A CHECK OF THE STRESSES IN THE PRINCIPAL MEMBERS OF THE STATE HIGHWAY BRIDGE AT ROCHESTER, MICH

THESIS FOR THE DEGREE OF B. S.

Homer McVean

1930

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Strains + stresses

Bridges

Civil engineering - Bridges + roofs

A Check of the Stresses in the Principal Members

of the State Highway Bridge

at Rochester, Michigan

A Thesis Submitted to

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Extracts of the articles from the specifications of the Michigan State Highway Department used in the check:

3.9 curbs. The width of the curb measured from face of the curb to closest vertical projection or the railing pilaster, of of the superstructure, shall preferably be not less than 1'-6" as a safety zone for occasional pedestrians. In no case shall this width be less than 1'-0". The curb height shall preferably be not less than 10 inches and in no case less than 9 inches. Curbs shall be of substantial construction capable of resisting a transverse force of not less than 500 lbs. per lineal foot, applied at top of curb.

3.10 Railings

3.10.1 Railing height. Substantial railings shall be provided along each side of the bridge for the protection of traffic. The top of the railing shall be not less than 3'-0" above top of curb, same height above sidewalks.

3.10.3 Design forces. Railings shall be designed to resist a horizontal force of not less than 150 lbs. per lineal foot, applied at top of the rail, and a vertical force of not less than 100 pounds per lineal foot.

3.14 Crown. Height of crown shall be not less than given by the formula $C = \frac{R^2}{833}$ where $C =$ crown in inches and $R =$ Roadway width in feet. Minimum crown shall be one inch.

Loads

	Weight/ cu. ft. in lbs.
Steel	490
Concrete, reinforced	150
Earth	100

System of loading specified on plans.

Pressure from retained material.

Equivalent liquid theory (HooC)

$P = \frac{1}{2} wh^2 c (h+2h_1)$

$y = \frac{h^2 + 3hh_1}{3(h+2h_1)}$

P = resultant earth pressure per foot length of wall

h = vertical height of wall in feet.

c = A coefficient, taken as .25

h_1 = height of surcharge in feet

Section 7 unit Stresses

7.2.1 Structural grade and rivet steel

7.2.1.1 Tension, lbs. per square inch.

axial tension, structural members, net section, 16000

7.211.2 Axial compression, gross section $15000 - 50 \frac{L}{r}$ but not to exceed 13,500 lbs.

L = length of member in inches

r = least radius of gyration in inches

7.2.1.3 Bending an extreme fiber

Rolled shapes, built sections and girders

	net section	16,000
pins		24,000

7.2.1.5 Shear

Girder web, gross section	10,000
Pins and shop driven rivets	12,000
Power driven rivets and bolts	10,000

7.2.1.6 Bearing

Pins, steel parts in contact and shop driven rivets 24,000

Power driven field rivets and bolts	20,000

7.4.1 Assumed Compressive Strengths for structural Grades Portland Cement concrete

Grade	28 day strength
A	2500
B	2000
C	1500

Grade A. All superstructure concrete

Grade B. Reinforced concrete structures except superstructures calculated as structural beams and slab units, and all exposed concrete of 18" thickness or less.

7.4.4. Columns with axial loading

7.4.4.1 Short columns, $\frac{h}{D} = 12$ or less where h = unsupported length; D = least outside diameter without spiral reinforcement.

Columns with longitudinal reinforcement and ties.

Allowable unit stress of total concrete area = $.20\ f^{1}c$

9.1.2 Design loads. Preferably, structures shall be proportioned to limit the maximum design load on timber piles to 15 tons per pile.

911.3 Spacing. Footing areas shall be so proportioned that pile spacing shall not be less than 2'-6" center to center. The distance from the center of any pile to the nearest edge of the footing shall be not less than 12 inches. In the design of pile foundations, the effect of eccentric loadings due to earth thrust shall be considered and the pile spacings shall generally be designed so as to require a minimum number of piles. The distribution of the foundation loading shall be computed as follows:

P = Load in tons per lineal foot of footing

S = Spacing, in feet, for any row of piles

$n = 1/5$ = Number of piles per foot in any row.

e = Eccentricity of loading

L = Width of footing in feet

a = Distance in feet from heel of footing to any row of piles.

d_o = Settlement in inches of heel of footing

k = Change in amount of settlement per foot width of footing

d = Settlement in inches of any pile under load

B = Load in tons per pile, to cause a settlement of one inch

p = Load in tons, on any single pile

r = Distance, in feet, from heel of footing to locus of the center of gravity of the piles.

Then $d = d_o + ka$

$Sn(d_o + ka) = \frac{P}{B} + L(e + \frac{L}{2})$

From Which $Bd_o = \frac{PSna^2 - L(e - \frac{L}{2})Sna}{SnSna^2 - (Sna)^2}$

$Bk = \frac{PL(e + \frac{L}{2})Sn - Sna}{Sn\, Sna^2 - (Sna)^2}$

$p = Bd_o + aBk$ and $r = \frac{Sna}{Sn}$

9.3.9 Counterfort abutments

Counterforts shall preferably be located under or near points of concentrated load. The face walls and the back of the base shall be designed as continuous slabs. For equally spaced counterforts, positive and negative moments of $1/12\ w\ l^2$ shall be assumed, provided that the end supports for the series of continuous slabs offer suitable restraint; otherwise $1/10\ w\ l^2$ shall be assumed for positive and negative moments.

The rear of the base slab shall be investigated for diagonal tension and bond stresses at the edge of the counterforts.

The toe projection shall be considered as cantilever beams fixed at the edge of the support, and shall be designed for the foundation reaction less the weight of the toe.

10.6. Plate Girders

10.6.2 Flange sections. The gross section of the compression flange shall not be less than the gross section of the tension flange. The compression flange shall be stayed against lateral deflection at intervals not exceeding 12 times its width. Flange cover plates shall be equal in thickness, nor have a thickness greater than that of the flange angles.

10.6.3. Web plates shall be proportioned for both horizontal and vertical shear. Splices in web plates shall be designed to develop the full value of the web plate for both bending and shearing stresses. The thickness of web plates shall not be less than $1/20 \sqrt{D}$, where D=distance between flanges in inches.

10.6.7. End Stiffness. Plate girders shall have stiffener angles over end bearings the outstanding legs of which shall be as wide as the flange angles will allow and shall fit tightly against them. These end stiffeners shall be proportioned for bearing on the outstanding legs of the flange angles, no allowance being made for the legs fitted to the fillets of the flange angles.

