AN ANALYSIS OF THE DESIGN OF A WELDED PLATE GIRDER BRIDGE

Thesis for the Degree of B. S.

MICHIGAN STATE COLLEGE

G. G. Sangster

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An Analysis of the Design of a

Welded Plate Girder Bridge

A Thesis Submitted to

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by

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<u>An Analysis of the Design of a Welded Plate Girder Bridge</u>

Sometime in the future a building to be used as a hotel is to be constructed for Michigan State College and will be located north of the Red Cedar River on Harrison Road in East Lansing, Michigan. A bridge is to be built to carry pedestrian traffic across the river at the building site. The analysis of the design of this bridge is the subject of this thesis.

The bridge will be of welded plate girder design spanning 94' - 0" c/c of bearings. The web is to consist of five sections welded together at panel points two, five, eight, and eleven for a total of thirteen panels. In order to make the structure attractive, the edges of the plate girder are to be cut on two separate parabolic curves with cambers of 12 inches and 30 inches upper and lower, respectively. This allows a minimum clearance of 1' - 0" above high water. These sections will be cut by the fabricator.

Preliminary soil investigations allow a maximum soil pressure of 5000 pounds per square foot for the structure. The maximum value for this design is 3920 pounds per square foot and well within the allowable limit.

The abutments are to be of reinforced concrete. The design is such that the fill need not be placed inside

and outside simultaneously. It will be noted that the area of reinforcing steel used generally exceeds the area required. This is due to the fact that the convenience of field layout was considered in determining the spacing and the size of the bars used. The saving in labor costs will exceed the cost of the extra steel. However, if the job were larger with long repeating sections this might not be an economical procedure.

The live load on the bridge is extremely light, 100 pounds per square foot, and is purely a pedestrian loading. Specifications for bridges carrying only pedestrian traffic are not available. Except as noted, this design follows the recommendations of the American Association of State Highway Officials.

The preliminary sketches of three bridges, each of different design, have been submitted by a consulting engineer to the Board of Agriculture. At the time of this writing a definite acceptance has not been received. Due to the fact that the structure shown as "Preliminary 2" is the most likely to receive approval, its design was chosen as the subject for analysis.

FOOT BRIDGE ACROSS RED CEDAR
MICHIGAN STATE COLLEGE
SCALE 1/8"=1'-0"

PRELIMINARY #2

2-28-49

CLAUD R. ERICKSON
CONSULTING ENGINEER

Data

Panels

11 @ 7' - 3"

2 @ 7' - $1\frac{1}{2}$"

Width

16' - 0"

14' - 0" (face to face)

Loads

LL = 100 $\#/ft^2$

Impact = 0

Working Stresses

f_s = 20000 $\#/in^2$ (reinforcing steel)

f'_c = 3000

f_c = 1050

n = 10

k = 160

v = .02 f'_c = 60 $\#/in^2$

u = .05 f'_c = 150 $\#/in^2$

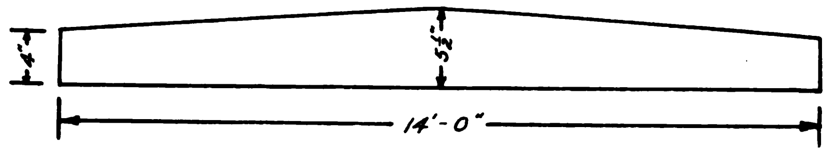
k = .344

j = .385

AASHO Specs

f_s = 18000

Deck Slab



A slab of this general shape is desired. Assume the 4 inch & 5½ inch dimensions.

<u>Weight</u>

$= \frac{5.5}{12} \times 150$

$= 68.8 \ \#/ft^2$

$= 70 \ \#/ft^2$

<u>Total Load</u>

LL $= 100 \ \#/ft^2$

Wt $= \underline{70 \ \#/ft^2}$

Total $= 170 \ \#/ft^2$

<u>Moment</u>

$M = 1/10 \ wl^2$

$l = 7' - 3"$

$= 7.25$ ft

$= 1/10 \times 170 \times (7.25)^2 \times 12$

$= 11{,}700$ in - lbs

<u>Effective Depth</u>

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

$= \sqrt{\frac{11700}{(160)(12)}}$

$= 2.48$ inches

$= 2.75$ inches giving 1.25 inches of cover

Deck Slab (Continued)

<u>Area of Steel</u>

$$A_s = \frac{M}{f_s\ j\ d} \qquad j = .885$$

$$= \frac{11700}{20000 \times .885 \times 2.75}$$

$$= .24\ in^2$$

Use 3/8" ø @ $5\frac{1}{2}$ inches (Σ_o = 2.6in)

