dead load

$$\text{D.L.} = 3010 \text{ lbs./ft.}$$

$$\text{Max L.L. per stringer} = 29200 \text{ lbs}$$

$$29200 \times 9 \text{ stringers} = 263000 \text{ lbs}$$

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

Abutments (cont.)

$$p = 28.7 \text{ h}$$

$$P_1 = 28.7 \times 19.5 = 559 \text{ psf}$$

$$P_2 = 28.7 \times 3.77 = 108 \text{ psf}$$

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

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

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

Base pressures; Moments about toe;

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

$$W_2 = 2.33 \times 3.23 \times 150 = 4630 \text{ lbs; } \times \frac{(3.5 - 2.133)}{2} = 20350 \text{ ft.-lbs.}$$

$$W_3 = 3.92 \times 19.50 \times 100 = 7650 \text{ lbs; } \times (5.834 - 3.917) = 59500 \text{ ft.-lbs.}$$

$$W = \frac{10320 \text{ lbs;}}{\Sigma W = 26,260 \text{ lbs}} \times 4.67 \quad M_R = \frac{48100 \text{ ft.-lbs.}}{145770 \text{ ft.-lbs.}}$$

Location of Resultant;

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

$$e = 4.87 - 4.39 = .48 \text{ ft. to left of center}$$

Toe and heel pressures;

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

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

Safety factors;

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

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

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• 1. The first step is to identify the problem. This involves understanding the current situation and the desired outcome. It is important to gather all relevant information and to define the problem clearly.

• 2. The second step is to analyze the problem. This involves breaking the problem down into smaller, more manageable parts. It is important to identify the causes of the problem and to determine the best way to solve it.

• 3. The third step is to develop a solution. This involves creating a plan of action that addresses the problem. It is important to consider all possible solutions and to choose the one that is most effective.

• 4. The fourth step is to implement the solution. This involves putting the plan of action into practice. It is important to monitor the progress of the solution and to make adjustments as needed.

• 5. The fifth step is to evaluate the solution. This involves assessing the effectiveness of the solution and determining whether it has solved the problem. It is important to gather feedback from others and to reflect on the process.

• 6. The sixth step is to communicate the solution. This involves sharing the solution with others and explaining how it was developed. It is important to be clear and concise in your communication.

• 7. The seventh step is to document the solution. This involves creating a record of the solution and the process used to develop it. This record can be used for future reference and to help others solve similar problems.

• 8. The eighth step is to review the solution. This involves looking back at the solution and the process used to develop it. It is important to identify what worked well and what could be improved.

• 9. The ninth step is to share the solution. This involves sharing the solution with others and explaining how it was developed. It is important to be clear and concise in your communication.

• 10. The tenth step is to implement the solution. This involves putting the plan of action into practice. It is important to monitor the progress of the solution and to make adjustments as needed.

• 11. The eleventh step is to evaluate the solution. This involves assessing the effectiveness of the solution and determining whether it has solved the problem. It is important to gather feedback from others and to reflect on the process.

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• 12. The twelfth step is to communicate the solution. This involves sharing the solution with others and explaining how it was developed. It is important to be clear and concise in your communication.

• 13. The thirteenth step is to document the solution. This involves creating a record of the solution and the process used to develop it. This record can be used for future reference and to help others solve similar problems.

• 14. The fourteenth step is to review the solution.

