THE DESIGN AND CONSTRUCTION

of

A BRIDGE OVER THE BROAD RIVER ON GEORGIA STATE ROUTE #8

A Thesis submitted by

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SUB-COMMITTEE ON APPROVAL OF C.E. DEGREES.
P R E F A C E

This paper deals with the location, design and construction of a bridge over the Broad River on Georgia State Route No. 8 between Danielsville and Royston.

This work was done under the supervision of the State Highway Board of Georgia and the Federal Bureau of Public Roads. The writer was Resident Engineer on the construction of the Broad River bridge, a bridge over the Hudson River and the adjacent grading.

Invaluable assistance in the preparation of this paper has been rendered by Mr. W. L. Robinson and Mr. C. N. Crocker of the Georgia State Highway Department.
LOCATION

The location on which this bridge is constructed is one of the two practical routes between Danieleville and Royston. This location crosses both the Broad River and the Hudson River, while the alternate route crosses the Broad River at a point below its junction with the Hudson. The alternate route would have eliminated one bridge, but would have traversed a thinly populated section entirely off the previously travelled road. The slight difference in cost between the two lines was therefore disregarded, and the present location constructed, as it is far more beneficial to local traffic, and the difference in distance is not great.

DESIGN

Design No. 1 is largely a check of the existing structure. Quantities from this design are approximately the same as those of the structure in place.

Design No. 2 is for a one hundred foot arch in place of the two fifty-two foot decks. The approach spans are the same as in design No. 1. This design is submitted merely for the sake of comparison.

The construction of this bridge was done under the State Highway Board of Georgia's specifications of 1927 which were in effect from that year until September 1930. These specifications are therefore followed in the designs submitted.

The arch design is made by the method given by Turnesure and Maurer in their 'Principles of Reinforced Concrete Construction.'

CONSTRUCTION

Foundations

All foundations were solid rock, but the formation encountered presented unusual difficulties. The profile sheet shows the actual foundation elevations, but a heavily folded material of varying hardness was encountered several feet above these elevations and it was necessary to shoot a
number of the foundations and key into the rock.

The end bent and intermediate bent footings were poured in the dry with very little water entering the excavation. Timber sheeting was used and gave no trouble even when using dynamite to level the foundations.

Footings for the outwater bents were all in the stream and an attempt to sheet with timber proved useless. Steel sheets were used with a timber sheeted clay chamber. The rock encountered in these footings was practically level so that the use of explosives was not necessary, but it was impossible to dry the foundations completely. As it was necessary to pour under water the size of the footings was increased from 8' x 6' to 10' x 10'. As no equipment for handling a tremie or dump bucket was available, an improvised tremie of short length handled by two men was used. A form was set and made as nearly water tight as possible. The foundation was carefully cleaned and the tremie placed on a dry spot and charged. Concrete was shovelled into the tremie from a hopper placed at the end of a chute inside the hole. After the tremie was charged the water was allowed to rise two or three feet until no current moved through the form. The concrete was then carefully deposited in a horizontal layer sufficiently thick to completely seal the form. The water was then pumped out as low as possible and the pouring continued dry. The concrete placed under water was found to be of good quality when exposed. The 10' x 10' base was brought up several feet and then stepped down to 8' x 8'.

False work

False work was built up from piling cut off and capped about eight feet above water level. Pine poles were used throughout for compression members. A stress of 650 pounds per square inch was allowed on short sticks and the unsupported length held to twelve feet, correction for
bending being made on this length. Struts were placed from pier to pier at the tops of the piles. Bents were placed approximately seven feet center to center and each bent consisted of six poles, one for each girder and one under each curb. Both tiers were cross braced longitudinally and transversely with two by six sawed pine spiked with sixty penny nails.

**Superstructure and Rail**

Two mixers, one 5 - s and one 10 - s were used in pouring the decks and it was thus possible to pour a fifty-two foot deck in seven hours. About two hours of this time was consumed by pouring the curbs and setting chamfer strips and panels. The curbs were only poured to the slab level as the girders and slab were poured. After the pouring was almost complete and sections adjacent to the curbs fully loaded, panels and chamfers were set to the proper elevations, all laitance was removed from the curbs and the pouring completed to the chamfer line.

Hand rail posts were lined top and bottom with the transit and elevation for the rail set with the level at each post. As the grade is a very light parabola, it was necessary to set and check the rail points very carefully.
DESIGN NO. 1

2 Deck Girder Spans - Length 52'
4 Deck Girder Spans - Length 40'

Substructure -

End Bent - Station 624/36
Intermediate Bent - Station 624/78
Cutwater Bent - Station 625/16
Cutwater Bent - Station 625/70
Cutwater Bent - Station 626/22
Intermediate Bent - Station 626/62
End Bent - Station 627/02

Width of Roadway - 20'
Loading - R - 15
Bearing for Decks Assumed - 12"

In general the specifications of the State Highway Department of Georgia prior to September 1, 1931 have been followed in this design.
Assumed Section of 52' Span

Typical 15 Ton Truck.
Load on One Front Wheel - 3000#
" " " Rear " - 12000#
52' Decks

Slab

Bending computed as 80% of bending on a simple span of the length of the clear span. Clear span = 4' - 4". 1 Rear Wheel = 12,000 lbs.

Longitudinal Distribution:

\[ E = \text{Effective width in feet for one wheel.} \]
\[ X = \text{Distance in feet from center of nearest support to center of wheel.} \]
\[ T = \text{Width of wheel in feet.} \]
\[ E = 0.7 (2X / T) = 5.13' \]
\[ \text{Impact factor} = \frac{50}{(1.25 + 125)} = 0.396 \]

Effective L.L. at center per foot of slab = \( \frac{12,000 \times 1.396}{5.13} = 3,860 \text{ lbs.} \)

Dead Load on slab:

Concrete in slab = 7 x 1" = 84 lbs. per sq. ft.

Steel (assumed) = 3" x 1" x 1" x 1" = 150 lbs. per sq. ft.

Paving = 50 lbs. per sq. ft.

Total = 137 lbs. per sq. ft.

L.L. B.M. = \( \frac{0.6 \times 3,860 \times 5.17}{2} = 2,830 \text{ ft. lbs.} \)

D.L. B.M. = \( \frac{6 \times 137 \times 4.33^2}{6} = 257 \text{ ft. lbs.} \)

Total B.M. = 3,087 ft. lbs.

Resisting moment of 7" slab:

\[ d = 5\frac{1}{2}" , \quad kd = 1.06" , \quad jd = 0.401' \]
\[ .401 \times 2.06 \times 12 \times 325 = 3,710 \text{ ft. lbs.} \]
\[ A_s = bd \times 0.0077 = 5.5 \times 12 \times 0.0077 = 0.506 \text{ sq. in.} \]

Use 5/6" Ø 6.5" C. to C.
Sening on Cantilever -

Dead Load:-

Curb - $12 \times 18 = 216$ lbs.
Slab - $7 \times 18 = 126$ lbs.
Steel - $3 \times 2 = 6$ lbs.
Rail (assumed) - $200$ lbs.

Total - $548$ lbs.

$S.M. = 548 \times 1 = 548$ ft. lbs.
52' Deck

Interior Girder

D.L. per Lin. Ft.:

31ab - 137 x 6.06 = 833 lbs.
3stem - 49 x 21 = 1,029 lbs.
Gird. Steel (assumed) = 60 lbs.
Total - 1,922 lbs.

L.L. - 15 ton truck - wheel loads - 3,000 lbs. and 12,000 lbs.

Impact factor = \( \frac{50}{128+14} = 0.36 \)

\( 1.36 \times 3,000 = 4,080 \) lbs. (front wheel).

\( 1.36 \times 12,000 = 16,320 \) lbs. (rear wheel).

Lateral Distribution

\( 6.06 = 1.35 \)

\( 4.5 \)

\( 1.35 \times 4,080 = 5,520 \) lbs. (on Int. Girders)

\( 1.35 \times 16,320 = 22,040 \) lbs. (on Int. Girders)

Bending -

\( R-1 = \frac{(5,520 \times 36.1)}{51} - 52,610 \) lbs.

\( R-2 = \frac{5,520}{22,040 \times 1,922 \times 25} = 61,050 \) "

B.M. \( \frac{(61,050 \times 24.1)}{24.1} = 1,922 \times 24.1 \times 12.05 \times 51,000 \) ft. lbs.

\( d = 51", \quad 12 = \frac{44.625}{3.72} = 12 \) ft. lbs.

\( A_s = \frac{912,000}{(3.72 \times 15,000)} = 15.35 \) sq. in.

\( k_d = \frac{2nd \ As}{2bt} = \frac{(2 \times 15 \times 15.35 \times 51)}{(2 \times 15 \times 12.35 \times 51)} - 27,026 = 16.5 \) in.

\( 2 = \frac{3kd - 2t \times t}{2kd - t} = \frac{(3 \times 18.5) - 14 \times 7}{3 \times 18.5 - 7} = 3.2 \) in.
\[ jd = d - z = 51 - 3.2 = 47.8 \text{ in.} = 3.98 \text{ ft.} \]

\[ fs = \frac{912,000}{(3.98 \times 15.35)} = 15,000 \text{ lbs. per sq. in.} \]

\[ fc = \frac{912,000 \times 18.5}{(3.96 \times 72 \times 7)} = 550 \text{ lbs. per sq. in.} \]
52' Deck
Shear on Interior Girders
Scales - $\frac{3}{32}'' = 1' \times 1'' = 10,000\#$.
52' Deck

Sending on Exterior Girder

Dead Load -

Slab - 65 x 7 = 455 lbs.
Slab steel - 7 x 3 = 21 lbs.
Curb - 18 x 12 = 216 lbs.
Gird. steel (assumed) = 60 lbs.
Paving - 4 x 50 = 200 lbs.
Rail (assumed) = 200 lbs.
Stem -(assumed 21x48) = 1,006 lbs.