10.6.8. Intermediate stiffeners angles shall be riveted in pairs to the web of the girder. Intermediate stiffeners shall be spaced at intervals not exceeding:

(a) 6 feet

(b) the depth of the web

(c) The distance given by the formula $d = \frac{t}{40}(12000-S)$

Design of 70' unencased Girders

Dimensions

Web Plate 46x3/8"

Flanges, 2 angles 6"x6"x11/16"

Depth of Girder 46½"

Check web plate thickness

t should not be less than 1/20 D

$t = 1/20 \sqrt{46} = .34"$; $t = 3/8"$ is O. K.

Design of Flange

Area required; $A = \frac{M}{Sh} - 1/8\ th_1$

h_1 = depth of web in inches

h = distance between centers of gravity of top and bottom flanges

$h_1 = 46\frac{1}{2} - 3\frac{1}{2} = 43"$

$A = \frac{933000 \times 12}{16000 \times 43} - 1/8 \times 46 \times 3/8 = 16.25 - 2.15 = 14.10$ sq. in.

net area = gross area minus rivet holes

Gross area of angles = 15.56

Gross Area = 15.56 - 2x1x3/8 = 14.81 sq. in.

area required is 14.10 sq. in.

Section Modulus, 70' unencased girders

Required, $\frac{M}{S} = \frac{933000 \times 12}{16000} = 700\ in^3$

Provided

$I = 2\ (1/3 \times 3/8 \times (23)^3) + 4(26.2) + 4(7.78 \times 21.5^2)$

$I = 3042 + 104.8 + 14350 = 17497\ in.^4$

$\frac{I}{C} = \frac{17497}{23.25} = 755\ in.^3$ O. K.

End Stiffeners

4 angles 6"x3½"x3/8"x45 1/8

Area of outstanding legs = 7.00 sq. in.

$S = 15000 - 50\frac{L}{r}$

$r = \sqrt{\frac{I}{A}} = 1.15$

$S = 15000 - \frac{\cancel{50}x45\ 1/8}{1.15} = 13,000$

Use 12,500 (specifications)

Area required $= \frac{54,075}{12,500} = 4.33$ sq. in.
O. K.

Spacing of intermediate stiffeners s

Specifications

(a) 6 feet

(b) The depth of the web

(c) $d = \frac{t}{40}$ (12000 - S) where s = web shear in lbs. per sq. in.

$S = \frac{54,075}{46x3/8} = 3140$

(b) = 46"

)b) = d = 3/8 x 1/40 (12000-3140) = 88"

The stiffeners are spaced at 36" which is o. k.

Spacing of rivets in flange angles, same as in 70' encased girders.

Design of 70' Encased Girders

Dimensions

Web Plate 46"x½"

2 Flange angles 6"x6"x5/8"

1 cover plate 14"x¾"

Total depth of girder = 48"

Gross area Flange 14.22 sq. in.

" " Plate 10.50 " "

25.72 " "

Net area Flange 14.22-4x1x5/8 = 11.72 sq. in.

" " Plate 10.50-2x1x¾ = 9.00 " "

20.72 " "

Distance between centers of gravity of flanges

$\frac{10.5}{24.72}$ (1.73 + .375) = .53

$h = 48-2(1.73-.53) = 45.6"$

Area required $= \frac{1,344,000x12}{16,000x45.6} - 1/8 \times 46 \times \frac{1}{2} = 17.05$ sq. in.

Area required is less than net area provided.

Design of Stiffeners

2 angles 5x3½x3/8x45 1/8

2 angles 4x6x3/8x45 1/8

area of outstanding legs = 8.75 sq. in.

area required $= \frac{77950}{12500} = 6.24$ sq. in.

area required for stiffeners is less than area provided.

Spacing of stiffeners

Specifications same as in 70' girder

Spacing is 36"

$d = \frac{t}{40}\ (12000-S)$

$S = \frac{77950}{46x\frac{1}{2}} = 3400$

$d = \frac{.5}{40}\ (8600) = 107" = 9$ foot

36" spacing O. K.

Section modulus 70' encase girder

$\frac{I}{C} = \frac{M}{S} = \frac{1,344,000x12}{16,000} = 1005$ in.^{3} required

Section Modulus provided

$I = 2(1/3 \times \frac{1}{2} \times 12167) + 4(24.2) + 4(7.11x21.5^2) + 1/12 \times 14(\frac{3}{4})^3 + 2(10.5x23.43^2)$

$I = 4050 + 96.8 + 13,146 + .5 + 11550 = 28845$

$\frac{I}{C} = \frac{28845}{24.0} = 1210$ in^3 O. K.

Pitch of rivets in flange angles

70' encased girder

$$P = \frac{R}{\sqrt{\left(\frac{V}{h}\right)^2 + m^2}}$$

P = allowable pitch in inches at section under consideration

R = allowable stress on rivet in pounds

h = distance between centers of gravity of flanges in inches

V = maximum external shear on given section in lbs.

m = load per lineal inch supported directly by flange.

1 rivet in double shear = .6x12000x2 = 14,400

1 rivet in bearing = $\frac{1}{2}$ x 7/8 x 24,000 = 10,500

V = maximum live and dead shear = 77,950

h = 45.6"

$\frac{w}{12}$ $M = 1/8\ wl^2$ $w = \frac{870000}{(70)^2} \times 8 = 1440 - 300 = 1144$
300 = wt. of girder

$$P = \frac{10{,}500}{\sqrt{\left(\frac{77{,}950}{45.6}\right)^2 + \left[\left(\frac{1140}{12}\right) + \left(\frac{12{,}500}{36}\right)\right]^2}}$$

12,500# = largest wheel load distributed over 3 ft.

This is largest shear, so all spacing is O. K.

All spacing is 6" or less

Design of 65' girders

Dimensions

Web 46" x 3/8"

Flanges 2 ∠s 6 x 6 x 5/8

Gross area of flange ∠s = 14.22

net area of flange = 14.22-2x1x5/8 = 12.97 Sq. in.

h = dist. between c.g. of flanges = 46.5 - 2(1.73) = 43.04

Area reqd. $= \frac{815000 \times 12}{16000 \times 43.04} - \frac{1}{8} \times 46 \times \frac{3}{8} = 14.2 - 2.15 = 12.05$ sq. in.

Area of flanges O. K.

Stiffeners same as 70' girders

Pitch of rivets 6"

Design of 50' Girders

Dimensions

Web 46" x 3/8"

Flanges 2 ∠s 6 x 4 x 3/8"

Gross area of flange ∠s = 7.22 sq. in.