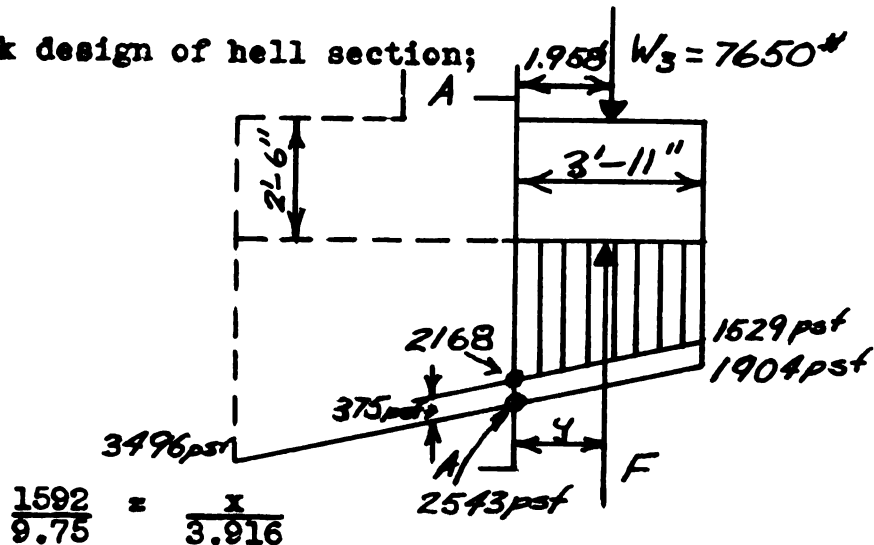
• 15. The fifteenth step is to share the solution.

• 16. The sixteenth step is to implement the solution. It is important to monitor the progress of the solution and to make adjustments as needed.

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Abutments (cont.)

Check design of heel section;



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

$$x = 639 \text{ psf}$$

$$1904 + 639 = 2543 \text{ psf}$$

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

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

Max Shear and Moment on Section A A;

$$W_3 = -7650 \text{ lbs; } x \ 1.958 = -15000 \text{ ft.-lbs.}$$

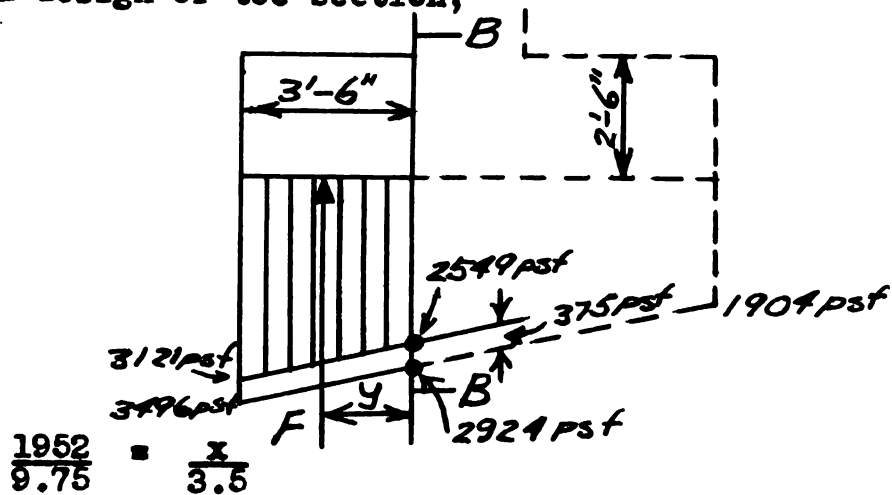
$$\text{earth } P = +7420 \text{ lbs; } x \ 1.845 = +13700 \text{ ft.-lbs.}$$

$$V = 230 \text{ lbs; } M = 1300 \text{ ft.-lbs.}$$

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

Abutments (cont.)

Check design of toe section;



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

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

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

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

Max Shear and Moment on Section B B;

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

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

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

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

$$d \text{ furnished} = 27.0 \text{ inches}$$

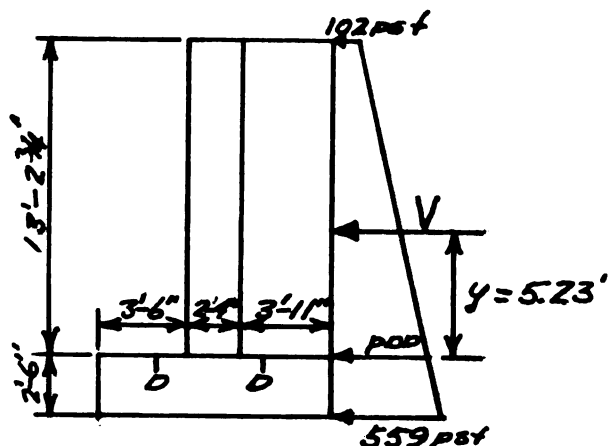
$$R = \frac{18600}{1 \times (27)^2} = .526 \text{ sq. in./ft}$$

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

$$A_s \text{ furnished} = .60 \text{ sq. in./ft.}$$

Abutments (cont.)