2,160 lbs.

Live Load -

15 ton truck - wheel loads - 3,000 lbs. and 12,000 lbs.
Max. Load - 1 set wheel loads. Impact factor = $\frac{50}{125 \times 14} = 0.36$

$1.36 \times 3,000 = 4,080$ lbs. $1.36 \times 12,000 = 16,320$ lbs.

$R-2 = (12.9 \times 4,080) + (26.9 \times 16,320) - (2,160 \times 25) = 63,640$ lbs.

$R-1 = 4,080 + 16,320 + (2,160 \times 50) - 63,640 = 64,760$ lbs.

3.61. = (24.1 x 63,640) - (24.1 x 2,160 x 12.05) = 906,000 ft. lbs.

d = 50", jd = 43.75" = 3.65", kd = 18.75", $As = \frac{906,000}{3.65 \times 16,000} = 15.6$ sq.in.

$kd = \frac{2nd \times As}{bt^2} = \frac{(30 \times 50 \times 15.6)}{30 \times 15.6} = \frac{26,536}{1,364} = 19.45"$

$z = \frac{(3kd - 2t) \times t}{(2d - t)} = \frac{(56.35 - 14) \times 7}{38.90 - 7} = \frac{44.35 \times 7}{31.90 \times 3} = 3.24"$

$jd = 50 - 3.24 = 46.76" = 3.69$, $fs = \frac{906,000}{15.6 \times 3.69} = 14,950$ lbs. per sq.in.

$fc = \frac{906,000}{(3.69 \times 64 \times 7)} = 623$ lbs. per sq. in.

Use 10 - 1\(\frac{1}{2}\) in. sq. bars.
52' Deck - Exterior Girder
Position of Live Load for Max.
Bending.
Shear on Exterior Girder.

\[ R-2 = \frac{(12.240 \times 6) + (4.080 \times 36)}{50} \times (25 \times 2,160) \times 16,320 = 74,726 \text{ lbs.} \]

\[ R-1 = 140,640 - 74,726 = 65,914 \text{ lbs.} \]

\[ v = \frac{74,726}{21 \times 43.75} = 82 \text{ lbs. per sq. in.} \]

\[ V' = 74,726 - (40 \times 21 \times 43.75) = 37,926 \text{ lbs.} \]

Vertical Component - 1\(\frac{1}{3}\) in. bar = 1.56 x 16,000 x .707 = 17,630 lbs.

- Bend up 2 at 3' - 3''
- Bend up 2 at 5' - 9''
- Bend up 2 at 8' - 3''

Use 5/8'' stirrups throughout to support slab steel at 18'' C. to C.
52' Deck
Shear on Exterior Girders.
Scales: $\frac{3}{32}$" = 1'

1" = 10,000#
40' Deck

Slab same as 52' Deck

Sending in Interior Girders -

Dead Load -

Slab - 6.08 x 137 833 lbs.
Stem - 34 x 21 714 lbs.
Steel - (assumed) 53 lbs.
Total - 1,600 lbs.

Live Load -

Same as for 52' Deck.

\[
R-1 = \frac{(5520 \times 31.6)}{39} \div \frac{(22,040 \times 17.6)}{39} \div \frac{(19 \times 1,600)}{39} = 44,530
\]

\[
R-2 = \frac{36 \times 1,600}{5,520} \div \frac{22,040}{44,530} = 43,530 \text{ lbs.}
\]

3.M. = \(43,530 \times 17.6\) - \(17.6 \times 1,600 \times 0.6\) = 766,000 - 248,000 = 518,000 ft. lbs.

\[
d = 36", \quad jd = 31.5" = 2.625', \quad kd = 13.5"
\]

\[
As = \frac{518,000}{(16,000 \times 2.625)} = 12.35 \text{ sq. in.}
\]

\[
kd = \frac{2ndAs}{bt} = \frac{(2 \times 15 \times 36 \times 12.35)}{(2 \times 15 \times 12.35)} \div \frac{72 \times 49}{72 \times 7} = 16,866 = 12.25"
\]

\[
z = \frac{(3kd - 2t)}{3} = \frac{(36.75 - 14)}{3} = 22.75 \div 3 = 3.03"
\]

\[
jd = d - z = 36 - 3.03 = 32.97" = 2.74'
\]

\[
fs = \frac{518,000}{12.35 \times 2.74} = 15,320 \text{ lbs. per sq. in.}
\]

\[
fc = \frac{518,000 \times 12.25}{(2.74 \times 72 \times 7)} = 500 \text{ lbs. per sq. in.}
\]

Use 10 - 1-1/8 in. sq. bars.
Assumed Section of 40' Span.

40' Deck - Interior Girder.
Position of Live Load for Max. Bending.
Shear in Interior Girder

Load at end -

\[ R-2 = \frac{(15,520 \times 24)}{22,040 \times 38} + (1,600 \times 19) = 56,000 \text{ lbs.} \]

\[ R-1 = 88,400 - 56,000 = 32,400 \text{ lbs.} \]

bjd x 40 = 21 x 32.97 x 40 = 27,700 lbs.

\[ V' = 56,000 - 27,700 = 28,300 \text{ lbs.} \]

Vertical Component of 1 - 1-1/8 in. sq. bar 6 45 degrees =

\[ 1.265 \times 15,000 \times 0.707 = 14,300 \text{ lbs.} \]

Bend up 2 at 2' - 0"

Bend up 2 at 4' - 6"

Bend up 1 at 7' - 0"

Bend up 1 at 9' - 6"

Use 5/8" φ stirrups at 18" throughout to support slab steel.

Rear wheel load at 7' - 10" from R. H. support.

\[ R-2 = \frac{(22,040 \times 30.2)}{5,520 \times 16.2} + (1,600 \times 19) = 50,300 \text{ lbs.} \]

See shear diagram.
40' Deck

Bending in Exterior Girders

D. L. per Lin. Ft. - Conc. - Slab - 65 x 7 = 455 lbs.
Curb - 12 x 18 = 216 "
Stem - 33 x 21 = 693 "
Steel - Slab - 3 x 7 = 21 "
Gird. (assumed) 53 "
Paving - 200 "

Rail - (assumed) 200 "

1,838 lbs. - Total

Live Load - 1 set wheel loads per Gird. - Impact factor = 0.36

3,000 x 1.36 = 4,080 lbs. (Fr. Wheel) - 12,000 x 1.36 = 16,320 lbs. (R. Wh.)

See position for bending.

R-2 = \( \frac{(4,080 \times 7.4)}{(16,320 \times 21.4)} \times \frac{1}{(1,840 \times 19)} = \frac{39}{39} \)

R-1 = \( \frac{(1,840 \times 38)}{16,320} \times \frac{4,080}{44,720} = 45,660 \) lbs.

B.M. = \( (44,720 \times 17.6) - (1,840 \times 17.6 \times 6.6) \) =
760,000 - 285,600 = 502,200 ft. lbs.

d = 35", \( jd = 30.625" = 2.55' \), \( As = \frac{502,200}{2.55 \times 18,000} = 12.35 \text{ sq. in.} \)

\( kd = \frac{2ndAs}{2bt} = \frac{(30 \times 35 \times 12.35)}{696} = 16,104 \) = 12.73"

\( x = \frac{(3kd - 2t)}{3} = \frac{(38.19 - 14)}{3} \times 7 = 24.19 \times 7 = 3.06" \)

\( zd = 35 - 3.06 = 31.94" = 2.66' \)

\( fs = \frac{502,200}{12.35 \times 2.66} = 15,300 \text{ lbs. per sq. in.} \)

\( fc = \frac{502,200}{(2.66 \times 64 \times 7)} = 555 \text{ lbs. per sq. in.} \)

Use 10 - 1-1/8" sq. bars.
40' Deck - Exterior Girder
Position of Live Load for Max. Bending.
40' Deck

Shear on Exterior Girders

\[ R-2 = \frac{(4.060 \times 24)}{38} + \frac{(1.840 \times 19)}{16.320} = 53,900 \text{ lbs.} \]

\[ \text{bd} \times (40) = 21 \times 31.94 \times 40 = 26,850 \text{ lbs.} \]

\[ V' = 53,900 - 26,850 = 27,050 \text{ lbs.} \]

Vertical Component of 1 - 1-1/8 in. sq. bar at 45 degrees =

\[ 1.266 \times 16,000 \times 0.707 = 14,300 \text{ lbs.} \]

Bend up 2 at 2' - 0" from support

Bend up 2 at 4' - 6" " "

Bend up 1 at 7' - 0" " "

Bend up 1 at 9' - 6" " "

Use 7/8 \( \phi \) stirrups at 18" throughout to support slab steel.
40' Deck
Shear on Exterior Girders.
Scales - $\frac{3}{32}$" = 1'
1" = 10,000 #
**Steel Base Plate**

1/2" Thick  

W = 15" for 52' Decks  
= 13" for 40' Decks  

Holes - 21/32"

**Bronze Top Plate**

1/2" Thick  

W = 12" for 52' Decks  
= 10" for 40' Decks  

Holes - 21/32"

**Bearing Plates for Girders**

Steel plate to be machined in direction of its length. Bronze plate to be machined in direction of its width. Bolts 5/8" x 12". Bolts for top plate to be threaded to countersunk heads and nuts tightened on top of plate.
Approximate quantities in Decks

52' Decks - Concrete -

Area Girders = \( \frac{21 \times (96 / 96)}{144} = 26.29 \text{ sq. ft.} \)

Area Slab = \( \frac{7 \times 23}{12} = 13.42 \text{ sq. ft.} \)

Area Curb = \( 1 \times 1.5 \times 2 = \frac{3.00 \text{ sq. ft.}}{44.71 \text{ sq. ft.}} \) Subtotal

Area Diaphragm = \( \frac{4.33 \times (97 + 49)}{12} = 56.72 \text{ sq. ft.} \)

Vol. = \( \frac{(44.71 \times 52)}{12} \times \frac{(58.72 \times 10 \times 2)}{12} = 2,330 \div 97.86 = 24.27 \text{ cu. ft.} = 69,992 \text{ cu. yds.} \)

52' Decks - Steel -

For Bent up bars add \( \frac{(2 \times 48)}{707} - 96 = 40'' = 3' - 6'' \)

For all hooks add 18''

Bent up bars - 51'' - 8'' \( / 3' - 6'' / 3' = 58'' \)

Straight bars - 51'' - 8'' \( / 3' = 55'' \)

Stirrups - 17'' \( / 102'' / 24'' \) for hooks = 143'' \( = 12' \) (35 to Girder)

Diaphragm bars - 6 - 5/8'' \( \phi \) at 22''

Long. slab bars - 21 - 1/2'' \( \phi \) at 54' \( = 1100 \times 5.31 = 5,628 \text{ lbs.} \)

Transverse slab bars - 144 - 5/8'' \( \phi \) at 25'

20 - 1\( \frac{1}{2} \) in. sq. bars \( \phi \) 58' = 1160 \times 5.31 = 6,165 lbs.