<u>Temperature Steel</u>

$$A_T = .002\ bd$$

$$= .002 \times 12 \times 2.75$$

$$= .007\ in^2$$

Use 3/8" ø @ 12 inches

<u>Shear at Edge of Support</u>

$$V = w \times \tfrac{1}{2}\ \text{clear span}$$

Assume a 7' - 0" clear span

$$= 170 \times \tfrac{1}{2} \times 7$$

$$= 595\ lbs$$

<u>Bond</u>

$$u = \frac{V}{\Sigma_o\ j\ d}$$

$$= \frac{595}{2.6 \times .885 \times 2.75}$$

$$= 94\ \#/in^2$$

Allowable bond = 150 #/in^2

<u>Shear Intensity</u>

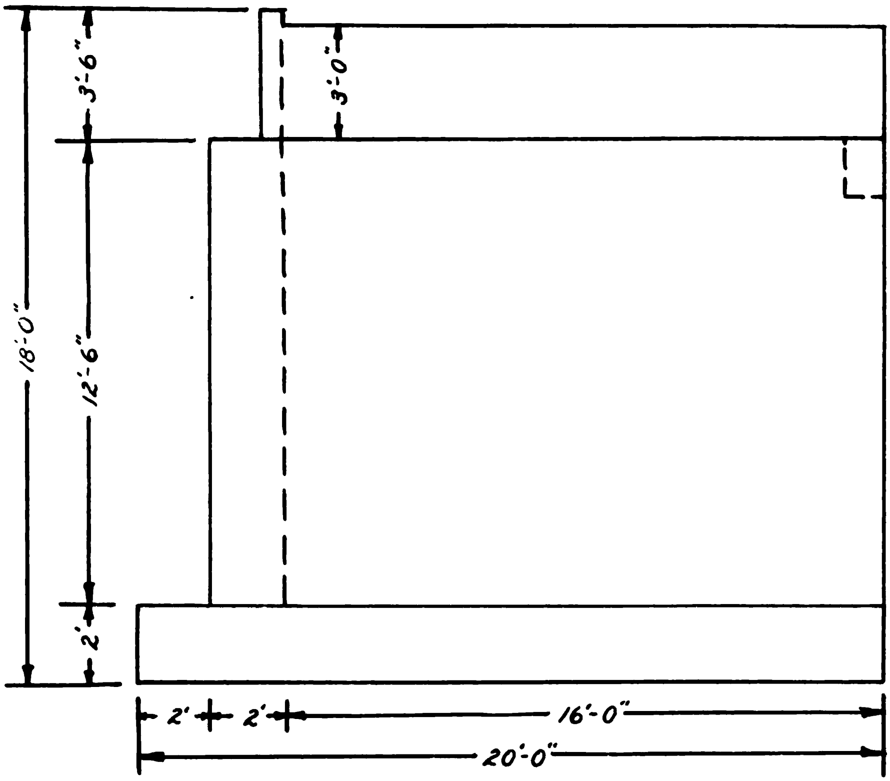
$$v = \frac{V}{b\ j\ d}$$

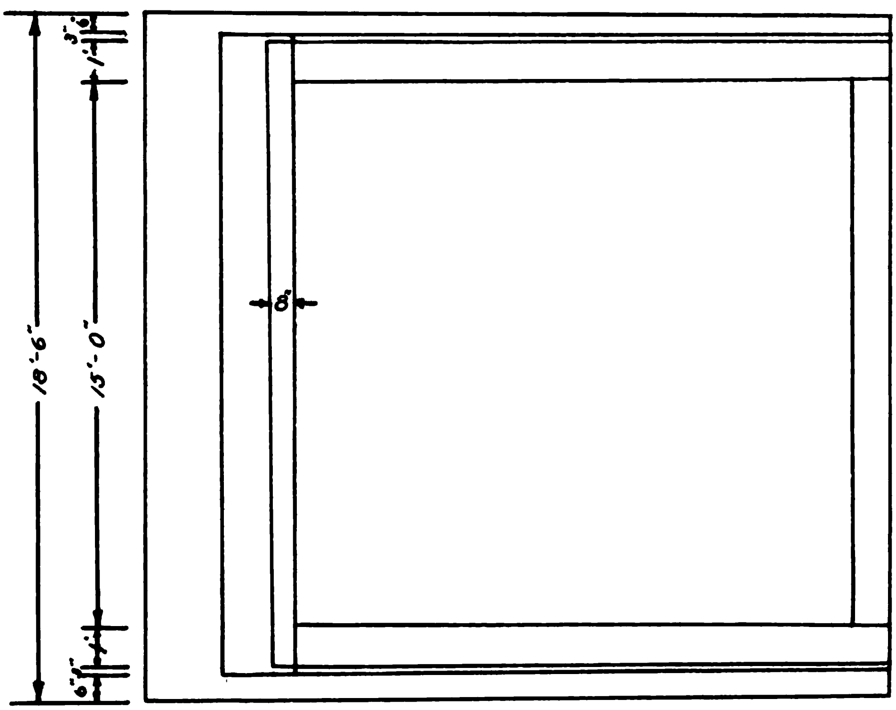
$$= \frac{595}{12 \times .885 \times 2.75}$$

$$= 20.4\ \#/in^2$$

Allowable v = 60 #/in^2

Abutment Design





Abutment Design

Super Structure Loading

400 #/ft assumed wt

100 #/ft^2 LL

60 #/ft^2 slab weight

400 x 2 = 800

100 x 14 = 1400

60 x 14 = 840

Total = 3040 lbs

Reaction

3040 x 94 x 1/2 = 143000

= 143 kips

Righting Moment

			Wt in kips	Lever Arm	Moments (kip feet)
1)	Footing 20 x 18.5 x 2 x .150	=	111	10	1110.0
2)	Front Wall 12.5 x 17.5 x 2 x .150	=	65.7	3	197.1
3)	Sidewalls 2 x 12.5 x 1.25 x 16 x .150	=	75.2	12	903.0
4)	Tie Beam 1.5 x 15 x 1 x .050	=	1.1	19.5	21.4
5)	Gravel Fill 12.5 x 15 x 16 x .100	=	300.0	12	3600.0
6)	Super Structure Load 94 x .5 x 3.040	=	143.0	2.75	394.0
7)	Backwall 3.5 x 16.5 x .67 x .150	=	5.8	3.67	21.3
8)	Wing Walls 2 x 6 x 1 x 16 x 150	=	28.8	12	346.0
9)	Approach Slab .5 x 14 x 16 x .150	=	16.8	12	202.0
10)	Gravel Fill 2.5 x 14 x 16 x 100	=	56.1	12	674.0

Vertical Reaction = 803.5 kips

Righting Moments = 6468.8

Abutment Design (Continued)

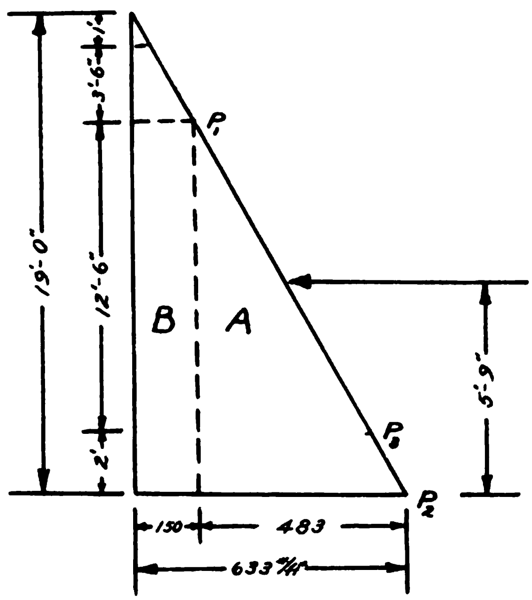
The forces acting on the abutment sections will be calculated by the "fluid-force" method. The fluid weighs 33 1/3 $\#/ft^3$ and acts on the back of all walls and is equivalent to earth acting in the same manner.

<u>Earth Pressure Acting on Walls</u>

$$A = 14.5 \times \frac{14.5}{2} \times \frac{100}{3} = 3510$$

$$B = 14.5 \times 4.5 \times \frac{100}{3} = 2180$$

$$\text{Total} = 5690 \text{ lbs/ft}$$



<u>Pressures</u>

$$p = wh$$

$$p_1 = \frac{100}{3} \times 4.5 = 150 \ \#/ft^2$$

$$p_2 = \frac{100}{3} \times 14.5 + p_1$$

$$= 483 + 150 = 633 \ \#/ft^2$$

$$p_3 = \frac{100}{3} \times 12.5 + p_1$$

$$= 417 + 150 = 567 \ \#/ft^2$$

(acting at base of wall)

<u>Position of Resultant Earth Pressure</u>

$$\bar{y} = \frac{H}{3} \frac{(p_2 + 2p_1)}{(p_2 + p_1)}$$

$$= 14.5 \frac{(633 + 300)}{(633 + 150)}$$

$$= \frac{14.5 \times 933}{3 \times 783}$$

$$= 5.75 \text{ feet from bottom}$$

Abutment Design (Continued)