Check design of stem;



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

$$P_{DD} = 489 \text{ psf}$$

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

Max Shear and Moment on Section D D;

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

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

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

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

$$d \text{ furnished} = 25.0 \text{ inches}$$

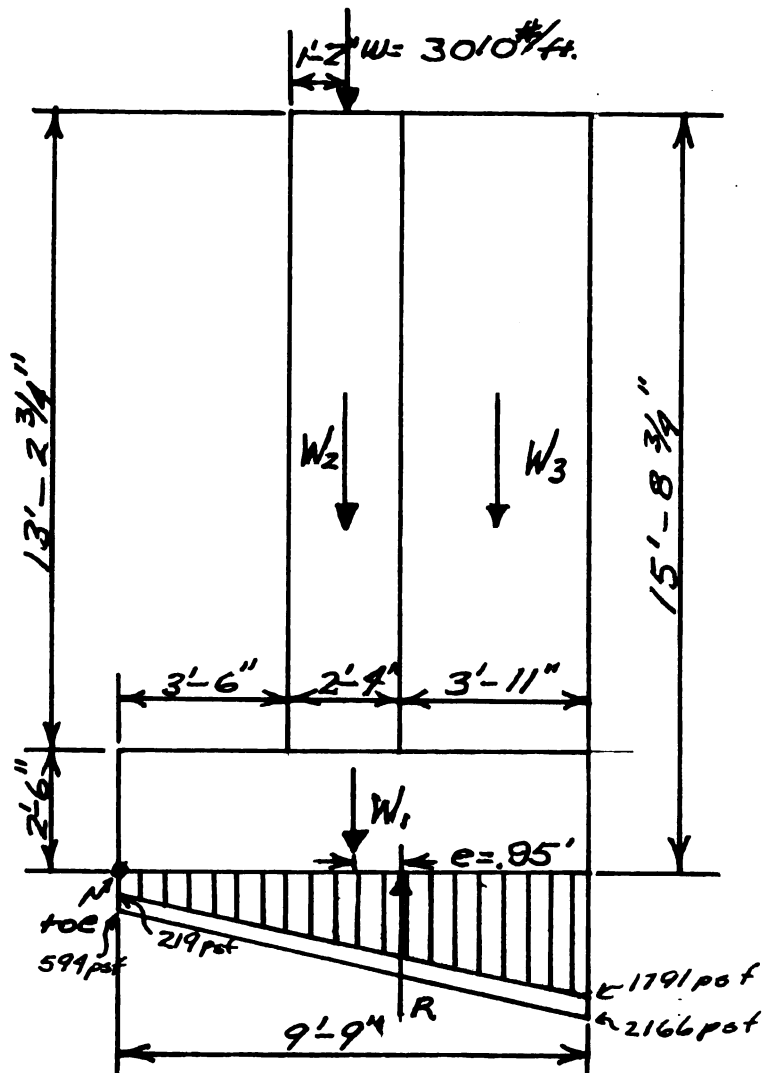
$$R = \frac{M}{bd^2} = \frac{20500}{1 \times (25)^2} = 32.8 < 248 \therefore f_c < 1350 \text{ psi}$$

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

$$A_s \text{ furnished} = .79 \text{ sq. in./ft.}$$

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 wall assume no horizontal force therefore there will be no moment cause overturning.

Abutments (cont.)

Base pressures; moments about toe;

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

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

$$W_3 = 3.92 \times 13.23 \times 100 = 5170 \text{ lbs; } \times (5.834 - 3.917) = 40300 \text{ ft.-lbs.}$$

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

$$W = 16470 \text{ lbs.}$$

$$M = 92520 \text{ ft.-lbs.}$$

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;

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

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

As there is no horizontal 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 horizontal force there will be no action on the vertical stem.

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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, the savings in the use 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.

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