20 - 1\( \frac{1}{2} \) in. sq. bars \( \phi \) 55' = 1100 \times 5.31 = 5,800 lbs.

140 - 5/8in. \( \phi \) \( \phi \) 12' = 1680 \times 1.04 = 1,732 lbs.

144 - 5/8in. \( \phi \) \( \phi \) 25' = 3600 \times 1.04 = 3,744 lbs.

6 - 5/8in. \( \phi \) \( \phi \) 22' = 132 \times 1.04 = 1,367 lbs.

21 - \( \frac{1}{2} \) in. \( \phi \) \( \phi \) 54' = 1134 \times 0.67 = 750 lbs.

Total weight of steel = 18,378 lbs.

Total Rail = 104 \times 200 = 20,800 lbs.

Total Paving = 52 \times 50 \times 20 = 52,000 lbs.
Total Dead Load

Concrete - 90 x 3,900 = 351,000 lbs.
Steel - 18,378 lbs.
Rail - 20,800 lbs.
Paving - 52,000 lbs.

442,178 lbs. - Total

Live Load Reaction = 4 x L.L. Reaction on Ext. Girder.

\[
4 \times \left( \frac{(12,240 \times 6) + (4,080 \times 36)}{50} \right) = 16,320
\]

\[
4 \times \left( \frac{73,440 + 147,000}{50} \right) = 4,410 + 16,320 = 20,730 lbs.
\]

Total Reaction = 20,730 / 442,178 = 304,000 lbs.

40' Deck - Concrete

Area Girders - 1.75 x \( \frac{124}{12} \) = 19.54 sq. ft.
Area Slab - 23 x 7/12 = 13.42 sq. ft.
Area Curb - 1 x 1.5 x 2 = \( \frac{3.00}{35.96} \) sq. ft.

Area Diaphragm = \( \frac{(4.35 \times 67)}{12} \) = 40.78 sq. ft.

Vol. = \( \frac{(35.96 \times 40) + (40.78 \times 10)}{12} \) = 1438.4 / 57.97 = 1506.37 c.ft. = 55.79 c.yds.

40' Deck - Steel

For Bent up bars add \( \frac{(2 \times 33)}{707} \) = 27" - 2' - 3"
For all hooks add 18"
Bent up bars - 39' - 8" / 2' - 3" / 3' = 48'
Straight Gird. bars - 39' - 8" / 3' = 42' - 8"
Stirrups - 17" / 74" / 24" (Hooks) = 9" - 8"
Diaphragm bars - 6 - 5/8" at 22'
Long. slab bars - 21 - 7/8" at 40'
Transverse slab bars - 111 - 5/8" at 25'
Approximate Quantities and Reactions

Summary of Steel

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Description</th>
<th>Length (ft)</th>
<th>Diameter (in.)</th>
<th>Weight (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>1-1/8 in. sq. bars @ 45'</td>
<td>1,080 x 4.30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>5/8 in. @ 42.67'</td>
<td>680 x 4.30</td>
<td>7,550 lbs.</td>
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<td>108</td>
<td>5/8 in. @ 9.7'</td>
<td>1,050 x 1.04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>5/8 in. @ 22'</td>
<td>132 x 1.04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>111</td>
<td>5/8 in. @ 25'</td>
<td>2,775 x 1.04</td>
<td>4,115 lbs.</td>
<td></td>
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<tr>
<td>21</td>
<td>5/8 in. @ 40'</td>
<td>840 x .67</td>
<td>563 lbs.</td>
<td></td>
</tr>
<tr>
<td>total</td>
<td></td>
<td>12,228 lbs.</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Wt. of Concrete = 55.79 x 4,000 = 223,160 lbs.
Total Rail = 80 x 200 = 16,000 lbs.
Total Paving = 50 x 40 x 20 = 40,000 lbs.

Total Dead Load

- Concrete - 223,160 lbs.
- Steel - 12,228 lbs.
- Rail - 16,000 lbs.
- Paving - 40,000 lbs.

291,388 lbs. - Total

Live Load Reaction = 4 \times \left( \frac{24 \times 4.080}{32} \right) = 75,640 lbs.

Total Reaction = \frac{75,640 + 291,388}{2} = 75,640 + 145,694 = 221,334 lbs.
Substructure

2 Column Bents for 40' Decks

Wt. of Cap. per lin. ft.

Concrete - \( (27 \times 48) \div (2 \times 24) = 1,344 \) lbs.

Steel - (assumed) \( 56 \) lbs.

Total - \( 1,400 \) lbs.

Tie Beam

16" x 15" x 12" - \( d = 16", \; j d = 14", \; k d = 6" \)

\[ M = 6" \times 15" \times 1.16 \times 325 = 33,900 \text{ ft. lbs.} \]

\[ B.M. = 2 \times 1.5 \times 1.25 \times 150 \times 12^2 = 10,150 \text{ ft. lbs.} \]

As - \( \frac{10,150}{1.16 \times 15,000} = 0.55 \text{ sq. in.} \)

2 - 5/8" # top and bottom.

Bending on Cap.

\[ B-1 = B-2 = (1,400 \times 6) \div 112,000 = 120,400 \text{ lbs.} \]

\[ B.M. = (120,400 \times 6) - \left[ (112,000 \times 3.06) \div (1,400 \times 6 \times 5) \right] = 722,400 - 340,420 - 25,200 = 356,720 \text{ ft. lbs.} \]

\( d = 42", \; j d = 36.75" = 3.06", \; k d = 15.75" \)

As - \( \frac{356,720}{3.06 \times 15,000} = 7.29 \text{ sq. in.} \) - Use 6 1-1/8 in. sq. bars.

\[ f c = \frac{356,720 \times 2}{3.06 \times 27 \times 15.75} = 548 \text{ lbs. per sq. in.} \]

Bending on Cantilever

\[ B.M. = (107,800 \times 2.13) \div (1,400 \times 2.5 \times 2.25) = 230,000 \div 7,880 = 237,880 \text{ ft. lbs.} \]

\( d = 42", \; j d = 36.75" = 3.06", \; k d = 15.75" \)

As - \( \frac{237,880}{3.06 \times 15,000} = 4.86 \text{ sq. in.} \)

Use 4 - 1-1/8 in. sq. bars, 2 bent up.
Intermediate Bents
for 40' Decks.

End View
of Cap.
Bending in Intermediate Bent Caps.
Shear in Intermediate Bent Cap.

\[ V = R = 120,400 \text{ lbs.} \]

\[ V - V' = 40 \times 36.75 \times 27 = 39,690 \text{ lbs.} \]

\[ V' = 120,400 - 39,690 = 80,710 \text{ lbs.} \]

\( 7/8" \) stirrups -

\[ 2 \times 0.601 \times 16,000 = 19,200 \]

\[ \frac{80,710}{19,200} = 4.21 \]

\[ \frac{36.75}{4.21} = 8.73 \text{ spacing} \]

Space \( 8 1/2" \) C. to C. for 4' - 6"

(7 stirrups at each end)

Cantilever

\[ V = 107,800 \div (1,400 \times 2.5) = 107,800 \div 3,500 = 111,300 \text{ lbs.} \]

\[ V' = 111,300 - 39,690 = 71,610 \text{ lbs.} \]

\[ \frac{71,610}{19,200} = 3.73 \]

\[ \frac{36.75}{3.73} = 9.84 \]

3 stirrups - \( 7/8" \) at 9.75"
Shear in Intermediate Bent Caps.
Scales - 3/8" = 1'.
1" = 20000#
Columns - Intermediate Bents -

Max. Col. Length = 39'

Load at Top = 112,000 + 107,600 + 14,700 = 234,500 lbs.

Max. Unsupported Length = 24'

\[ 600 - \left( \frac{15 \times 24}{2} \right) = 420 \text{ lbs. per sq. in.} \]

With 1 percent. steel - (8 - 1 in. sq. bars)

\[ 420 \times 1.14 = 478 \text{ lbs. per sq. in.} \]

\[ \frac{234,500}{24 \times 27} = 362 \text{ lbs. per sq. in.} \]

Note: Above computations for Class "A"

for Class "B" - \[ \frac{17}{22} \times 478 = 369 \text{ lbs. per sq. in.} \]

Wt. of Col. = \[ 39 \times \left( \frac{[27 \times 24] + [47 \times 44]}{2} \right) + 32 \times 39 = 54,100 \text{ lbs.} \]

Load at Foot. = 234,500 - 54,100 = 288,600 lbs.