Net area of flange = 7.22 - 2 x 1 x 3/8 = 6.47 sq. in.

h = distance between c. g. of flanges = 46.5 - 2(.94) = 44.62

Area req. $= \frac{500,000 \times 12}{16,000 \times 44.62} - \frac{1}{8} \times 46 \times \frac{3}{8} = 6.25$ sq. in.

Area is O. K.

Stiffeners are of same dimensions and spacing as on 70' girders

Pitch of flange rivets 6"

Section modulus of 65' girders

Required; $\frac{M}{S} = \frac{815000}{16000} = 610$ in.^{3}

Provided

$I = 3042 + 96.8 + 13,146 = 16285$ in.^{4}

$\frac{I}{c} = \frac{16285}{23.25} = 705$ in.^{3} O. K.

Section modulus of 50' girders

Required $\frac{500,000}{14,000} = 375$ in.^{3}

Provided

$I = 3042 + 4(4.9) + 4(3.61 \times 22.31^2) = 3062 + 19.6 + 7200 = 10242$ in.^{4}

$\frac{I}{c} = \frac{10262}{23.25} = 440$ in.^{3} O. K.

Design of box girder C7

Dimensions

2 web plates 72" x 3/8

2 flange angles 8" x 8" x 5/8"

2 plates 8" x $\frac{1}{2}$"

Gross area Flange		Net area
19.22	$- 2 \times 1 \times 5/8$	17.97
8.00	$- 2 \times 1 \times \frac{1}{2}$	7.00
		24.97 in^2

Area required

$\frac{2,663,000 \times 12}{16,000 \times 87.2} - 1/8 \times 72 \times 6/8 = 29.80 - 6.75 = 23.05$ $in.^2$

Area provided is sufficient

Section Modulus

Required $\frac{M}{S} = \frac{2,663,000}{16,000} \times 12 = 1995$ $in.^3$

Provided

Dimensions of G7 same as those of G6, so section modulus required is 2340 $in.^3$

Design of box girder G9

Dimensions and area of flange

2 web plates 72" x 3/8"

2 flanges angles 8" x 8" x $\frac{3}{4}$"

2 plates 8 x $\frac{1}{2}$

Gross area flange		net area
22.88	$- 2 \times 1 \times \frac{3}{4}$	21.38 in^2
8.00	$- 2 \times 1 \times \frac{1}{2}$	& 7.00
		28.38 in^2

Thickness of web plate not less than $1/20 \sqrt{D}$; $t = 1/20 \sqrt{72} = .425"$ 6/8" O. K.

or $t = \frac{357000}{10000 \times 72} = .497"$ O. K.

Design of flange

Area required $= \frac{M}{Sh} - 1/8\ th_1$

$h = \frac{2(77.8 \times 33.97 + 27.2 \times 32.25)}{21.6 + 77.8} = 67.2"$

$h_1 = 72"$

Area of flange required

$A = \frac{3,042,000 \times 12}{16000 \times 67.2}$ $- 1/8 \times 72 \times 6/8 = 27.12$ " which is sufficient

Section modulus box girder G9

Required $\frac{M}{S} = \frac{3042000 \times 12}{16000} = 2280\ in^3$

Provided

$I = 4(1/3 \times 3/8 \times 36^3) + 4(69.7) + 4(11.44 \times 33.97^2) + (1/12 \times \frac{1}{2} \times 8^3)4$
$+ (4 \times 32.25^2)$

$I = 23328 + 278.8 + 52600 + 16640 + 85 = 92932\ in.^4$

$\frac{I}{c} = \frac{92932}{36.25} = 2570\ in.^3$ O.K.

Stiffeners

2 angles $6 \times 3\frac{1}{2} \times \frac{3}{8}$ — area of outstanding legs $= 6$ inches^2

71" long

$v = 16000 - 120\ d/t$

$\frac{357000}{71 \times 3/8} = 16000 = 120\ d/t$

$\frac{13200 - 16000}{120} \times 3/8 = \frac{2300}{120} \times 3/8 = 9"$

Spaced 9 inches or 18" on each of the two web plates.

This checks with the design

Number of rivets in hitch /s for Box Girders

These are spaced the same in all of the Box Girders. Therefore, if the pitch is O. K. for the one bearing the largest load, the rest which carry less must be O. K. as bearing controls in each case and the web Pl. thickness is the same for each –3/8".

1 7/8" rivet in double shear $= .6 \times 2 \times 12000 = 14,400$

1 7/8" rivet in shear $= 7/8 \times 24000 \times 3/8 = 7,875$ "

$$P = \frac{7.875}{\sqrt{\left(\frac{178,750}{67.38}\right)^2 + \left(\frac{12500}{86}\right)^2}} = \frac{7.875}{2600} = 3"$$

P = 3" this checks closely with the design

Design of web splice box girder G. 9 #1

4 plates, 18" x 3/8" x 56"

rivets spaced 12 at 4" 1 @ 8" = 14 rivets

net area of splice pls.

4(3/8 x 56 - 14 x 3/8 x 1) = 63.00 sq. in.

net area of web plate = 72" x 2 x 3/8" = 54 sq. in.

The area of splice plates is greater than web, therefore the splice place are sufficient for shear.

Bending moment splice plates can carry.

$$M = \frac{bd^2s}{6} = \frac{16000}{6} \times 4(3/8) \times (56)^2 \tfrac{3}{3} = 9,250,000 \text{ in lbs.}$$

Bending moment web plates can carry

$$A = \frac{M}{sh} - \frac{th}{8}$$

$$M = \frac{6/8 \times 72}{8} \times 16000 \times 67.2 = 7,250,000 \text{ in lbs.}$$

Splice plates can carry more bending moment than web plates, so are sufficient for bending moment.

Check of rivets in web splice #1

1 rivet double shear = 14,400 # sq. in.

1 rivet bearing = 3/8 x 1 x 24000 = 9000# sq. in.

Bearing governs

Shear web can take

72 x 2 x 3/8 x 10000 = 540,000

6 rows rivets @ 14 = 82 rivets

$$\text{vertical stress} = \frac{540000}{82} = 6,580 \text{ lbs.}$$

Horizontal stress $= \sqrt{9000^2-6,580^2} = 6050$ lbs.