Moment of Overturn

$$M_o = E \times \bar{y} \times \text{length}$$

$$= 5690 \times 5.75 \times 17.5$$

$$= 572 \text{ kip ft}$$

Factor of Safety Against Overturning

$$FS_o = \frac{M_r}{M_o}$$

$$= \frac{6468.8}{572}$$

$$= 11.3 \quad (\text{Required FS} \geqq 2)$$

Factor of Safety Against Sliding

The coefficient of friction between concrete and damp clay is taken as tangent = .33

$$FS_s = \text{Forces Producing} \div \text{Forces Preventing}$$

$$= \frac{803.5 \times .33}{5.690 \times 17.5}$$

$$= 2.66 \quad (\text{Required FS} \geqq 2)$$

Point of Action of Resultant

$$X = \frac{M_r - M_o}{R_v} \quad \text{From} \quad M_{toe}$$

$$= \frac{6468.8-572}{803.5}$$

$$= \frac{5896.8}{803.5}$$

$$= 7.35 \text{ ft from toe}$$

Abutment Design (Continued)

Toe and Heel Pressures

$$e = 10 - 7.35$$
$$= 2.65$$

$$= \frac{W}{L}\left(1 \pm \frac{6e}{L}\right) \times \frac{1}{\text{Width}}$$

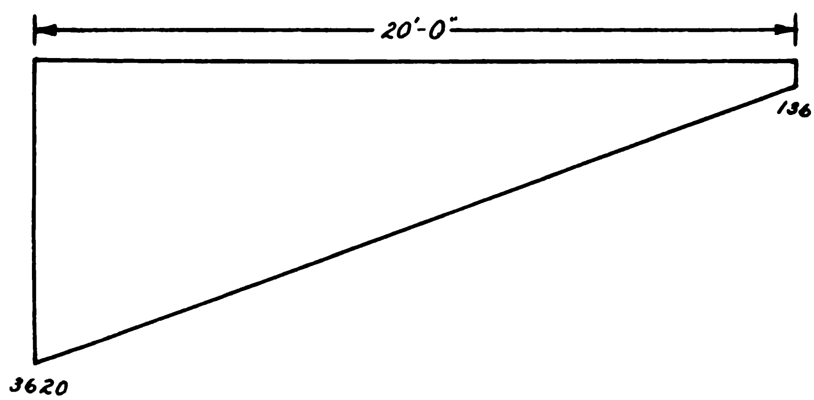
$$= \frac{803500}{20}\left(1 \pm \frac{6 \times 2.65}{20}\right) \times \frac{1}{18.5}$$

$$= \frac{40175}{18.5}(1 \pm .80)$$

$$P_{Toe} = 3920 \text{ lbs/ft}^2$$

$$P_{Heel} = 436 \text{ lbs/ft}^2$$

Footing Soil Pressures



Pressures:

The weight of the concrete is subtracted from the pressures as found above due to the fact that the downward force is opposed by the upward forces of the soil.

$W_c = 150 \times 2 = 300$ lbs

Slope

$$= \frac{3620 - 136}{20}$$

$$= \frac{3484}{20}$$

$$= 174.2 \text{ \#/ft}$$

Footing Slab

Toe

Assume that 3620 $\#/ft^2$ acts uniformly over the 2 foot width of the toe. This is logical because of the small dimension.

Moment

$$M = 3620 \times 2 \times 1$$

$$= 7240 \text{ lb - ft}$$

Effective Depth

$$d = \sqrt{\frac{M}{K\ b}}$$

$$= \sqrt{\frac{7240 \times 12}{160 \times 12}}$$

$$= 6.75 \text{ inches}$$

Assume:

3/4" ø steel

3 1/2" of cover

Use 20 1/2 inches

Area of Steel

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

$$= \frac{7240 \times 12}{20000 \times .885 \times 20.5}$$

$$= .240 \text{ in}^2$$

Use 3/4 inch ø @ 21 inches in the direction of the width.

$$A_s = .25 \text{ in}^2$$

$$\Sigma_o = 1.35 \text{ inches}$$

Check f_c

$$f_c = \frac{2M}{k\ j\ b\ d^2}$$

$$= \frac{2 \times 7240 \times 12}{.344 \times .885 \times 12 \times 20.5^2}$$

$$= 113\ \#/in^2 \qquad \text{Allowable } f_c = 1050.0\ \#/in^2$$

Footing Slab (Continued)

Toe

Minimum Steel

$$A = .002\ bd$$

$$= .002 \times 12 \times 20.5$$

$$= .493\ in^2$$

Minimum Allowable 3/4" ø @ 10.5 inches.

This is the minimum allowable reinforcing steel which can safely be used running lengthwise in the top section of the floor slab.

Floor

It is desired that the stress be distributed with uniform intensity between the steel running lengthwise and that running across the width of the floor.

First it will be necessary to find the maximum loading which can be carried by a steel area of 0.493 square inches.

Loading

$$M = 1/8\ wl^2 \qquad l = 16\ 1/2 \text{ ft c/c of walls}$$

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

$$W = \frac{f_s\ j\ d\ A_s}{1/8 \times l^2}$$

$$= \frac{20000 \times .885 \times 20.5 \times .493}{.125 \times 12 \times 16.5^2}$$

$$= 438\ \#/ft$$

Footing Slab Floor (Continued)

Second, find the length of cantilever from the front wall over which this loading acts.

Length of Cantilever

Referring to soil pressure diagram

$$l_c = 20 - (2 + 2 + x)$$

x = distance of action from heel

$$\text{Slope} = 174.2 \ \#/\text{ft}$$

$$w = 438 - 136$$

$$= 302 \ \text{lbs/ft}$$

$$x = \frac{302}{174.2}$$

$$= 1.73 \ \text{ft}$$

$$l_c = 20 - (2 + 2 + 1.73)$$

$$= 20 - 5.73$$

$$= 14.27 \ \text{ft from front wall}$$

Moment Required to Carry 433 #/ft

$$M = \text{Slope} \times l_c \times \frac{l_c}{2} \times \frac{l_c}{3}$$

$$= 174.2 \times \frac{14.27^3}{6}$$

$$= 85000 \ \text{lb - ft}$$

Footing Slab Floor (Continued)

Check f_c

$$f_c = \frac{2M}{k\ j\ b\ d^2}$$

$$= \frac{2 \times 85000 \times 12}{.344 \times .885 \times 12 \times 20.5^2}$$

$$= 1370\ \#/in^2$$

Allowable $f_c = 1050\ \#/in^2$. This will necessitate an increase in the area of steel. The new loading will be recomputed in the same manner as above.