Area Base = 47" \times 44" = 2,068 sq. in. \( A_s = 8 \text{ sq. in.} \)

\[ 288,600 \div 1,142 = 253 \text{ lbs. per sq. in.} \]

\[ \left( 600 - \left( \frac{15 \times 24}{2} \right) \right) \times \frac{17}{22} = 364 \text{ lbs. per sq. in.} \]

Steel - 8 - 1 in. sq. bars - Hoops - 3/8" \( \phi \) at 12"

Footing

Base - 7'-11" x 7'-8" x 1'-6", Cap - 5'-11" x 5'-6" x 1'-6"

Punching at Col. = \[ \frac{288,600}{182 \times 36} = 34 \text{ lbs. per sq. in.} \]

Punching at Cap = \[ \frac{288,600 + (71 \times 68 \times 1.5) \times 3,920}{276 \times 18} = 27 \text{ lbs. per sq. in.} \]

Bearing = \[ \frac{288,600 + (71 \times 68 \times 1.5) + (95 \times 92 \times 1.5)}{722 \times 7.66} = \frac{308,950}{7.92 \times 7.66} = 5,100 \text{ lbs. per sq. ft.} = 2.55 \text{ tons per sq. ft.} \]
Wind Pre asupe - mediate, Ben s

8,000 lbs.

3,450 lbs.

2,700 lbs.

7,430 lbs.

405 lbs.

1,860 lbs.

W. P. on L.L. at 55.75° - 40 x 200

W. P. on Rail - 40 x 1.92 x 1.5 x 30

W. P. on Curb - 40 x 1.5 x 1.5 x 30

W. P. on Girds. - 40 x 2.75 x 2.25 x 30

W. P. on Caps - 4 x 2.25 x 1.5 x 30

W. P. on Cols. - 16 x 2.56 x 1.5 x 30

Resultant of wind Pre

8,000 x 55.75 = 446,000

3,450 x 51.46 = 177,500

2,700 x 49.50 = 134,000

7,430 x 47.38 = 352,000

405 x 44.00 = 17,850

1,860 x 34.00 = 62,100

23,845

1,190,450

1,190,450 = 49.99'

23,845
Wind Pressure on Intermediate Bents.
Intermediate Bents - Resultant of Axial Load & Wind Pressure.
Bending in Cutwater Bent Caps

Wt. of Cap = (30 x 66) / (2 x 24) / (Steel assumed - 75 lbs.) = 2,040 / 48 / 75 = 2,163 lbs. per Lin. Ft.

d = 62", jd = 54.25" = 4.52', kd = 23.25"

R = (2,163 x 8.63) / 152,100 = 170,800 lbs.

B.M. = (170,800 x 9.13) - (2,163 x 8.63 x 4.31) - (152,100 x 3.04) = 1,551,000 - 80,600 - 462,000 = 1,008,400 ft. lbs.

As = 1,008,400 / 4.52 x 16,000 = 14 sq. in. (12 - 1-1/8 in. sq. bars.)

fc = 1,008,400 x 2 / (4.52 x 30 x 23.25) = 643 lbs. per sq. in.

Note: Cols. of Class "B" - Cap of Class "A"

Shear in Cutwater Bent Caps

R-2 = (2,163 x 8.63) / 152,100 = 18,700 / 152,100 = 170,800 = V

V = 170,800 / (54.25 x 30) = 105 lbs. per sq. in.

V' = (40 x 54.25 x 30) = 65,000 lbs.

V = 170,800 - 65,000 = 105,800 lbs.

7/8" @ Stirrups = 2 x .601 x 16,000 = 19,200

19,200 = 5.5

54.25 = 9.86 = Spacing - Use 9.5"

Use 10 Stirrups tp each side, beginning 1' off center, ending 8' -1/2" off center.
Cutwater Bents for 52' Decks.

Batter each side ½" in 12".

End View of Cap

Footing
Shear in Cutwater
Bent Caps.
Scales - $\frac{1}{4}'' = 1'$. 
$1'' = 25000#$. 

Cap - 2163#/ft
26,650#
158700#
170800#
Compression on Cols.

Cutwater Bents -

Area at Top = (31 x 31) - (7 x 14) = 863 sq. in.

Load at Top = (11.38 x 2,163) / 152,100 / 149,500 = 326,250 lbs.

\[
\frac{326,250}{863} = 379 \text{ lbs. per sq. in.}
\]

(Assume 6 -1\(\frac{1}{2}\) in. sq. bars = 9.37 = 1.089 percent.)

Allowable Comp. = \[
\frac{\left(450 \times 0.0108 \times 14\right)}{450} \times \frac{17}{22} = 400 \text{ lbs. per sq. in.}
\]

Area of Base = 55 x 55 = 3,025 sq. in.

Effective Area of Base = 9.37 x 1,000 = 1,339 sq. in.

Wt. of Col. = \[
\frac{\left(31 \times 31\right) - (55 \times 55)}{2} \times 24\)
\]

(Steel - 50 x 34) = 48,000 / 1,700 = 49,700 lbs.

Load at Base = 49,700 / 326,250 = 376,000 lbs.

Allowable Comp. = \[
\frac{\left(450 \times 0.007 \times 14\right)}{450} \times \frac{17}{22} = 380 \text{ lbs. per sq. in.}
\]

\[
\frac{376,000}{1,339} = 281 \text{ lbs. per sq. in.}
\]

Steel - 6 -1\(\frac{1}{2}\) in. sq. bars and 3/8" @ Hoops @ 12" C. to C.

Footings

Depths - 8' to 12'

Total Load on Found. = 376,000 / (96 x 96 x 9) / (120 x 120 x 3) = 376,000 / 63,000 / 43,200 = 502,200

Area Base = 100 sq. ft., Area Cap = 64 sq. ft.

Area Col. = 21 sq. ft.

Punching at Col. = \[
\frac{\left(456,100 \times 0.79\right)}{220 \times 8 \times 12} = 17 \text{ lbs. per sq. in.}
\]

Punching at Cap = \[
\frac{\left(456,100 \times 0.36\right)}{384 \times 36} = 12 \text{ lbs. per sq. in.}
\]
Bearing = \frac{455,000}{100} = 4,500 \text{ lbs. per sq. ft.} = 2.3 \text{ tons per sq. ft.}

or \frac{455,100}{64} = 7,120 \text{ lbs. per sq. ft.} = 3.56 \text{ tons per sq. ft.}

Note: Depth of base may be increased as desired. Base to be Class "A" if poured under water. Entire footing may be Class "B" 8 x 8 if foundation is good and dry.
River Bents - Wind and Current Pressures

Wind Pres. on Rail - \(1.92 \times 52 \times 30 = 3,000\) lbs.

" " on Curb - \(1.5 \times 52 \times 30 = 2,340\) lbs.

" " on Girds. - \(4 \times 52 \times 30 = 6,240\) lbs.

" " on Cap - \(5.66 \times 2.5 \times 30 = 425\) lbs.

" " on Cols. - \(3.13 \times 2 \times 30 = 193\) lbs.

" " on Cols. - \(4.08 \times 3 \times 30 = 368\) lbs.

Cur. Pres. on Cols. - \(5.36 \times 19 \times 40 = 4,080\) lbs.

Cur. Pres. on Foot. - \(11.33 \times 4 \times 40 = 1,820\) lbs.

Resultant of Wind and Current Pressures:

\[\begin{align*}
1,820 \times 10 &= 18,200 \\
4,080 \times 20.75 &= 84,900 \\
366 \times 32.50 &= 12,000 \\
193 \times 35.00 &= 6,760 \\
425 \times 36.83 &= 16,500 \\
6,240 \times 43.66 &= 273,000 \\
2,340 \times 46.41 &= 106,800 \\
3,000 \times 48.37 &= 145,200 \\
18,466 &= 18,466 \\
665,360 &= 36.00' \\
18,466 &= 18,466 \\
\end{align*}\]

Vertical Dead Load

Decks - 442,200

Cap - \(22.75 \times 2,163 = 49,200\)

Cols. - \(49,700 \times 2 = 99,400\)

Foots. - \(96 \times 96 \times 24 = 221,000\)

Total - 811,800

\(\frac{1}{2}\) L.L. = 82,920 at 10' from Center Line.
Rai/CA+

42c# .95 #

Cur men f

Oar

tu rre

71

30"

47"

51"

11'-6"

11'-4"

3000#

2340#

6240#

425#

193#

368#

4080#

1820#

1075',

115'

10'

81/2'

1/2'

Wind & Current Pressures - River Bents.
Cutwater Bents

Resultant of Wind and Current Pressures with axial load.

Resultant of D. L. and \( \frac{1}{3} \) L. L.

Moments about a

\[
\frac{(6,116,000 + 1,658,400)}{811,800 + 82,920} = 9,776,400 = 10.97'
\]

Resultant of vertical and horizontal forces. See sketch.

\( y = \) eccentricity due to side pressures.

\( e = \) Total eccentricity.

\( y = \frac{18,470 \times 36}{894,720} = 0.75 \)

\( e = 0.75 / 0.97 = 1.72' \)
River Bents.
Resultant of Wind
$\&$ Current Pressures
with Axial Load.
Superload -

2 - 15 ton trucks over 20' x 14' area. \( (30 \times 2,000) - 215 \text{ lbs. per sq. ft.} \)

\( \frac{215}{100} = 2.15' \)

Parapet Wall

\( \phi = 30 \text{ degrees}, \quad C_e = 0.333, \quad P = C_e \times wh^2 - \frac{1}{3} \times 30 \times \frac{1}{2} \times h^2 = 5h^2 \)

\( (5 \times 7.15^2) - (5 \times 2.15^2) = 256 - 23 = 233 \text{ lbs.} \)

B. M. = \( 2.1 \times 2.13 = 490 \text{ ft. lbs.} \)

\( d = 9'', \quad jd = 7.875'' = 0.656', \quad kd = 3.375'' \)

\( f_c = \frac{490}{(0.656 \times 3.375 \times 12)} \times 2 = 37 \text{ lbs. per sq. in.} \)

\( A_s = \frac{490}{(0.656 \times 16,000)} = 0.047 \text{ sq. in.} \)

Use \( \frac{1}{2}'' \phi \) at 12'' C. to C.