$\frac{6050}{54.5}(1.5^2+6.5^2+10.5^2+14.5^2+18.5^2+\text{------}54.5^2)2 = 3,330,000$

Resistance of one row of rivets

$\frac{7,250,000}{3,330,000} = 2+$ 3 row rivets is sufficient

Design of web splice G9, #2 splice

4 plates 18" x 3/8" x 52"

Rivets spaced, 8 at 5" 2 at $4\frac{1}{2}$"   11 rivets

net area of splice plates

$4(3/8 \times 52 - 11 \times 3/8 \times 1) = 61.52$ sq. in.

net area of web plate $= 2(69) \times 3/8 = 52.$ sq. in.

net area of splice plate is sufficient

Bending moment splice plates can carry

$\frac{16000}{6} \times 4(3/8) \times (52)^2 \times \frac{3}{4} = 8,200000$ in lbs.

Bending moment web plates can carry

$M = \frac{6/8 \times 69}{8} \times 16000 \times 64.2 = 6,650000$ in lbs.

Splice plates sufficient to carry bending moment

Check of rivets in web splice #2

6 rows @ 11 = 66

shear web plates can carry

$69 \times 2 \times 3/8 \times 10000 = 520000$ lbs.

$\frac{520000}{66} = 7860$ lbs.

Horizontal stress $= \sqrt{9000^2-7860^2} = 4,400$ lbs.

$\frac{4,400}{51.5}(1\frac{1}{2}^2+6^2+11^2+16^2+21^2+26^2+31^2+36^2+41^2+46^2+51.5^2)2 = 1750000$

Resistance one row of rivets

$\frac{6,650,000}{1750,000}$ 4 rows sufficient

Web splices in all box girders are the same.

Dimensions of box girder G8

2 web plates 72" x 3/8"

2 flange angles 8" x 8" x 5/8"

2 plates 8" x ½"

area of flanges

gross	net
19.22	17.97
8.00	7.00
27.22 sq. in.	24.97 sq. in.

area required

$$A = \frac{2,821,000}{16000 \times 67.2} - 6.75 = 24.68 \text{ sq. in.}$$

Area of flange is O. K.

Section modulus box girder G8

Required:

$$\frac{M}{S} = \frac{2,821,000}{16,000} \times 12 = 2120 \text{ in}^3$$

Provided

$$I = 4(1/3 \times 3/8 \times 36^3) + 4(58.4) + 4(9.61 \times 34.03^2) + (\tfrac{1}{12} \times \tfrac{1}{2} \times 8^3)4 + 4(4 \times 32.25^2)$$

$$I = 23328 + 237.6 + 44500 + 85 + 16640 = 84780 \text{ in.}^4$$

$$\frac{I}{C} = \frac{84780}{36.25} = 2340 \text{ in}^3 \text{ O. K.}$$

Stiffeners and pitch of rivets same as G9

Design of box girder G2

Dimensions

2 web plates 72" x 3/8"

Flange angles 8" x 8" x 5/8

Gross area flange angles	net area
19.22 sq. in	17.97 sq. in.

Area required

$A = \frac{2,127,000 \times 12}{16000 \times 68.04} - 6.75 = 16.70$ sq. in.

Area o. k.

Spacing of stiffeners for G7

$6 \times 3\frac{1}{2}'' \times \frac{1}{2}''$ angles

$v = 16000 - 120\ d/t$

$\frac{228000}{71 \times 3/8} = 16000 - 120\ D/t$

$D = \frac{16000 - 8,800 \times 3/8}{120} = 22.5''$

Stiffeners spaced 45" on each plate which check with design

Rivet spacing same as G8

Design of Girder G1

Dimensions

2 web plates $72'' \times \frac{3}{4}''$

4 flange angles $8'' \times 8'' \times \frac{3}{4}''$

area of flange

Gross area 2 $\angle$s	net area
22.88 sq. in.	21.38 sq. in.

Area required

$A = \frac{2,423,000 \times 12}{16000 \times 67.94} = 26.75 - 6.75 = 20.00$ sq. in.

Section modulus G1

Required

$M = \frac{2,423,000 \times 12}{16000} = 1820\ in^3$

Provided

$I = 4(1/3 \times \frac{3}{4} \times 36^2) + 4(69.7) + 4(11.44 \times 33.97^2) = 23328 + 278.8 + 52600 = 76207\ in.^4$

$\frac{I}{C} = \frac{76207}{36.25} = 2110\ in^3$ O. K.

Section Modulus G 2

Required $\frac{M}{S} = \frac{2,127,000 \times 12}{16000} = 1595\ in^3$

Provided $I = 4(1/3 \times 3/8 \times 36^3) + 4(5.96) + 419.61)\ (34.03^2)$

$I = 23325 + 237.6 + 44500 = 68065$ in.^{4}

$\frac{I}{c} = \frac{68065}{36.25} = 18.75$ in.^{3} O. K.

No rivets to connect hitch ∠s to webs of girders

70' encased, maximum shear 77,950

unit stress, field driven rivets

Shear = 10,000 # sq. in.

Bearing = 20,000 # sq. in.

1 7/8" rivet, doubleshear = .6 x 10000 x 2 = 12,000#

1/7/8" rivet bearing = 7/8 x ½ x 20000 = 8750# per sq. in.

bearing decides

$\frac{77,950}{8750} = 9$ -12 provided   O. K.

To fasten hitch ∠s to box girder

Single shear = 6000 3 per sq. in.

$\frac{77950}{6000} = 13$ provided O. K.

This girder has largest shear. All others have same number of rivets in hitch ∠s, therefore, they must be O. K. also.

Flanges stayed by diaphrams at distances not to exceed 12 times the width of flange.

Width of flange = 16"

12 x 16/12 = 16 ft. which is greater than design, therefore is safe.

Unit stress in piers

Piers 1-6-7-8-13

Specifications

Columns where h/D ∠ 12', with longitudinal reinforcement and ties. Allowable unit stress on total concrete area is .20 fs.

Class B concrete fc = 2000 lbs. per sq. in.

Dimensions of pier 4' x 5' x 31.6'

h/D = 31.6/4 = 7.9

fc^1 = .20 x 2000 = 400 lbs. per sq. in.

total load = 9[illegible]50950 + 54075) = 945225 lbs.

Each pier takes $\frac{945225}{2}$ = 477,600 lbs.

Weight of pier = 5 x 4 x 31.5 x 150 = 96,000#

Weight of box girder = 22,800

Weight of box girder taken by one pier = 11,400#

Total load = 585,000 lbs.

fc^1 = P/bd = $\frac{585,000}{20 \times 144}$ = 205 lbs. per sq. in.

This is low, but the bridge was designed for a 40 ft. roadway, so these piers are sufficiently large to take care of future widening.