Try 7/8 in. ∅ @ 6 inches c/c

$A_s = 1.2\ in^2$

$\Sigma_o = 5.5$ in

Loading

$$w = \frac{f_s\ j\ d\ A_s}{1/8 \times l^2}$$

$$= \frac{20000 \times .885 \times 20.5 \times 1.2}{.125 \times 12 \times 16.5^2}$$

$$= 1070\ lbs/ft$$

Footing Slab Floor (Continued)

Length of Cantilever

$$l_c = 20 - (2 + 2 + x)$$

$$x = \frac{1070}{174.2}$$

$$= 6.14 \text{ ft}$$

$$l_c = 20 - (2 + 2 + 6.14)$$

$$= 20 - 10.14$$

$$= 9.86 \text{ ft}$$

Moment

$$M = \text{Slope} \times l_c \times \frac{l_c}{2} \times \frac{l_c}{3}$$

$$= 174.2 \times \frac{9.86^3}{6}$$

$$= 28000 \text{ lb - ft}$$

Check f_c

$$f_c = \frac{2M}{k\ j\ b\ d^2}$$

$$= \frac{2 \times 28000 \times 12}{.344 \times .885 \times 12 \times 20.5^2}$$

$$= 452\ \#/in^2 \qquad \text{Allowable } f_c = 1050\ \#/in^2$$

Footing Slab Floor (Continued)

<u>Check A_s</u>

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

$$= \frac{28000 \times 12}{20000 \times .885 \times 20.5}$$

$$= .926 \text{ in}^2$$

Use 7/8 in ø @ 6 inches c/c for widthway steel in the top section of the floor slab

$$A_s = 1.2 \text{ in.}^2$$

$$\Sigma_o = 5.5 \text{ in.}$$

Sidewalls

The loading against the sidewalls is partially transferred through the wall base and into the footing floor slab. Due to this, it will be assumed that the design load is equivalent to that caused by a fluid depth of 4 feet and weighing 33 1/3 $\#/ft^3$. As an added safety measure the entire load at this point will be considered to act on each wall, individually.

Load Acting Over 4 Ft Depth

$$= 633 - 2 \times \frac{100}{3}$$

$$= 567 \text{ lbs/ft}^2$$

Moment

$$M = 567 \times 4 \times 2$$

$$= 4536 \text{ lb - ft}$$

Effective Depth

If possible, a depth of 12 1/2 inches with a 2 1/2 inch cover will be used.

$$d = \sqrt{\frac{M}{k\ b}}$$

$$= \sqrt{\frac{4536}{160 \times 12} \times 12}$$

$$= \sqrt{28.4}$$

$$= 5.4 \text{ inches}$$

Use $d = 12\ 1/2$ inches

Sidewalls (Continued)

Check f_c

$$f_c = \frac{2M}{k\ j\ b\ d^2}$$

$$= \frac{2 \times 4536 \times 12}{.344 \times .885 \times 12 \times 12.5^2}$$

$$= 197\ \#/in^2 \qquad \text{Allowable } f_c = 1050\ \#/in^2$$

Area of Steel

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

$$= \frac{4536 \times 12}{20000 \times .885 \times 12.5}$$

$$= .246\ in^2$$

5/8" ø @ 15 inches is allowable.

However use 5/8" ø @ 12 inches for convenience in field layout.

$A_s = 31\ in^2$

$\Sigma_o = 2$ inches

"U" Bars

It is chosen to place "U" bars of 5/8" ø @ 12 inches c/c in addition to the above steel. The length of protrusion into the vertical wall is 48 bar diameters. The depth of placement into the floor slab is governed by the effective depth of the section. The horizontal portion of the "U" will be placed at the depth of the reinforcing steel of the floor slab.

Sidewalls (Continued)

<u>Temperature Steel</u>

$A_t = .002\ b\ d$

$= .002 \times 12 \times 12.5$

$= .30\ in^2$

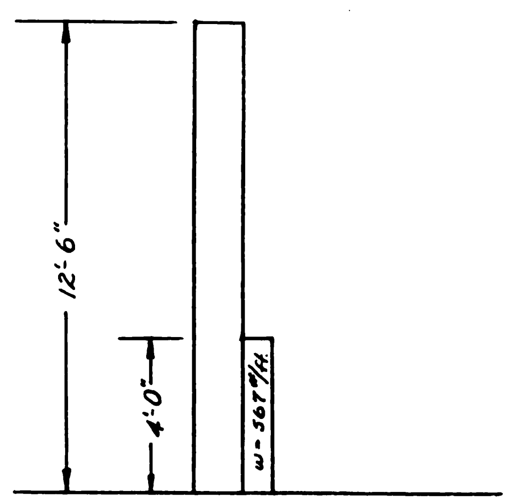
This area is divided between the front and back surface leaving an effective

$A_t = .15\ in^2$

Use 5/8" ø @ 24 c/c inches on each face

$A_t = .155\ in^2$

$\Sigma_o = 1$ inch



<u>Shear</u>

$V = wl'$

$= 567 \times 4$

$= 2268$ lbs

<u>Bond</u>

$$u = \frac{V}{\Sigma_o\ j\ d}$$

$$= \frac{2268}{2 \times .885 \times 12.5}$$

$= 103\ \#/in^2$ Allowable $u = 150\ \#/in^2$

<u>Shear Intensity</u>

$$v = \frac{V}{b\ j\ d}$$

$$= \frac{2268}{12 \times .885 \times 12.5}$$

$= 17.1\ \#/in^2$ Allowable $v = 60\ \#/in^2$

Front Wall

The loading against the front wall is partially transferred through the base of the wall and into the floor slab of the footing. Due to this, it will be assumed that the design load is equivalent to that caused by a 33 1/3 $\#/ft^3$ fluid at the bottom 4 ft of the pressure triangle. As an added safety measure the entire load is considered to act only on the front wall.

<u>Load Acting Over 4 ft. Depth</u>

$$W = 633 - 2 \times \frac{100}{3}$$

$$= 567 \text{ lbs/ft}^2$$

<u>Moment</u>

$$M = 567 \times 4 \times 2$$

$$= 4536 \text{ lb - ft}$$

<u>Effective Depth</u>

The wall thickness is 24 inches. We desire a cover of 2 1/2 inches so d = 21 1/2 inches if allowable.

$$d = \sqrt{\frac{M}{k\ b}}$$

$$= \sqrt{\frac{4536 \times 12}{160 \times 12}}$$

$$= \sqrt{28.4}$$

$$= 5.4 \text{ inches}$$

Use d = 21 1/2 inches

Front Wall (Continued)

<u>Check f_c</u>

$$f_c = \frac{2M}{k\ j\ b\ d^2}$$

$$= \frac{2 \times 4536 \times 12}{.344 \times .885 \times 12 \times 21.5^2}$$

$= 66\ \#/in^2$ Allowable $f_c = 1050\ \#/in^2$

<u>Area of Steel</u>

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

$$= \frac{4536 \times 12}{20000 \times .885 \times 21.5}$$

$= .142\ in^2$

Use 5/8" ø bars @ 24 inches c/c

$A_s = .155\ in^2$

$\Sigma_o = 1.0$

"U" bar of 5/8" ø @ 12 inches c/c will also be used as was done in the sidewalls. The above steel at 24 inch spacing will be placed on alternate "U" bars.