Bending in Cap

Wt. of Cap:

- Conc. - \( (24 \times 36) / (12 \times 2) = 866 \text{ lbs.} \)
- Steel - (assumed) \( 50 \text{ lbs.} \)
- Total - \( 936 \text{ lbs.} \)

\( R_2 = \frac{(53,900 \times 9.13) / (56,000 \times 3.04) / (936 \times 5)}{10} \)

\( (492,000 / 170,400) / 4,690 = 70,930 \text{ lbs.} \)

\( R_1 = (56,000 / 53,900 / 9,380) - 70,930 = 48,350 \text{ lbs.} \)

\( \phi = 3.8 \text{ ft.} = \left[ (48,350 \times 3.04) - (3.04 \times 936 \times 1.52) \right] \times 0.8 = \)

\( (147,400 - 4,340) \times 0.8 = 114,500 \text{ ft. lbs.} \)

\( T = 36'', \quad d = 33'', \quad jd = 28.875'' = 2.4', \quad kd = 12.375'' \)

\( f_c = \frac{114,500 \times 2}{(2.40 \times 24 \times 12.37)} = 322 \text{ lbs. per sq. in.} \)
As = \frac{114,500}{(2.4 \times 16,000)} = 2.99 \text{ sq. in.}

Use 4 - 1\" \varphi \text{ top and bottom. Carry 2 in top to end and hook.}

End - 2 - 4\' past outer face of col.

Shear:

\[ v = \frac{70,900}{(28.8 \times 24)} = 103 \text{ lbs. per sq. in.} \]

\[ V' = 70,900 - (28.8 \times 24 \times 40) = 43,300 \]

5/8\" \varphi \text{ stirrups} = 2 \times 0.307 \times 16,000 = 9,800 \text{ lbs.}

\[ \frac{43,300}{98,000} = 4.42 \quad \frac{26.87}{4.42} = 6.03\text{"} \]

Space 6\(\frac{1}{2}\)\" C. to C. from Interior Col. faces.

Use 12 stirrups to each span.

Shear on Cantilever:

\[ \text{Wt. of cap} = \frac{[(24 \times 16) + (24 \times 36)] \times 7.75}{2} = 5,020 \text{ lbs.} \]

\[ \text{Wt. of parapet} = (5 \times 12 \times 24) - (5 \times 12 \times 32) = 3,360 \text{ lbs.} \]

Steel - (assumed) \[ \frac{100 \text{ lbs.}}{8,480 \text{ lbs.}} \]

Total Wt. of Cantilever - 8,480 lbs.

Pressure of fill on Cantilever:

\[ (5 \times 10.15^2) - (5 \times 2.15^2) = 492 \text{ lbs. per lin. ft.} \]

Total Ph = 492 \times 7 = 3,444 \text{ lbs.}

\[ V = \frac{8,460}{\sin \text{ arc tan} \frac{8,460}{3,444}} = \frac{8,460}{0.9272} = 9,110 \text{ lbs.} \]

\[ v = \frac{9,110}{(28.8 \times 24)} = 14 \text{ lbs. per sq. in.} \]
Shear in End Bent Caps.
Scales - ¼" = 1'.

1" = 10,000 lbs.
End Bents - Cols.

Wt. of Cap = \((936 \times 23.5) \div 8,480 = 37,700\) lbs.

St. of Deck = \((53,900 \div 55,000) \times 2 = \frac{212,600}{2} = 257,500\) lbs. - Total

Load per Col. = \(\frac{257,500}{3}\) lbs. = 85,830 lbs.

The of Col. using 4 - 1\(\frac{1}{2}\) in. sq. bars and 3/8" Hoops
Conc. - \([(24 \times 24) \div (99 \times 24)] \times \frac{30}{2} = 44,280\) lbs.

Steel - \((5.31 \times 4 \times 34) \div (30 \times 14 \times .38) = 884\) lbs.

Total - 
45,164 lbs.

Load from Cap - 65,830 lbs.

Total Load to Footing - 130,994 lbs.

Percent. of steel at Top = \(\frac{6.25}{576} = 1.08\) percent.

At Base - Conc. for 0.7 percent. = \(\frac{6.25 \times 1,000}{7} = 893\) sq. in.

600 - \(\frac{151}{2}\) = 375 lbs. per sq. in.

Allowable unit stresses including steel

Base - \((375 \times .007 \times 14) \div 375 = 36 \div 375 = 411\) lbs. per sq. in.

Top - \((375 \times .0108 \times 14) \div 375 = 56 \div 375 = 431\) lbs. per sq. in.

Comp. from Axial Load

Base - 130,994 = 147 lbs. per sq. in. Top - 65,830 = 150 lbs. per sq. in.

893

Note: See additional stress for bending.

Bearing: Cap and Decks - 83,500 lbs. per Col.

Col. - 45,164 " " "

Foot. - \([42 \times 111] \div (66 \times 135)] \times 1.5 = \frac{20,400}{893} = 2410\) lbs. per sq. ft.

Total - 149,064 lbs. per Col.

Bearing per sq. ft. = \(\frac{149,064}{5.5 \times 11.25} = 1.205\) tons per sq. ft.
2nd Story - Pressure of Fill

\[ P-1 = (10.2^2 \times 5) - (2.2^2 \times 5) = 547 \text{ lbs.} \]
\[ P-2 = (40.2^2 \times 5) - (10.2^2 \times 5) = 7,509 \text{ lbs.} \]
\[ P-3 = (41.7^2 \times 5) - (40.2^2 \times 5) = 620 \text{ lbs.} \]
\[ P-4 = (43.2^2 \times 5) - (41.7^2 \times 5) = 650 \text{ lbs.} \]

Length of Cap = 37.5'

Pressures per ft. width of Col.

\[ P-1 = \frac{547 \times 37.5}{6} = 3,420 \text{ lbs.} \]
\[ P-2 = 2,510 \text{ lbs.} \]
\[ P-3 = \frac{620 \times 3.5}{2} = 1,090 \text{ lbs.} \]
\[ P-4 = \frac{650 \times 5.5}{2} = 1,790 \text{ lbs.} \]

Resultant of overturning Forces

\[ 3,420 \times 36.2 = 124,000 \text{ ft. lbs.} \]
\[ 7,509 \times 15.1 = 113,600 \text{ ft. lbs.} \]
\[ 1,090 \times 2.3 = 2,510 \text{ ft. lbs.} \]
\[ \frac{1,790 \times 0.6}{13,690 \text{ lbs.}} = 1,430 \text{ ft. lbs.} \]
\[ 241,540 \text{ ft. lbs.} \]
\[ 13,809 \]
\[ 241,540 = 17.5 \text{ ft.} \]

Resultant of Vertical Loads

\[ 41,500 \times 8.6 = 365,500 \]
\[ 5,700 \times 10.5 = 59,850 \]
\[ 22,580 \times 6.9 = 156,000 \]
\[ 10,200 \times 5.6 = 57,100 \]
\[ 79,980 \text{ lbs.} \]
\[ 636,450 \text{ ft. lbs.} \]
\[ 636,450 = 7.98 \text{ ft.} \]

Resultant of Fill Pressure and Vertical Loads

\[ y = \text{ eccentricity from vertical Load.} \]
\[ e = \text{ eccentricity from center of footing.} \]
\[ y = \frac{13,809 \times 17.5}{79,980} = 3.06, \quad e = 3.06 - 2.38 = 0.68 \text{ ft.} \]
Overturning Forces on End Bent.
Resistance to Overturning in End Bent.
Compression in Col. due to Fill

Resultant of weight of Bent and Decks

Moments about Footing toe

\[ 41,500 \times 6.8 = 365,500 \]
\[ 22,580 \times 6.9 = 155,800 \]
\[ 64,080 \text{ lbs.} \quad 521,300 \text{ ft. lbs.} \quad \frac{521,300}{64,080} = 8.13 \text{ ft.} \]

Resultant of Overturning Forces

Moments about Footing Base

\[ 3,420 \times 36.2 = 124,000 \]
\[ 7,509 \times 15.1 = 113,300 \]
\[ 10,929 \text{ lbs.} \quad 237,300 \text{ ft. lbs.} \quad \frac{237,300}{10,929} = 21.7 \text{ ft.} \]
\[ y = 10,929 = 16.7 \quad 64,080 \quad y = \frac{10,929 \times 16.7}{64,080} = 3.19 \text{ ft.} \]
\[ e = y - 2.5 = 3.19 - 2.5 = 0.69 \text{ ft.} \]
\[ f = \frac{m \times n}{I} \]

Where \( m = \text{wt. of wall} \times e = 0.69 \times 64,080 = 44,150 \)
\[ n = \frac{1}{2} \text{ Base} = 4.13 \]
\[ I \text{ is about axis through center of base} = \]
\[ \frac{pd^3}{12} = \frac{6.26^3}{12} = 46.6 \]
\[ f = \text{change in pressure due to eccentricity} \]
\[ f = \frac{44,150 \times 4.13}{46.6} = 3,910 \text{ lbs.} \]
\[ \text{Av. Comp.} = \frac{64,080}{8.25} = 7,760 \text{ lbs. per sq. ft.} \]
\[ \text{Comp. at Toe} = 7,760 + 3,910 = 11,670 \text{ lbs. per sq. ft.} = 81 \text{ lbs. per sq. in. Total fc} = 81 + 147 = 228 \text{ lbs. per sq. in.} \]
Compression in Cols. Due to Bending. End Bent.