This is for pier No. 7 where 70' - 65' girders meet. Piers 1-6-8-13 are same size and of less height, carrying smaller loads, so they are O. K. for unit stresses

Design of base

Dimensions 12.5' x 12.5' x 2.5'

Each base has total bearing of 585,000# + $(12.5)^2(2.5)(150)$ = 590,800 #

This load is carried by 25 piles

590,800/25 = 23632 lbs. = 11.3 Tons per pile

Bending moment of base

$M = W/2\ (a + 1.2c)c^2$

$W = \frac{590,800}{(12.5)^2}$ = 3800 lbs. per sq. ft.

$M = \frac{3800}{2}\ (4.25 + 1.2(4.25)\ 4.25^2$ = 320000 ft. lbs.

320000 x 12 = 3,840,000 in. lbs.

or $M = 3800/2\ (5 + 1.2(3.75)\ 3.75^2$ = 254,000

254,000 x 12 = 2,848,000 inch lbs.

Shear

$V = w(L^2-(a+2d)^2) = 3800\ (12.5^2-(4.25+5)^2) = V = 258{,}000$ lbs.

$v = \dfrac{258{,}000}{4(4.25+5)(.875)12(30)} = 23$ lbs. per sq. in.

40# per sq. in. is permissible so this will care for future widening.

$As = \dfrac{M}{fs\,jd} = \dfrac{3{,}840{,}000}{16000x.875x30} = 9.13$ sq. in.

16-1' square bars provided or $\dfrac{2{,}848{,}000}{16000x.875x30} = 6.8$ sq. in.

14-1" sq. bars provided this way.

Unit bond stress on bars

$U = \dfrac{w(ac+c^2)}{\Sigma o\,jd} = \dfrac{3160(4.25x4x4.25^2)}{14(4)(.875)(30)} = 75\#$sq. in.

$\dfrac{3160((3.75 x 5) + 3.75^2)}{16(4).875(30)} = \dfrac{3160 x 33}{64 x .875 x 30} = 62\#$ sq. in.

Check of pile loads of abutments load; 70' unencased girders, maximum

shear = 54,075 lbs.

Spacing of girders 4'-5"

Weight per linear ft. = 54075/4.4	12300
weight of vertical wall = 150x1.5x14.6	3280
weight of base = 2 x 10 x 1.50	3000
weight of earth = 5.5 x 100 x 20.5	11275
surcharge = 25000/10	2500
	32,355 lbs.

P = load per lineal ft. in tons = 16.2 tons

Total surcharge = 6 + 4.5 = 10.5'

Total h of earth = 14.5 + 10.5 = 25'

W_1 = 25x100x5.5 =	13750#
W_2 = 12,300 + 3280 =	15580
W_3 = 2x10x150 =	3000
	32330 lbs.

$x = \dfrac{13750x7.25 + 15580 x 3.75 + 3000 x 5}{32330} = 5.35"$

$y = \dfrac{16.6^2 + 3(16.6)(10.5)}{3(16.6 + 21)} = 6.69$ ft.

$P = \frac{1}{2}(25)(37.6)(16.) = 7802$ lbs.

$\dfrac{32,330}{6.89} = \dfrac{7802:}{x}$ $\quad x = 1.60$

$x_0 = 160 - (5.35 - 5.00) = 1.25$ Puts base directly under vertical wall

Pile load, south abutment

Row No.	S	n	a	na	na^2
1	5	.200	1.00	.200	.2.000
2	3.75	.67	4.00	1.068	4.072
3	2.5	.400	6.50	2.600	16.900
4	2.5	.400	9.00	3.600	32.400
Summations		1.267		7.468	53.372

$L = 10$; $e = 1.25/10 = .125$; $P = 16.2$ tons; $r = \dfrac{7.468}{1.267} = 5.9'$

$Bk_0 = \dfrac{53.372 + 10(.5 + .125) \times 7.468)}{(1.267 \times 53.372) - (7.467)^2}$ $\quad P = .564P$

$Bk = \dfrac{(10(.5 + .125) \times 1.267) - 7.468}{(1.267 \times 53.372) - (7.468)^2} \times P = .038P$

$P_1 = (.564 + (1 \times .038))16.2 = 9.75$ tons

$P_2 = (.564 + (4 \times .038))\ 16.2 = 11.6$ tons

$P_3 = (.564 + (6.5 \times .038))\ 16.2 = 13.14$ tons

$P_4 = (.564 + (9 \times .038))16.2 = 14.67$ tons

Vertical wall of abutment

Total surcharge = 10.5'

$h = 10.5 + 14.6 = 25.1$

Base = 2'

height of curtain wall = 14.6'

$P = wh$ at any point $\quad w = 25$; $h = 25$

$P = 25 \times 25 = 625$ at top of footing

$P = 25 \times 10.5 = 262$ at top of wall

$M = \dfrac{wl^2}{10}$ $\quad l = 8.5$

$M = \frac{625(8.5)^2}{10} = 4500$ lbs. per ft. of width

$d = \sqrt{\frac{4500}{146.9}} = 6"$

$As = pbd = .0107 \times 12 \times 6 = .77$ sq. in per ft. of width

$\frac{3}{4}"$ bars placed 6" on centers is sufficient

Length of steel for negative moment

$x = \frac{f t_s}{4u} = \frac{16000 \times \frac{3}{4}}{4 \times 80} = 37.5"$ 38" is provided

Bond and shear

$V = 640 \times 4.25 = 2720$ lbs.

$u = \frac{2720}{\left(\frac{12}{6}\right)(2.356)(15)(.857)} = 38$ lbs. per sq. in.

$v = \frac{2720}{\left(\frac{12}{6}\right)(.857)} = 10$ lbs. per sq. in.

Check vertical wall for load from girders

Load per foot from girders	12,300#
wt. of one foot of wall	3,280
Total wt. per lineal ft.	15,580 lbs.

Compression in wall per square in.

$\frac{15580}{1.5 \times 12 \times 12} = 72$ lbs. per sq. in.

This is very safe.

Outer Cantilever

$P_1 = \frac{32330}{10}\left(1 + \frac{6 \times 1.25}{10}\right) = 5650\#$

$P_2 = \frac{32330}{10}\left(1 - \frac{6 \times 1.25}{10}\right) = 808\#$

P at outer edge of vertical wall

$\frac{x'}{4840} = 7/10;\ x' = 3390;\ P = 3390 + 808 = 4200\#$

Downward pressure

$3 \times 2 \times 150 = 900$ lbs.

Moment $= 900 \times 12 \times 1.5 = 16200$" in. lbs.