<u>Temperature Steel</u>

$A_t = .002\ b\ d$

$= .002 \times 12 \times 21.5$

$= .52\ in^2$

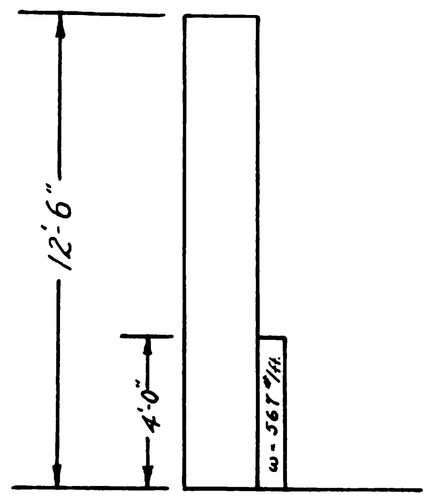
This area is divided between the two wall faces leaving an effective $A_t = .26\ in^2$

Use 5/8 in ø bars @ 12 inches c/c

$A_t = .31\ in^2$

$\Sigma_o = 2.0$

Front Wall (Continued)



Shear

$$V = wl'$$
$$= 567 \times 4$$
$$= 2268 \text{ lbs}$$

Bond

$$u = \frac{V}{\Sigma_o \; j \; d}$$
$$= \frac{2268}{2 \times .885 \times 12.5}$$
$$= 103 \; \#/in^2$$

Allowable $u = 150 \; \#/in^2$

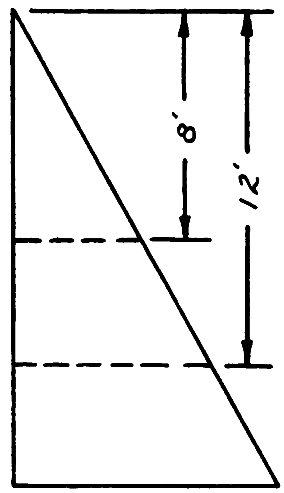
Shear Intensity

$$v = \frac{V}{b \; j \; d}$$
$$= \frac{2268}{12 \times .885 \times 12.5}$$
$$= 17.1 \; \#/in^2$$

Allowable $v = 60 \; \#/in^2$

Horizontal Reinforcement

Sidewalls



Loading

The load acting on the wall varies with the height of the wall and is greatest at the bottom. Therefore it is not necessary to run the same area of steel the entire height of the wall. Loadings will be assumed as those caused by a 33 1/3 $\#/ft^3$ fluid at top 8 feet and 12 feet of the pressure triangle. Results for the "8" foot computations will be 2/3 of those found for the 12 foot loading.

$$W = lh$$
$$= 33\ 1/3 \times 12$$
$$= 400\ \#/ft$$

Horizontal Reinforcement

Sidewalls

Moment

Each sidewall is restrained at one end and is considered to be simply supported at the other. Therefore the maximum moment would not exceed $1/10\ wl^2$ x 1 = 16 ft

$$M = 1/10\ wl^2$$

$$= 1/10 \times 400 \times 16^2$$

$$= 10{,}240 \text{ ft lbs}$$

Check f_c

$$f_c = \frac{2M}{k\ j\ b\ d^2}$$

$$= \frac{2 \times 10240 \times 12}{.344 \times .885 \times 12 \times 12.5^2}$$

$$= 430\ \#/in^2 \qquad \text{Allowable} = 1050\ \#/in^2$$

Area of Steel

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

$$= \frac{10240 \times 12}{20000 \times .885 \times 12}$$

$$= .557\ in^2$$

Horizontal Reinforcement

Sidewalls

Area of Steel (Continued)

Each sidewall offers a certain restraint to the front wall requiring an area of steel to withstand this.

Loading $= 15 \times 400 = 6000$ lbs

Reaction $= 6000 \times 1/2 = 3000$ lbs

$$A_{s_r} = \frac{3000}{20000} = .15 \text{ in}^2$$

Add $.08 \text{ in}^2$ of steel to each face of wall to withstand the stress of restraint.

$$A_s = .557 + .08$$
$$= .637 \text{ in}^2$$

Use 3/4 in ø bars @ 8 inches c/c on each face of wall. These will extend 9 ft up from the base of the wall.

For the 8 ft Loading

Area Steel

$$A_s = 2/3 \times .637$$
$$= .425 \text{ in}^2$$

Use 5/8 in ø @ 8 inches c/c

Horizontal Reinforcing

Front Wall

Moment

$M = 1/10\ wl^2$

$w = 400\ \#/ft$

$l = 17.5 - 1.25$

$= 16.25\ ft$

$= 1/10 \times 400 \times 16.25^2$

$= 10560\ ft\ lbs$

Check f_c

$$f_c = \frac{2M}{k\ j\ b\ d^2}$$

$$= \frac{2 \times 10560 \times 12}{.344 \times .885 \times 12 \times 21.5^2}$$

$= 150\ \#/in^2$

Allowable $f_c = 1050\ \#/in^2$

Horizontal Reinforcing

Front Wall (Continued)

Area of Steel

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

$$= \frac{10560 \times 12}{20000 \times .885 \times 21.5}$$

$$= .333 \text{ in}^2$$

Additional steel is required to withstand the stress of the restraint which must be offered to the sidewalls

Loading $= 400 \times 16 = 6400$ lbs

Reaction $= 6400 \times 1/2 = 3200$ lbs

$$A_{s_r} = \frac{3200}{20000} = .16 \text{ in}^2$$

Add .08 in^2 of steel to each face of the wall.

$$A_s = .333 + .08$$

$$= .413 \text{ in}^2$$

Use 5/8 in ϕ bars at 8 inches c/c ($A_s = .47 \text{ in}^2$)

This is greater than required but the spacing is desirable for convenience in layout.

Topwalls

Front

Steel (Horizontal)

Use 5/8 ø @ 8 inches c/c for convenience in field layout. $A_s = .47 \text{ in.}^2$

Steel (Vertical)

$A_s = .002\ b\ d$

$= .002 \times 12 \times 6.5$

$= .156$

Use 1/2" ø @ 12" c/c for convenience in field layout.

$A_s = .25 \text{ in.}^2$

Vertical "Acting" Beam

A tie beam is to be used at the upper rear corner of the side walls to render support against the outward thrust acting on each side wall. If possible, reinforcing steel of sufficient area will be used vertically at the outer end of each side wall to act as a vertical beam. The horizontal tie beam (12" x 18" x 15') is connected between these acting beams and provides the required support. The wall is considered fixed on one end and supported at the other.

<u>Loadings on Side Walls</u>

P = w h x 1/2 the distance from the front wall to the center of the horizontal tie beam.