R = 10,923#
Hand Rail

Wt. of Rail - Concrete - 7 x 9 = 63 lbs.

Steel (Assumed) = 3 lbs.

Total = 66 lbs.

Horiz. ld. of 150 lbs. per lin. ft.

Vert. ld. of 100 / 66 = 166 lbs. per lin. ft.

\[ v = \frac{226 \times 5}{9} = 1,140 \text{ lbs.} \]

Horiz. \( d = 5\frac{3}{4} \text{ in.}, \) \( jd = 5.04 \text{ in.} = .42 \text{ ft.}, \) \( kd = 2.15 \text{ in.} \)

\[ v = \frac{1,140}{(5.04 \times 9)} = 26 \text{ lbs. per sq. in.} \]

Vert. \( d = 7\frac{3}{4} \text{ in.}, \) \( jd = 6.78 \text{ in.} = .565 \text{ ft.}, \) \( kd = 2.90 \text{ in.} \)

\[ v = \frac{1,140}{(6.78 \times 6)} = 26 \text{ lbs. per sq. in.} \]

Hanging on Rail -

Vert. 3:\# = \( \frac{166 \times 10 \times 10}{8} = 2.075 \text{ ft. lbs.} \)

\[ fc = \frac{2.075 \times 2}{(5.65 \times 2.90 \times 7)} = 362 \text{ lbs. per sq. in.} \]

As = \( \frac{2.075}{(5.65 \times 16,000)} = 0.23 \text{ sq. in. (3 - 3/8'' $\phi$)} \)

Horiz. 3:\# = \( \frac{150 \times 10 \times 10}{8} = 1,875 \text{ ft. lbs.} \)

\[ fc = \frac{1,875 \times 2}{(42 \times 2.15 \times 9)} = 460 \text{ lbs. per sq. in.} \]

As = \( \frac{1,875}{(42 \times 16,000)} = 0.28 \text{ sq. in. (3 - 3/8'' $\phi$)} \)

\[ u = \frac{1.140}{(5.04 \times 3 \times 1.178)} = 64 \text{ lbs. per sq. in.} \]

(Hook all Bars.)
**Rail Posts**

Horizontal force from rail at top of post = 150 x 10 = 1,500 lbs.

\[ d = 10'', \quad jd = 6.75'' = 0.73', \quad kd = 3.75'' \]

\[ B.M. = 1,500 \times 3 = 4,500 \text{ ft. lbs.} \]

\[ 4,500 = 6,160 \text{ lbs.} \]

\[ fc \text{ at } 1'' \text{ from extreme fiber must equal } \frac{6.160}{22} = 280 \text{ lbs. per sq. in.} \]

\[ fc = \frac{280 \times 3.75}{2.75} = 384 \text{ lbs. per sq. in.} \]

\[ As = \frac{4,500}{(0.73 \times 16,000)} = 0.39 \text{ sq. in.} \]

Use 2 - 1/2" @ each side.

Axial load on post = \( (10 \times 166) \div (11 \times 11 \times 3) \div (10 \times 66) = 2,683 \text{ lbs.} \)

\[ \frac{2,683}{(11 \times 11) - (7 \times 8)} = 42 \text{ lbs. per sq. in.} \]

Total \( fc = 384 \div 42 = 426 \text{ lbs. per sq. in.} \)

\[ v = \frac{1,500}{(11 \times 8.75) - (7 \times 8)} = 37.5 \text{ lbs. per sq. in.} \]

\[ u = \frac{1,500}{(8.75 \times 2 \times 1.57)} = 55 \text{ lbs.} \]

Use one thickness of tar paper around rail at expansion end.

Use 1/4" expansion material at one end of each rail.

Crown top rail 1/2".

Send 1/2" @ bars over top of top rail and lap.

Hook all rail bars.
Hand Rail.

Resultant of Vertical & Horizontal Forces on Rail.
Scale - 1" = 100#

Section of Post at Rail.
### Estimate of Design No. 1

#### Concrete

<table>
<thead>
<tr>
<th>Member and Number</th>
<th>C.Y. Cl.A.</th>
<th>C.Y. Cl.B.</th>
</tr>
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<tbody>
<tr>
<td><strong>Superstructure</strong></td>
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<td>2 - 52' Decks</td>
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<td>4 - 40' Decks</td>
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<td>68.34</td>
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<td>6 - End Bent Cols</td>
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<td>6 - End Bent Footings</td>
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<td></td>
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<td>2 - Intermediate Bent Caps</td>
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<tr>
<td>4 - Intermediate Bent Cols</td>
<td></td>
<td>56.20</td>
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<td>4 - Intermediate Footings</td>
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<tr>
<td>3 - Cutwater Bent Caps</td>
<td>35.78</td>
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<tr>
<td>6 - Cutwater Bent Cols</td>
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<td><strong>Totals</strong></td>
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### Estimate - Design No. 1

### Steel

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<th>Lbs. Total</th>
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<tr>
<td><strong>Superstructure</strong></td>
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<tr>
<td>2 - 52' Decks</td>
<td>36,756 lbs.</td>
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<tr>
<td>4 - 40' Decks</td>
<td>48,912 lbs.</td>
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<tr>
<td><strong>Substructure</strong></td>
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<tr>
<td>2 - 2nd Bent Caps</td>
<td>1,882 lbs.</td>
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<td>6 - 2nd Bent Cols.</td>
<td>5,256 lbs.</td>
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<td>2,700 lbs.</td>
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<td>5,520 lbs.</td>
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<td>3 - Cutwater Bent Caps</td>
<td>5,550 lbs.</td>
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<tr>
<td>6 - Cutwater Bent Cols.</td>
<td>6,174 lbs.</td>
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<td><strong>Sub-total</strong></td>
<td>112,750 lbs.</td>
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<td><strong>Bearing Plates and Bolts</strong></td>
<td>2,000 lbs.</td>
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<td><strong>Total</strong></td>
<td>114,750 lbs.</td>
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## Excavation

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</tbody>
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12,693.28 Cu. Ft. = 470.12 Cu. Yds.
100' Arch

Slabs -

Span - 11.6'

Clear span - 10.1'

Thickness (assumed) - 12"

Dead Load -

Paving - 50 lbs. per sq. ft.
Slab - 12 x 12 = 144 lbs. per sq. ft.
Steel (assumed) 6 lbs. per sq. ft.

200 lbs. per sq. ft.

Live Load -

One rear wheel of 15 ton truck at center = 
.4 x 15 = 6 tons = 12,000 lbs.

Impact factor = \( \frac{50}{(1 + 125)} = 0.397 \)

Lateral Distribution -

\[ E = (0.7 \times 10.1) \div 1.25 = 6.32 \quad \frac{7 + 6}{2} = 6.5 \]

Concentrated Load per ft. width = \( \frac{1.397 \times 12,000}{6.5} = 2,560 \text{ lbs.} \)

3.M. = \( \frac{(200 \times 10.6^2)}{6} \) \( \div (5.4 \times 2580) \) = \( 9,860 \text{ ft. lbs.} \)

For 12" slab, \( d = 10" \), \( jd = 8.75" = 0.729', \quad kd = 3.75" \)

\[ fc = \frac{9,860 \times 2}{(0.729 \times 12 \times 3.75)} = 6.02 \text{ lbs. per sq. in.} \]

\[ As = \frac{9,860}{(0.729 \times 16,000)} = 0.047 \text{ sq. in.} \]

Use 5/8" @ at 4 1/2" C. to C.
Bending on Curb

Curb = 18 x 18 = 324 lbs.

Rail = 200 lbs.

Steel = 6 lbs.

Total Dead Load = 530 lbs.

B.M. = \( \frac{(530 \times 10.6^2)}{8} \) / (2,580 x 5.4 x .5) = 14,690 ft. lbs.

d = 16", j = 14" = 1.17", k = 6"

\[ \text{fc} = \frac{14,690 \times 2}{(1.17 \times 18 \times 6)} = 232 \text{ lbs. per sq. in.} \]

\[ \text{As} = \frac{14,690}{(1.17 \times 16,000)} = 0.782 \text{ sq. in.} \]

Use 3 - 3/4" @ at 6" C. to C. under curb.

Use Transverse 1/2" @ at 12"

Load to Pilasters -

Slab between curbs -

<table>
<thead>
<tr>
<th>Material</th>
<th>Dimensions</th>
<th>Load (lbs)</th>
</tr>
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<tbody>
<tr>
<td>Slab</td>
<td>20 x 144 x 11.6</td>
<td>33,400</td>
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<tr>
<td>Paving</td>
<td>20 x 11.6 x 50</td>
<td>11,500</td>
</tr>
<tr>
<td>Steel</td>
<td>(54 x 11.6 x 1.04) / (11 x 23 x 0.67)</td>
<td>635</td>
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</table>

Total = 45,835 lbs.

U. L. to Beams = \( \frac{45,835}{20} \) = 2,292 lbs. per Lin. ft.

Curbs and Rail -

<table>
<thead>
<tr>
<th>Material</th>
<th>Dimensions</th>
<th>Load (lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curbs</td>
<td>(18 x 36 x 11.6) / (6 x 11.6 x 1.5)</td>
<td>7,624</td>
</tr>
<tr>
<td>Rail</td>
<td>2 x 11.6 x 200</td>
<td>4,640</td>
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</tbody>
</table>

Total = 12,264 lbs.