Pressure up $= (4200 \times 3 \times 3/2 + 1450 \times \frac{1}{2} \times 3 \times 3 \times 2/3)12 = 279000$ in lbs.

Resulting moment $= 279000 - 16200 = 262,800$

$$d = \sqrt{\frac{262,800}{146.9 \times 12}} = 12.2 \text{ in.}$$

$$V = \frac{5350 + 4200}{2} \times 3 - 900 = 13,775 \text{ lbs. total shear}$$

Assuming $= .90$ for the depth to satisfy shearing stress

$$d = \frac{13,775}{12 \times 40 \times .90} \qquad 32"$$

For this depth bond will govern steel 5 inch spacing of $\frac{3}{4}$" bars

$$P = \frac{.442}{5 \times 21.3} = .0041; \quad j = .903$$

$$u = \frac{13775}{\left(\frac{12}{5}\right)(2.36)(.903)(32)^2} = 84 \text{ lbs. sq. in.}$$

$$fs = \frac{262,800}{(.0041)(.903)(12)(32)^2} = 6,000 \text{ lbs. sq. in.}$$

$$\frac{f1}{4u} = \frac{6000 \times \frac{3}{4}}{4 \times 80} = 14 \text{ inches}$$

Inner floor slab

Resultant downward pressure on outer one foot strip.

$$\text{The upward pressure} = \frac{800 + 1284}{2} = 1042$$

weight of earth $= 2300$

weight of footing $= 300$

Uniform load on end strip $= 2600 - 1042 = 1600$#

$$M = \frac{1600 \times (9.5)^2}{10} = 11600 \text{ ft. lbs.}$$

$$d = \sqrt{\frac{11600}{146.9}} = 9 \text{ in.}$$

Depth of slab is controlled by shear.

The total shear at the edge of the counterfort is $(16.00)4.25 = 6800$ lbs. per linear ft.

$d = \frac{6800}{(12)(.90)(30)} = 21"$ total thickness $= 24$ which checks with design

Size and spacing of bottom rods

$K = \frac{M}{bd^2} = \frac{11,600}{24^2} = 20$

$P = \frac{K}{fsj} = \frac{20}{16000 \times .90} = .0014$

$As = (.0014)(12)(24) = .403$ sq. in.

$\frac{1}{2}"$ square spaced at 6" is sufficient which is used in design

Rods of same size used over supports

Bond will control

$\text{Spacing} = \frac{(80)(12)(2)(.90)(24)}{6800} = 6"$

This checks with design

Spacing of rods increases for each one ft. strip the decrease in load =

$\frac{5550-900}{10} = 465$ lbs.

at $1\frac{1}{2}$ feet from end the unit load is $1600-700=900$

$\text{spacing} = \frac{(6)(1600)}{900} = 10"$ which checks

and spacing of bottom rods $= \frac{(6)(1600)}{900} = 10"$ O. K.

Counterfort

Total force transmitted to counterfort is $P = \frac{1}{2}(14.6)(25)(35.1) = 6400$ lbs.

$y = \frac{(14.6)^2 + 3(14.6)(10.5)}{3(35.1)} = 6.4$ ft.

Total $P = 6400 \times 10 = 64000$

$M = 64000 \times 6.4 \times 12 = 4,900,000$ in lbs.

moment arm scales 4.85'

$K = \frac{M}{bd^2} = \frac{4,900,000}{(18)(4.85)^2(12)^2} = 81$

$P = .0056$ (table 6)

6—1" square bars used

$As = .0056 \times 9.85 \times 12 \times 18 = 5.9$ sq. in.

2 bars are stopped at 7 ft.

$P = \frac{1}{2} \times (7.6)(25)(28.1) = 1170\#$

$$y = \frac{(7.6)^2 + 3(7.6)(10.5)}{3(28.1)} = 3.53$$

$M = 1170 \times 3.53 \times 12 = 495{,}000$ in lbs.

Moment arm circle = 3 ft

$$K = \frac{495000}{18 \times 9 \times 144} = .0021 \text{ required}$$

5 rods carried through

$p = 4/18x5x12 = .0032$ provided

Horizontal rods to tie counterfort to vertical wall

at top of footing shear = wh (25)(25(8.5)) = 5300 lbs.

½" in round bars used

required number in a foot of height $= \frac{5300}{(.196)(16000)} = 1.7$ 2 per foot used.

at 4½ feet above footing

wh = 20 x 20.5 x 8.5 = 3400 lbs.

no. bars per foot $= \frac{3400}{.196x1600} = 1.1$

2 at 14" spacing used

These rods are in pairs each hooked around outer horizontal reinforcing

bars to obtain satisfactory bond

Vertical bars to tid counterforts to base

Shear on outer foot = 1600 x 8.5 = 12,800 on succeeding 1 ft. strips ten-

sion is lessened by $\frac{5330-800}{10}$ (8.5) = 3900 lbs.

½" bars used at 1½ ft. 12,800-1½(3900) = 6950#

No. rods per ft. $= \frac{6950}{(.196)16000} = 2.2$ 2 rods used

At 2½ ft. V = 12,800 - 2½(3900) = 3050

no. rods per ft. $= \frac{3050}{.196 \times 16000} = 1.3$

2 at 15" used

factor of safety for overturning

$5.36/1.30 = 3.4$

Factor of safety for sliding

$$\frac{.31 \times 32.330}{7300} = 1.36$$

Design of wing walls

Dimensions

Height of wall 8 ft.

Thickness $1\frac{1}{2}$'

Angle of earth, 30^0

Base = 2' thick

outer cantilever = 2'-3"

$\frac{3}{4}$" bars spaced 6"

$$P = \frac{1}{2} \times 100(8) \quad (.866) \qquad \frac{.866 - .281}{.866 + .281} = 2200 \text{ lbs.}$$

$H = 2200 \times \cos 30^0 = 1900$ lbs.

acts at 2.67'

$M = 1900 \times 2.67 = 5060$ ft. lbs.

$$d = \sqrt{\frac{5060}{146.9}} = 5.9$$

$P = .0107$

$As = .0107 \times 12 \times 5.9 = .757$ sq. in required

$\frac{3}{4}$" bars spaced at 6" provides .88 sq. in. per ft.

Design of floor slabs

$\frac{3}{8}$" sq. bars top and bottom spaced 5"

Distance from compression surface to center b

bottom steel = $7\frac{1}{2}$ inches

$$M = \frac{wl^2}{8}; \; w = \frac{815000 \times 10}{(65)^2} = 1930\# \text{ per ft.}$$

wt. of girder = 200 lbs. per ft.