Top of Side Wall

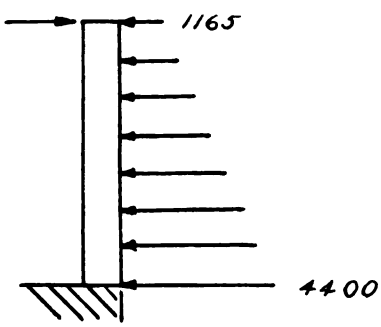
$$P_t = \frac{100}{3} \times 4.5 \times \frac{15.5}{2}$$

$$= 1165 \text{ lbs}$$

Base of Side Wall

$$P_b = \frac{100}{3} \times 17 \times \frac{15.5}{2}$$

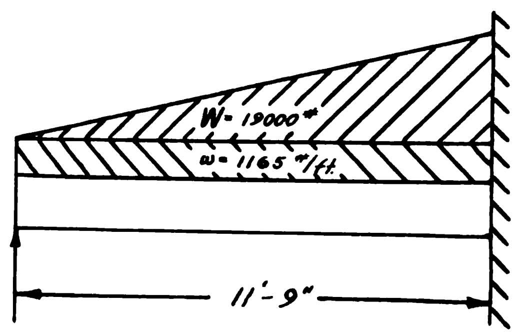
$$= 4400 \text{ lbs}$$



Vertical Acting Beam (Continued)

Moment

The moment will be the summation of the moment of a uniform loading and that of a triangular loading acting over a span length of 11.75 feet.



$W = 3235 \times 11.75 \times 1/2$

$= 19000$ lbs

$M = 1/8\ w\ l^2 + .128\ W\ l$

$= .125 \times 1165 \times 11.75^2 + .128 \times 19000 \times 11.75$

$= 20100 + 28600$

$= 48700$ ft - lbs

Minimum Value of "b"

$$b = \frac{M}{k\ d^2}$$

$$= \frac{48700 \times 12}{160 \times 12.5^2}$$

$= 23.8$ inches

Vertical Acting Beam (Continued)

Area of Steel

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

$$= \frac{48700 \times 12}{20000 \times .885 \times 12.5}$$

$$= 2.65 \text{ in}^2$$

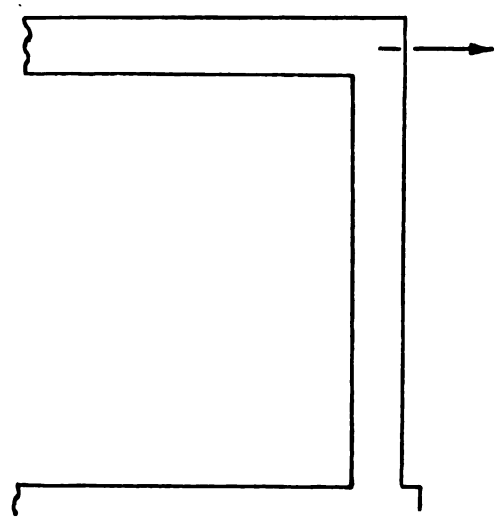
Use 5 - 7/8" ø bars @ 6 inch c/c

$$A_s = 3 \text{ in}^2$$

$$\Sigma_o = 13.75 \text{ inches}$$

Horizontal Tie Beam

The total reaction on the beam will be an axial load in tension. It is the summation of reactions caused by a uniform loading and a triangular loading from the base of the wall to the center of the horizontal beam. The wall will be considered as fixed at one end and supported at the other.



<u>Reaction</u>

$$R = R_u + R_t$$

$$R_u = w \frac{l}{2} \qquad\qquad R_t = 1/3 \times P \frac{l}{2}$$

$$= 1165 \times \frac{11.75}{2} \qquad\qquad = 1/3 \times 3235 \times \frac{11.75}{2}$$

$$= 6850 \text{ lbs} \qquad\qquad = 6350 \text{ lbs}$$

$$= 6850 + 6350$$

$$= 13200 \text{ lbs}$$

Horizontal Tie Beam (Continued)

Moment

Due to the beam being axially loaded the moment is that which is caused by the dead load of the beam.

Dead Load

$$w = 18/12 \times 1 \times 150$$
$$= 225 \text{ lbs/ft}$$

As a safety measure in case of any settling of the fill this value is doubled

Use $w = 450$ lbs/ft

It is assumed that the maximum moment will be less than that for a simply supported beam with uniform loading, $M = 1/8\ w\ l^2$, and greater than that for a fixed end beam with uniform loading, $M = 1/12\ w\ l^2$.

Use $M = 1/10\ w\ l^2$ $\quad l = 16.25$ ft c/c of walls

$$= \frac{450 \times 16.25^2}{10}$$

$$= 11900 \text{ ft - lbs}$$

Horizontal Tie Beam (Continued)

Effective Depth

$$d = \sqrt{\frac{M}{k\ b}}$$

$$= \sqrt{\frac{11900 \times 12}{160 \times 12}}$$

$$= 8.64 \text{ inches}$$

Use $d = 9.5$ inches

This allows the use of a 12" x 12" beam instead of a 12" x 18".

Area of Steel

$$A_{s_1} = \frac{M}{f_s\ j\ d}$$

$$= \frac{11900 \times 12}{20000 \times .885 \times 9.5}$$

$$= .85 \text{ in}^2$$

Additional steel will be added to withstand the axial loading on the beam.

$$A_{s_2} = \frac{R}{f_s}$$

$$= \frac{13200}{2000}$$

$$= .68 \text{ in}^2$$

Add $.34 \text{ in}^2$ to each section

Horizontal Tie Beam (Continued)

Area of Steel (Continued)

$$A_s = A_{s_1} + A_{s_2}$$

$$= .85 + .34$$

$$= 1.19 \text{ in}^2$$

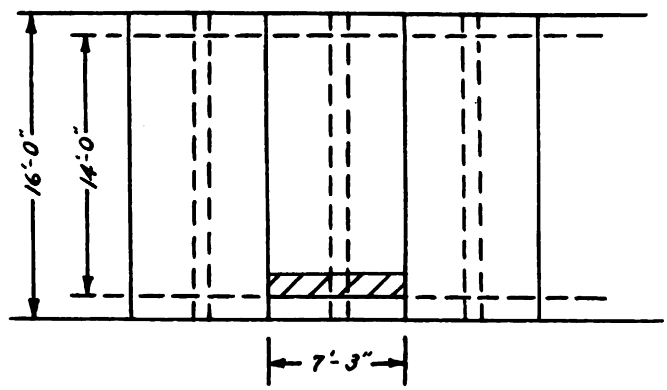
Use 2 - 7/8" ø bars top and bottom with right angle bends extending downward into each wall.

$L = 36$ inches

$A_s = 1.2$ in^2/ft/section

$\Sigma_0 = 5.5$ in^2/ft/section

Floor Beam



Loading Per Foot of Beam

Assume a 10 - wf 21 beam

Slab Load

$\frac{4 + 5.5}{2} \times 1/12 \times 150 \times 7.25 = 435$ #/ft

Live Load

100 x 7.25 = 725 #/ft

Beam Weight

Assume 21 #/ft = 21 #/ft

Total Load w = 1181 #/ft

Floor Beam (Continued)

Moment

$M = 1/8\ w\ l^2$ $\qquad l = 14$ ft (face to face)

$= 1/8 \times 1181 \times (14)^2 \times 12$

$= 347000$ in - lbs

Section Modulus

$S = \frac{M}{Z}$

$Z = \frac{347000}{18000}$

$= 19.3\ in^3$

Allowable $Z = 21.5\ in^3$

Use a 10 WF 21 beam

Plate Girder

<u>Panel Loads</u>

Each girder is going to carry 1/2 of the total loading.