Reaction to Beam each side = 6,132 lbs.
Bending in Beam -

D. L. of Beam = (20 x 30) / (steel assumed 35 lbs.) = 635 lbs. per Lin. ft.

Total U. L. = 635 / 2,927 = 2,927 lbs. per lin. ft.

B.W. = \( \frac{1}{2} (2,927 \times 12.5^2) \) / (16,770 x 4.75) = 136,800 ft. lbs.

d = 26", \( j_d = 24.5" = 2.042' \), \( k_d = 10.5" \)

\[ fc = \frac{136,800 \times 2}{(2.042 \times 20 \times 10.5)} = 637 \text{ lbs. per sq. in.} \]

\[ As = \frac{136,800}{2.042 \times 16,000} = 4.16 \text{ sq. in.} \]

Use 6 - 1" @: Bend up 2 at 2'-6" 60 degrees with horizontal

Bend up 2 at 1'-6" 60 degrees with horizontal

Place 2 - 1" @ straight in top.
Pilasters & Beams

Scale - 1/4" = 1'

Loads for Bending

Loads for Shear
Shear in Beams -

\[ R = (6.25 \times 2927) \div 16,770 \div \left[ 16,770 \times (3.5 \div 6.5) \right] \div 12.5 = 49,490 = V \]

\[ V = \frac{49,490}{(20 \times 24.5)} = 101 \text{ lbs. per sq. in.} \]

\[ V' = 49,490 - (20 \times 24.5 \times 40) = 29,860 \text{ lbs.} \]

Bent up bars - \( 2 \times 0.785 \times 16,000 \times .066 = 21,700 \text{ lbs.} \)

\[ V'' \text{ (stirrups)} = 29,860 - 21,700 = 8,160 \text{ lbs.} \]

\[ V \text{ at } 2'-6'' \text{ from support} = 49,490 - \left[ 16,770 \div (2927 \times 2.5) \right] = 25,400 \text{ lbs.} \]

\[ V'' = 25,400 - 19,600 = 5,800 \text{ lbs.} \]

For 3/8" Ø stirrups - \( s = \frac{24.5 \times .392 \times 16,000}{11,160} = 19'' \)

Cantilever -

\[ V = (2.25 \times 2927) \div (2 \times 635) \div 16,770 \div 6,130 = 30,760 \text{ lbs.} \]

\[ V' = 30,760 - (20 \times 24.5 \times 40) = 11,160 \text{ lbs.} \]

For 3/8" Ø stirrups - \( s = \frac{24.5 \times .392 \times 16,000}{11,160} = 13.8 \text{ in.} \)

Space 3/8" Ø stirrups 13 1/2" - C. to C. beginning 8" from center.

5 stirrups to each side.

Space 3/8" Ø stirrups 13 1/2" - C. to C. in Cantilever beginning 8" from Col. face. 3 stirrups to each side.

Weight of Beams -

Conc. - \( (20 \times 30 \times 24) - (20 \times 12 \times 2) = 13,920 \text{ lbs.} \)

Steel - 8-1" Ø @ 28.5' = \( 8 \times 28.5 \times 2.67 = 566 \)

- 16-3/8" Ø @ 7.33' = \( 16 \times 7.33 \times 0.67 = 72 \)

\( \frac{645}{645} = 645 \text{ lbs.} \)

Total - 14,565 lbs.
Pilasters

Max. Length = 17' - 00"

Batter 4" per ft.

Dimensions of top - 20" x 20"

Dimensions of Base - 28½" x 28½"

Steel - 4 - 1" φ and 3/8" φ hoops at 12" C. to C.

Weight of Pilaster -

Conc. - \[ \left( \frac{20 \times 20}{2} \right) + \left( \frac{28.5 \times 28.5}{2} \right) \times \frac{17}{2} = 10,325 \text{ lbs.} \]

Steel - 4 - 1" φ & 22' = 88 \times 2.67 = 225 \text{ lbs.}

- 17 - 3/8" φ & 9' = 153 \times 0.38 = 58 \text{ lbs.}

Total = 10,618 lbs.

Total Dead Load on one Pilaster (max.)

Pilaster - 10,618 lbs.

Beam - \[ \frac{14,565}{2} = 7,283 \text{ lbs.} \]

Slab - \[ \frac{45,825}{2} = 22,918 \text{ lbs.} \]

Curb - 6,232 lbs.

Total = 47,051 lbs.

Allowable unit stress on conc. =

\[ 600 - \frac{(15 \frac{1}{4})}{D} = 600 - \frac{(15 \times 17 \times 12)}{20} = 447 \text{ lbs. per sq. in.} \]

Total unit stress -

\[ 447 \times (0.00785 \times 447 \times 14) = 415 \times 49 = 464 \text{ lbs. per sq. in.} \]

\[ P = 464 \times 400 = 185,600 \text{ lbs.} \]
Pedestal Bents

Weight of cap -

- Conc. \((24 \times 36) \neq (6 \times 4)\) = 896 lbs. per lin. ft.
- Steel (assumed) = 45 lbs. per lin. ft.
- Total = 941 lbs. per lin. ft.

D. L. from slab = \(\frac{56,100}{(20 \times 2)}\) = 1,463 lbs. per lin. ft.

Curtain wall extra width = \(12 \times 34\) = 408 lbs. per lin. ft.

Total D. L. over 20 ft. = 2,602 lbs. per lin. ft.

Bending

\[ R = (2,800 \times 6) \neq 56,000 = 72,800 \text{ lbs.} \]

\[ B.M. = (72,800 \times 6) - (56,000 \times 3.12) - (16,800 \times 3) = 436,600 - (175,000 \neq 50,400) = 311,400 \text{ ft. lbs.} \]

\[ d = 33", \quad j_d = 26.875" \times 2.406", \quad k_d = 12.375" \]

\[ f_c = \frac{311,400 \times 2}{(2.406 \times 24 \times 12.37)} = 595 \text{ lbs. per sq. in.} \]

\[ A_s = \frac{311,400}{(2.406 \times 16,000)} = 5.48 \text{ sq. in. (6 1" sq. bars)} \]

Cantilever

\[ B.M. = (53,900 \times 1.21) \neq (5,600) \neq (1,880 \times 3) = 76,440 \text{ foot lbs.} \]

\[ A_s = \frac{76,440}{(2.406 \times 14,000)} = 1.99 \text{ sq. in.} \]

Carry 2 lin. sq. bars in top

Bend up 2 \(3'-2"\) from Center

Bend up 2 \(3'-10"\) from Center

Angle of bend = arc. tan. \(33/16 = 64 \text{ degrees} - 10'\)

Carry bent up bars past outer face of column and hook.
Shear

\[ V = 72,800 \]

\[ V' = 72,800 - (24 \times 28.87 \times 40) = 72,800 - 27,700 = 45,100 \text{ lbs.} \]

Bent up bars - \( 2 \times 16,000 \times 0.900 = 28,800 \text{ lbs.} \)

\[ V'' = 45,100 - 28,800 = 16,300 \text{ lbs.} \]

Using 5/8" @ stirrup -

\[ \text{No.} = \frac{16,300}{2 \times 0.307 \times 16,000} = 1.7 \]

\[ \text{Spacing} = \frac{28.87}{1.7} = 16.9" \]

Use 5 each side @ 15"

Shear on Cantilever

\[ V = (2 \times 2,800) \div (2 \times 940) \div 53,900 = 61,380 \text{ lbs.} \]

\[ V' = 61,380 - 27,700 = 33,680 \text{ lbs.} \]

Stirrups -

\[ \text{No.} = \frac{33,680}{2 \times 0.307 \times 16,000} = 3.43 \]

\[ \text{Spacing} = \frac{28.87}{3.43} = 8.4" \]

Use 5 each side @ 8"

Cols. -

With 4 1 in. sq. bars effective area = \( \frac{4}{0.007} = 570 \text{ sq. in.} \)

\[ 600 - \frac{154}{2} = 600 - \frac{15 \times 20.6}{2} = 444 \text{ lbs. per sq. in.} \]

Allowable stress = \( 444 \div (0.007 \times 444 \times 15) = 450 \text{ lbs. per sq. in.} \)

\[ P = \frac{17}{22} \times 490 \times 570 = 215,000 \text{ for Class } "F" \]

Total Load = \( (2,800 \times 10) \div (940 \times 2) \div 53,900 \div 56,000 \div \]

\[ \left[ (24 \times 24) \div (35 \times 35) \right] \times 10.4 \div (4 \times 3.43 \times 25) = 158,900 \text{ lbs.} \]
**Foundations**

Load from Col. - 158,900 lbs.

L.L. = Int. Gird. \((5,520 \times 24) / 36 \) \(\times 22,040 = 28,530 \text{ lbs.}\)

Ext. Gird. \((4,080 \times 24) / 36 \) \(\times 16,320 = 18,900 \text{ lbs.}\)


Weight of Abutment -

Section under Col. - \(42 \times 42 \times 0.5 \) \(= 682 \text{ lbs.}\)

\(52 \times 66 \times 21 \) \(= 72,100 \text{ lbs.}\)

72,982 lbs.

Rectangular Section - \(18 \times 36 \times 66 \times 72,982 \text{ lbs.}\)

Trapezoidal Section - \((18 \n/ 3) \times \frac{13.67}{2} \times 5.5 \times 144 \times 113,900 \text{ lbs.}\)

Center of Gravity -

Load Concentric with Col. - 114,470 lbs.

72,800 lbs.

187,450 lbs.

\((187,450 \times 2.17) / (42,800 \times 5.63) / (113,900 \times 12.56) \times (187,450 / 42,800 / 113,900) = 6.06' \text{ from face of Abutment.}\)
Arch Abutment and Skewback.
Scale - \( \frac{1}{4}" = 1' \)
**Division of Arch Ring**

Is is based on 0.5% of area of crown in each face.