$w = 1930-200 = 1730$ per linear ft.

clear distance between girders $4'-5" - 16" = 3'-1"$

w for 1ft strip of pavement $= w = \frac{1730}{308} = 562$ lbs.

Considering each strip as a 1 ft. beam partly continuous

$m = \frac{wl^2}{10}$; $m = \frac{562 \times (37)^2}{10} = 7700$ lb. in.

$P = \frac{K}{fsj} = \frac{92.8}{16000 \times .9}$ .00573

$K = 92.8$ (Table 6)

$As = .00573 \times 12 \times 7.5 = .52$ sq. in. per sq. ft.

$\frac{1}{2}"$ square bars spaced 5" = .6 sq. in. per sq. ft.

Same size and spacing of bars take care of negative moment

Shear test

$v = \frac{V}{bjd} = \frac{2610}{12 \times .867 \times 7.5} = 34$ lbs. safe

Crown

$C = \frac{R^2}{533} = \frac{(28)^2}{533} = 1.42$ inches

This is above minimum

Discussion of Check Design of Steel Deck Girder Bridge

The following description, analyses and computations for the check design pertains to a bridge which is located on South Main Street, Rochester, Michigan. It is in reality a grade crossing, as it carries the roadway over First Street, the Grand Trunk Railway, Clinton River, and South Street. It is steel deck girder type, with spans as follows: 4-70', 2-65', 8-50', 28' roadway and 1-5' sidewalk, with provision for future widening to a 40' roadway and 2-5' sidewalks. One 70' span is over the G. T. R. R. and these girders are encased. The Detroit United Railway runs close beside the bridge, and the bridge abutments are wide enough to take care of this. They are of counterfort construction, and rest on 15 ton piles, as do the piers.

Test holes show that the soil is sand ranging in grading from fine to medium coarse, and contain a large percentage of clay. Beneath this layer of sand and at an average elevation of 153' is a layer of very hard sandy clay in which the clay predominates.

The plans were drawn in the latter part of 1926, and the bridge built in 1927. The plans used in this thesis were obtained from the Michigan State Highway Department at Lansing, as were the specifications used in the computations made in checking the stresses. C. A. Melick was the Bridge Engineer

The loading used in the design of this bridge is shown in the loading diagram, also the moments and shears which are used with this loading. These values were figured by the State Highway Department. The complete plans do not accompany this thesis. The loading diagram, tabulated shears and moments, and drawings showing details of the steel girders are included. Also drawings showing dimensions of abutment, and general plan of bridge. All steel girders, piers, pile loads, abutments, wing walls, and floor of bridge have been calculated and checked. Each of these members and the stresses in the composing materials will be thoroughly discussed in the following report. The discussion will follow the check calculations as well as possible.

The floor design was checked first, and the stresses checked very closely with the specifications. The reinforcement was considered as running only one way, and the floor was figured as a partly continuous beam. The reinforcement which ran parallel with the girders was for the purpose of holding the bars in place. Expansion joints were provided. The 70 foot encased girders were checked next. It is a built up girder, and it checked closely, in every way.

The pitch of the flange angle rivets was found to be satisfactory, and as this is the maximum spacing of 6", it governs the pitch of the flange angle rivets of all the girders, as this one carries the largest load.

The 70' unencased girders were next checked, and were found to be satisfactory. The stiffener spacing is the same on all the girders, so a check on the first is in reality a check on them all as far as the stiffeners are concerned.

The 65' and 50' girders were all so found to be correct as to design.

The deck girders were connected to box girders which rested on the piers. The hitch angle rivets were checked and were found to be satisfactory.

The box girders are built up of two webs and filled with concrete. These girders are heavier than needed for the load they now carry, but were made so in anticipation of future widening. They are designed for a 40' roadway. The check was made for a load of this size, and the check proved that they were correctly designed. Box girder G9 was checked first. This girder carried the heaviest load, and it checked closely with the requirements. The flange angle rivet pitch was uneven, but this was necessary in order to place the stiffeners. My check for pitch of the rivets was about the actual average pitch.

Box girder G 8 was checked, and the provided Section modulus was found ample. The pitch of the rivets and the stiffener area and spacing was the same as G 9. Box girder G 7 is identical with G 8 with a smaller load. Likewise Girders G 1 and G 2 were checked and found to be satisfactory.

The design of the piers were checked next. The unit stress in the reinforced concrete was found to be only 205 lbs. per square inch, while 400 lbs. is allowable. The extra strength is no doubt provided to take care of future widening, as these piers are probably designed for a forty foot roadway.

The design of the base was checked and was found to be understressed, but will not be when the roadway is widened. The pile loads were checked, and they are sufficient for future widening.

The check of the south abutment followed. Only one abutment was checked as they are both the same. This checked quite closely in most ways, except for a few things. There seemed to be a slight excess of steel in the vertical wall, and the base did not seem to be quite thick enough for vertical shear. However, these variations were slight. The stresses in the inner floor and in the counterforts checked very closely. There was an ample factor of safety both for overturning and for sliding. The short retaining wall at the ends of the abutment was checked next.

This afforded an opportunity to use Rankin's formula for inclined earth back of the wall. The stress were found to be well within the allowable limits.

The pile loads were checked for the piles beneath the base of the abutment and were found to check very closely. The allowable load was 15 tons per pile. The check showed that the heaviest load taken by any pile was taken by the ones in the outside row, and was 14 2/3 tons. This was a very good result.

The design of the railings and curbs are according to the specifications of the Highway Department with the exception that the curb on the side of the road opposite the sidewalk is only nine inches from the railing, whereas the specifications say it must not be closer than 12 inches.

The crown of the roadway is correct.

The railings are standard, the design being according to specifications for bridges built by the Michigan State Highway Department

General Discussion of Thesis

In checking the design of this structure, the work afforded an excellent study of steel girder design, reinforced concrete piers, counterforted retaining walls, and pile loads. The design of a bridge requires many phases of civil engineering. Also this thesis gave me valuable practise in the reading and interpreting blueprints.

I chose this subject because of the fact that it involved a number of problems which are important to an Engineer. Also I wanted to discover for myself how the theories and formulas we have been using in our studies would work out when applied to an actual structure. It has been very pleasing to me to see how closely my calculations have checked with the actual design of the bridge. It gives one more confidence in the knowledge he has gained in the classroom.