Slab Load $= \frac{4 + 5.5}{2} \times 1/2 \times 150 \times 7 =$ 415 #/ft

Live Load $= 100 \times 7 \times 1$ $=$ 700 #/ft

Girder $=$ Assume 400 #/ft $=$ 400 #/ft

Total $w =$ 1515 #/ft

7' - 3" Panel

$P = 1515 \times 7.25$

$= 11$ kips

The loading for the two end panels is going to be computed from the average span of 7' - 3" and 7' - 1 1/2".

$P = 1515 \times 7.19$

$= 10.9$ kips

In computing moments and shears, loadings will be considered as carried to the girders through the floor beams thus rendering concentrated loads at each panel point.

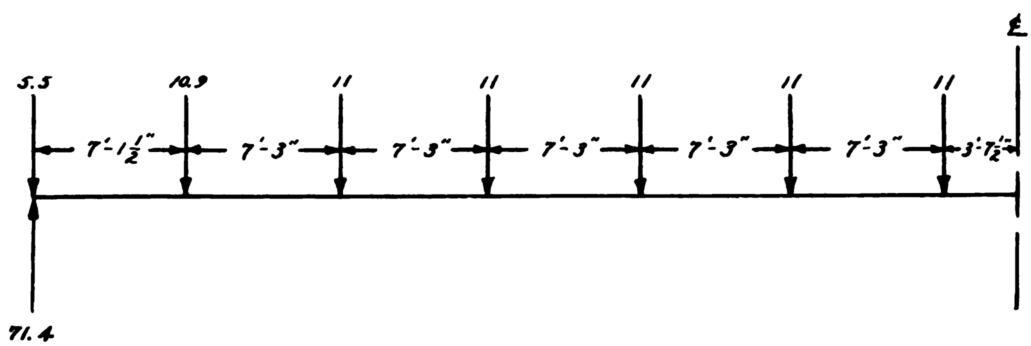


Plate Girder (Continued)

Depth of Girder

$d = 1/25$ span

$= 1/25 \times 94$ Use 48 inches at center and 66

$= 3.76$ ft inches at supports.

Thickness of Web (AASHO Spec. 3.6.75)

At Center

$t_c = 1/20\ D$ D = Depth of Web

$= 1/20\ 46$ Assume 46 3/8 in

$= .34$ inches

Use 3/8 inch web ($t = .375$ inches) at center.

At Support

$t_s = 1/20\ 66$

$= .403$ inches

Referring to AISC Spec. section 26-b

$t_e = 1/170 \times 66$

$= .388$ inches

Use 3/8 inch web at supports. This is under specifications but is considered allowable due to the shape of the girder, its light loading, and the fact that the bridge is subject only to pedestrian traffic. No specifications are established for purely pedestrian loadings.

Plate Girder (Continued)

<u>Moments and Shears</u>

$$M_{max} = 65.9 \times 47 - 11 \times 3.625 - 11 \times 10.875 - 11 \times 18.125 - 11 \times 25.375 - 11 \times 32.625 - 10.9 \times 39.875$$

$$= 3100 - 39.8 - 119.8 - 199.2 - 278.2 - 358.0 - 438.0$$

$$= 3100 - 1433$$

$= 1667$ kip - ft at the center of the span

$$V_{max} = 11 + 11 + 11 + 11 + 11 + 10.9 + 5.5$$

$= 71.4$ kips at support

$M_o = 0$ kip - ft	$V_o = 71.4$ kips
$M_1 = 470$ kip - ft	$V_{o-1} = 65.9$ kips
$M_2 = 868$ kip - ft	$V_{1-2} = 55$ kips
$M_3 = 1187$ kip - ft	$V_{2-3} = 44$ kips
$M_4 = 1429$ kip - ft	$V_{3-4} = 33$ kips
$M_5 = 1575$ kip - ft	$V_{4-5} = 22$ kips
$M_6 = 1656$ kip - ft	$V_{5-6} = 11$ kips
	$V_{6-7} = 0$ kips

Plate Girder (Continued)

<u>Check Thickness Against Shear</u>

Maximum shear will occur at the end reaction with half going into each girder.

$$V = 1/2 \times 143$$

$$= 71.5 \text{ kips}$$

$$S_s = \frac{P}{A}$$

$$= \frac{71500}{66 \times .375}$$

$$= 2890 \text{ \#/in}^2$$

Allowable:

A.I.S.C. Specs $= 13000$ #/in^2

<u>Stiffeners</u> (A.I.S.C. 26-e)

Intermediate stiffeners are required if

$$\frac{h}{t} = 70$$

$$\frac{h}{t} = \frac{66}{.375} = 176$$

Stiffeners are required.

Stiffener Spacing

$$d = \frac{11000\ t}{S_s}$$

$$= \frac{11000 \times .375}{2890}$$

$$= 77 \text{ inches maximum allowable}$$

Plate Girder (Continued)

Intermediate Stiffeners

The maximum allowable spacing is found to be 77 inches. Stiffeners will be placed on both sides of web at each welded joint of web; and singly on the inside of web at each panel point and at the midpoint of each panel. This is allowable because of the light loading.

$w = D$ approximately

$t = 1/12$ width

Use 5" x 7/16" stiffener plates for a total of 29 per girder.

Welds

3/4" rivets @ 5" c/c to 3/8" web = 11200 lbs

4 - 1/4" fillet welds will withstand 9600 lbs

If spacing is in 6" lengths

$$L \times 5/6 = \frac{11200}{9600}$$

$$L = 1.4 \text{ in}$$

Use 2 inch length of 1/4" fillets in each 6 inches of length.

Plate Girder (Continued)

Data (At Center of Span)

Trial Web = 46 3/8" x 3/8"

Assumed distance c/c gravity = 47 inches

Assumed overall depth = 49 inches

Maximum Moment = 1667 kip ft

Reduction of Flange Stress

$$S = 18000 \times \frac{47}{49}$$

$$= 17250 \ \#/in^2$$

Required Gross Flange Area

A = Net flange area minus 1/8 gross web area

$$= \frac{1667 \times 12}{47 \times 17.25} - \frac{46.375 \times .375}{8}$$

$$= 24.7 - 2.2$$

$= 22.5\ in^2$ required in each flange

Top Flange

Use a 15" channel @ 50#/ft = 14.64

Add a 14" x 5/8" cover plate = 8.75

Total Area A = $23.39\ in^2$

Bottom Flange

The flange plate will be formed by splitting a 30" x 1" plate

Use a 15" x 1" plate = 14.64

Add a 14" x 5/8" cover = 8.75

Total Area A = $23.39\ in^2$

Plate Girder (Continued)

Check Stress

Maximum moment occurs at center

$$S = \frac{Mc}{I}$$

Top Flange

$$S = \frac{1667000 \times 12 \times 23.45}{29340}$$

$$= 16000 \ \#/in^2$$

Bottom Flange

$$S = \frac{1667000 \times 12 \times 24.7}{29340}$$

$$= 16850 \ \#/in^2$$

Allowable $= 18000 \ \#/in^2$

Point of Cutoff

$$M = \frac{SI}{c} \qquad I = I_w + I_c + I_{15} = 19040$$

$$= \frac{18000 \times 19040}{23.11}$$

$$= 1230 \text{ kip - ft}$$

Moment @ panel point No. 3 is 1187 kip - ft.