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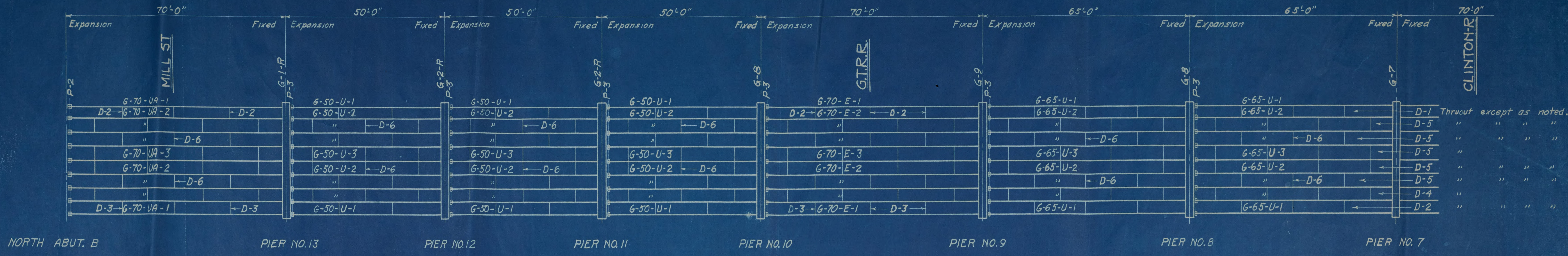
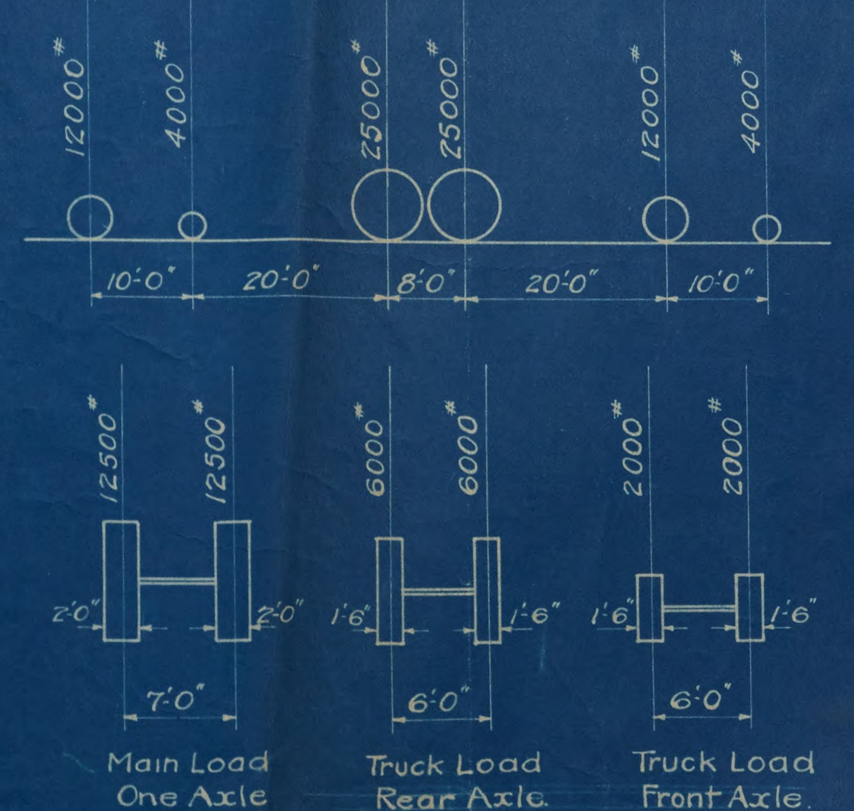
[redacted]



TABULATION OF MOMENTS AND SHEARS

FOR FLOOR GIRDERS AND BOX GIRDERS.

SPAN	MOMENTS - FT. LBS.				SHEAR - LBS.				SECTION MODULUS	
	LIVE LOAD	DEAD LOAD	IMPACT.	TOTAL	LIVE LOAD	DEAD LOAD	IMPACT	TOTAL.	REQUIRED	PROVIDED-GROSS
FLOOR GIRDERS - SAME SECTION PROVIDED FOR OUTSIDE AND INTERMEDIATE GIRDERS.										
70'-ENCASED.	379000	870000	94800	1344000	22000	50450	5500	77950	1005	1210
70'-UNENCASED	379000	459000	94800	933000	22000	26575	5500	54075	700	755
65'-UNENCASED	345000	383000	86400	815000	21600	23950	5400	50950	610	705
50'-UNENCASED	232800	209000	58200	500000	19000	17100	4750	41000	375	440
BOX GIRDERS - FIGURED FOR FUTURE ROADWAY OF 40' AND 2-5'-0" SIDEWALKS.										
70' ENCASED - 65' GIRDER	-391000	-2553000	-98000	-3042000	62000	279000	16000	357000	2280	2570
70' ENCASED - 50' GIRDER	-371000	-2357000	-93000	-2821000	59000	259000	15000	333000	2120	2340
70' UNENCASED - 65' GIRDER	-391000	-2174000	-98000	-2663000	62000	228000	16000	306000	1995	2340
70' UNENCASED - 50' GIRDER	-371000	-1959000	-93000	-2423000	59000	208000	15000	282000	1820	2110
50' - 50' GIRDER.	-337000	-1706000	-84000	-2127000	54000	181000	13000	248000	1593	1875



—LOADING DIAGRAM—

NOTE:- See Sheets No. 15 to 23 inclusive for details of Floor Girders, Box Girders, Pedestals, Bearing Shoes, Stirrup Castings, Bolts, & Nuts.

MICHIGAN STATE HIGHWAY DEPARTMENT

SEC. 14, T3N, R11E AVON TWP. OAKLAND CO.

BRIDGE FILE NO. 6302001 ROAD R.B. AT SOUTH LIMITS OF ROCHESTER

CROSSING OVER FIRST ST., G.T.R.R., CLINTON RIVER AND SOUTH ST.

ERECTION DIAGRAM

4-70', 2-65', 8-50' STEEL DECK GIRDER SPANS, 28' ROADWAY, 1-5' SIDEWALK
PROVISION FOR 40' ROADWAY & 2-5' SIDEWALKS
1-40' SPAN, 24' ROADWAY, 1-5' SIDEWALK, CROSSING CLINTON RIVER

Approved L. M. Foster
Chief Draftsman

Approved W. McDonald
Engineer of Design

Approved C. A. Melick
Bridge Engineer

Drawn by	Wright	8-26-26
Traced by	"	9-1-26
Checked by	Nelson	10-19-26
Filed by	Gill	10-26-26
SHEET No. 14 of 44		

X1 of 63-2-21