Cut 14" plate at points No. 3 and No. 10.

(25' - 4 1/2" from center line)

Plate Girder (Continued)

Center of Gravity of Girder

$$\bar{y} = \frac{A_w\bar{y}_w + A_c\bar{y}_c + A_{14}\bar{y}_{14} + A_{15}\bar{y}_{15} + A_{14}\bar{y}_{14}}{A_w + A_c + A_{14} + A_{15} + A_{14}}$$

$$A = 17.4 + 14.64 + 8.75 + 14.64 + 8.75$$

$$= 64.2$$

$$= \frac{(17.4 \times 24.81) + (14.64 \times 47.92) + (8.75 \times 49.03)}{64.2}$$

$$\frac{+ (14.64 \times 1.13) + (8.75 \times .31)}{64.2}$$

$$= \frac{432 + 702 + 429 + 16.6 + 2.7}{64.2}$$

$$= \frac{1582.3}{64.2}$$

= 24.70 in. from bottom of lower flange

or 23.08 in. from bottom of web

Moment of Inertia

$$I_w = 1/12\ bh^3 + Ad^2 = \frac{.375 \times 46.375^3}{12} = 3100$$

$$I_c = Ad^2 = 14.64 \times 23.03^2 = 7790$$

$$I_{14} = Ad^2 = 8.75 \times 24.13^2 = 5090$$

$$I_{15} = Ad^2 = 14.64 \times 23.58^2 = 8150$$

$$I_{14} = Ad^2 = 8.75 \times 24.39^2 = \underline{5210}$$

$$I = 29340 \text{ in.}^4$$

Plate Girder (Continued)

<u>Flange Welds</u>

The maximum shear will occur at the end of reaction.

D $=$ 66 inches

C.G. $=$ 32.9 inches from bottom of the web

V $=$ 71,500 lbs.

I $=$ 41,390 in.^{4}

$$S_s = \frac{VQ}{Ib}$$

$$= \frac{71500 \times 14.64 \times 33.82}{41390 \times 1}$$

$=$ 358 #/inch

Use 2 ft continuous from ends then

use 1/4" fillets (2400 #/in)

stitched 2" in 6".

Slippage will occur between the plates making up the flange. The maximum value will occur at the point of cutoff.

D $=$ 52 inches

C.G. $=$ 26.08 inches from bottom of web

V $=$ 44,000 lbs.

I $=$ 36,490 in.^{4}

$$S_s = \frac{44000 \times 8.75 \times 26.64}{36490}$$

$=$ 280 #/inch

Use 1/4" fillets (2400 #/in)

stitched 2" in 6".

Plate Girder (Continued)

<u>End Stiffeners</u>

Area Reqd. $= \frac{P}{S}$

$= \frac{71500}{17000}$

$= 4.2 \text{ in.}^2$

Use two pair of intermediate stiffeners at each end of each girder.

Welding Required

Each pair of end stiffeners will transfer 2/3 of the end shear.

$P = 2/3 \times 71500$

$= 47700$

Length of 1/4" fillet weld $= \frac{47700}{2400}$

$= 20$ inches

Use 12" continuous at each end then stitch 2" in 6".

Plate Girder (Continued)

Check Weight $\qquad$ Steel Weight = 490 $\#/ft^3$

Web

$(46\ 3/8" \times 47' + 19\ 1/2" \times \frac{47'}{2}) \times 3/8" \times 490 = 3340$

Plates

$2 \times 14" \times 5/8" \times 50' - 9" \times 490 = 3420$

$1 \times 15" \times 1" \times 94' \times 490 = 4800$

Channel

$1 \times 94' \times 50\ \#/ft = 4700$

Stiffeners

$29 \times 5" \times 7/16" \times 56" \times 490 = 1210$

$4 \times 5" \times 7/16" \times 66" \times 490 = 170$

= 17640 lbs

$\frac{17640}{94} = 188\ \#/ft$ of girder

Reaction Pin

A 16" roller segment with a 24" diameter will be used. Through this, two 1 1/2" holes will be drilled to hold 1 1/4" anchor bolts, thus allowing for deflection.

Length = 16 - 2 x 1 1/2

l = 13 in.

Stress $= \frac{P}{l}$

$= \frac{71500}{13}$

= 5500 #/lineal inch

Allowable = 600D

= 14,400 #/lineal inch

Plate Girder (Continued)

Masonry Bearing Plate

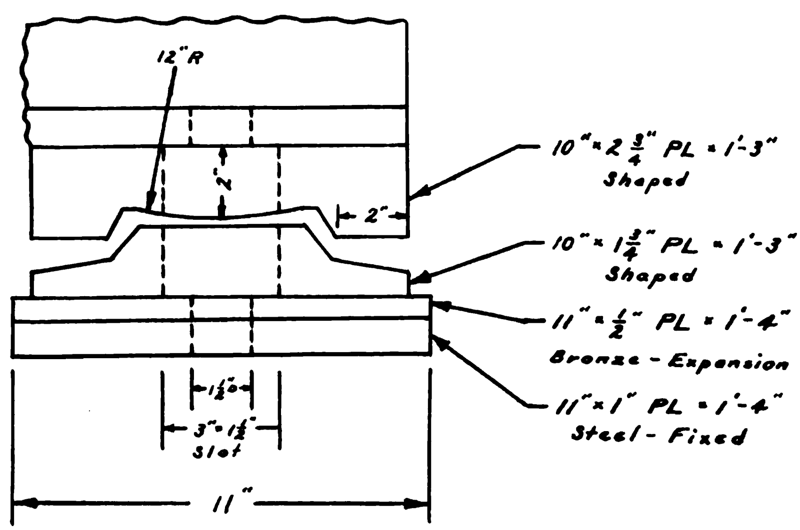
$f'_c = 600\ \#/in^2$

$S = \frac{P}{A}$

$A = \frac{71500}{600}$

$= 120\ in.^2$

Use a 16" x 11" plate, A = 176 in.^{2} The thickness will be 1 1/2" consisting of 1" of steel acting as a fixed plate faced with a 1/2" bronze expansion plate.



14' FOOT BRIDGE ACROSS RED CEDAR

MICHIGAN STATE COLLEGE

SCALE 1/8" = 1'-0"

#1

PRELIMINARY SKETCH.

FEB. 28, 1949

CLAUD R. ERICKSON
CONSULTING · ENGINEER.
LANSING

FOOT BRIDGE ACROSS RED CEDAR
MICHIGAN STATE COLLEGE
SCALE 1/8" = 1'-0"

#3

PRELIMINARY SKETCH
2-28-49

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