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<thead>
<tr>
<th>No.</th>
<th>Trial Length</th>
<th>Trial Depth</th>
<th>1/2 Depth - 3/16</th>
<th>Depth Cubed</th>
<th>R^2</th>
<th>Io</th>
<th>Is</th>
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<th>Length</th>
<th>C. - C.</th>
<th>Depths Revised</th>
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\[
\Delta x = \frac{59.13 \times 0.1649}{10} = 0.974
\]
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<table>
<thead>
<tr>
<th>Coordinates of Loads</th>
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</thead>
<tbody>
<tr>
<td><strong>Point</strong></td>
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<tr>
<td>Base of Ring - P-1</td>
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<tr>
<td>Pilaster No. 1 -P-2</td>
</tr>
<tr>
<td>Pilaster No. 2 -P-3</td>
</tr>
<tr>
<td>Pilaster No. 3 -P-4</td>
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<tr>
<td>Pilaster No. 4 -P-5</td>
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</table>
### Loads to Arch Ring

#### Weight of Pilasters -

<table>
<thead>
<tr>
<th>Pilaster</th>
<th>Weight (lbs)</th>
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<tbody>
<tr>
<td>17' Pilaster</td>
<td>10,618</td>
</tr>
<tr>
<td>10' Pilaster</td>
<td>5,125</td>
</tr>
<tr>
<td>Conc. ( \frac{1}{4} \left( (20 \times 20) \div (25 \times 25) \right) \times 10 )</td>
<td>160</td>
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<tr>
<td>Steel - 4 - 1' ( \phi ) @ 15' = 60 x 2.67</td>
<td>32</td>
</tr>
<tr>
<td>Total -</td>
<td>5,317</td>
</tr>
<tr>
<td>5.8' Pilaster</td>
<td>2,700</td>
</tr>
<tr>
<td>Conc. ( \frac{1}{4} \left( (20 \times 20) \div (23 \times 23) \right) \times 5.8 )</td>
<td>117</td>
</tr>
<tr>
<td>Steel - 4 - 1' ( \phi ) @ 11' = 44 x 2.67</td>
<td>18</td>
</tr>
<tr>
<td>Total -</td>
<td>2,835</td>
</tr>
<tr>
<td>3.8' Pilaster</td>
<td>1,680</td>
</tr>
<tr>
<td>Conc. ( \frac{1}{4} \left( (20 \times 20) \div (22 \times 22) \right) \times 3.8 )</td>
<td>96</td>
</tr>
<tr>
<td>Steel - 4 - 1' ( \phi ) @ 9' = 36 x 2.67</td>
<td>12</td>
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<tr>
<td>Total -</td>
<td>1,788</td>
</tr>
</tbody>
</table>

#### Super Struct. Dead Loads -

| Pilaster No. 1 - (17') | 47,050 |
| Pilaster No. 2 - (10') | 41,740 |
| Pilaster No. 3 - (5.8') | 39,267 |
| Pilaster No. 4 - (3.8') | 38,220 |
Weight of Arch Ring

These weights are computed by taking the average scaled dimensions and using 150 lbs. per cu. ft. as the weight of concrete and steel.

Width of ring assumed 3'.

Below Pilaster No. 1

(3.66 x 10.66 x 3 x 150) 17,300 lbs.

At Pilaster No. 1

(3.32 x 12.66 x 3 x 150) 18,940 lbs.

At Pilaster No. 2

(2.87 x 12.33 x 3 x 150) 15,950 lbs.

At Pilaster No. 3

(2.44 x 11.20 x 3 x 150) 13,000 lbs.

At Pilaster No. 4

(2.24 x 11.50 x 3 x 150) 11,600 lbs.

Total Dead Loads

Base of Ring - 17,300 lbs.

Pilaster No. 1 - 47,050 lbs.

18,940 lbs. 65,990 lbs.

Pilaster No. 2 - 41,749 lbs.

15,950 lbs. 57,699 lbs.

Pilaster No. 3 - 39,267 lbs.

13,000 lbs. 52,267 lbs.

Pilaster No. 4 - 38,220 lbs.

11,600 lbs. 49,820 lbs.
Pilaster Loads for Changing L.L.

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<tr>
<th></th>
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</thead>
<tbody>
<tr>
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<td>71,560</td>
<td>63,270</td>
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<td>11'-7&quot;</td>
<td>11'-7&quot;</td>
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<tr>
<td>#2</td>
<td>71,560</td>
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<td>11'-7&quot;</td>
<td>11'-7&quot;</td>
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<td>71,560</td>
<td>63,270</td>
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<tr>
<td>#4</td>
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</tr>
<tr>
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Note: Base of Arch Ring - 17,300 # at x = 48.10
### Loading No. 1

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<th>y</th>
<th>$x^2$</th>
<th>$y^2$</th>
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<th>mR</th>
<th>(mL - mR)y</th>
<th>(mR - mL)x</th>
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</table>

\[
\begin{align*}
\text{Ho} & = \frac{n \times \text{Sum. mR} - \text{Sum. mL}}{2} = \frac{10 \times (-346,961,336) - (-29,532,196 \times 48.2)}{2} = 300,708 \\
\text{Vo} & = \frac{\text{Sum. mR} - \text{mL}}{\text{Sum. x}} = 0 \quad \text{Ho} = \frac{\text{Sum. m} \times \text{mR} - \text{mL} \times \text{m}}{2} = \frac{-29,532,196 \times 48.2}{2} = 27,197
\end{align*}
\]
Thruts - Loading No. 1
Scale - 1" = 50,000 #
### Loading No. 2

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<th>mR</th>
<th>(mL - mR) y</th>
<th>(mR - mL)x</th>
</tr>
</thead>
<tbody>
<tr>
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<td>-116,080</td>
<td>-109,115</td>
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<td>-282,977</td>
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<td>-459,638</td>
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<tr>
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<td>-4,933,577</td>
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<tr>
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<tr>
<td>10</td>
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<td>-49,206,941</td>
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</table>

**Summary**

<table>
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<tr>
<th>Point</th>
<th>mL</th>
<th>mR</th>
<th>(mL - mR) y</th>
<th>(mR - mL)x</th>
</tr>
</thead>
<tbody>
<tr>
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<td>-315,381,843</td>
<td>-49,206,941</td>
<td>49,206,941</td>
</tr>
</tbody>
</table>

\[ \text{Ho} = \frac{(10 \times 315,381,843) - (-26,193,977 \times 49.2)}{-6871} = \frac{-1,897,266,739}{-6871} = 275,254 \]

\[ \text{Vo} = \frac{49,206,941}{41,370} = 4,328 \]

\[ \text{Mo} = \frac{-26,193,977 \times (2 \times 275,254 \times 49.2)}{20} = -17,025 \]
### Loading No. 3

<table>
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<th>(mL / mR)y</th>
<th>(mR - mL)x</th>
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<tbody>
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<td></td>
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<td></td>
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**Summation**

<table>
<thead>
<tr>
<th>mL</th>
<th>mR</th>
<th>(mL / mR)y</th>
<th>(mR - mL)x</th>
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</thead>
<tbody>
<tr>
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**Springing**

<table>
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<th>mL</th>
<th>mR</th>
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</thead>
<tbody>
<tr>
<td>-7,570,564</td>
<td>-6,795,429</td>
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</tbody>
</table>

\[
Ho = \frac{(10 \times -336,209,976) - (-26,156,124 \times 46.2)}{-6671} = 291,768
\]

\[
Vo = \frac{49,161,609}{41,370} = 4,325
\]

\[
Mo = \frac{26,156,124}{20}(2 \times 291,768 \times 48.2) = 1,488
\]
Thrusts - Loading No. 3
Scale - 1" = 50,000#.
<table>
<thead>
<tr>
<th>Point</th>
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<th>mR</th>
<th>(mL + mR)y</th>
<th>(mL - mL)x</th>
</tr>
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<tbody>
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<tr>
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<tr>
<td>Springing</td>
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<td></td>
<td>-7,112,072</td>
<td></td>
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</tbody>
</table>

\[
H_o = \frac{(10 \times -338,100,598) - (-26,812,840 \times 48.2)}{6871} = \frac{289,947}{6871}
\]

\[
V_o = 0
\]

\[
H_o = \frac{-26,812,840 + (2 \times 289,947 \times 48.2)}{20} = \frac{43,098}{20}
\]
Thruts - Loading No. 4
Scale - 1" = 50,000#
### Loading No. 5

<table>
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<th>Point</th>
<th>mL</th>
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<th>(mL - mR)x</th>
<th>(mR - mL)x</th>
</tr>
</thead>
<tbody>
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<td><strong>-311,896,627</strong></td>
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<td>6,898,263</td>
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</table>

\[
H_0 = (10 \times -311,896,627) - (-25,601,972 \times 48.2) = 727,931
\]

\[
V_0 = 0
\]

\[
H_0 = -25,601,972 + (2 \times 727,931 \times 48.2) = -25,429
\]
Thrusts - Loading No. 5.
Scale - 1" = 50,000#.
Line of Pressure to Foundation
Full Live Load.
Scales - \(\frac{1}{8}'' = 1'\) & \(1'' = 40,000\) Lbs.
Temperature Stresses

For a 30 degree Rise:

\[ H_0 = \frac{286,000,000 \times 0.000006 \times 30 \times 100 \times 10}{0.974 \times 2(5758.62 - 2323.24)} = 7,746 \text{ lbs.} \]

\[ M_0 = \frac{H_0 \text{ sum. } y}{n} = \frac{7746 \times 46.2}{10} = 37.336 \]

For a 40 degree Fall: \( M \) and \( H \) are \( 1.33 \times \frac{M}{H} \) for 30 degree Rise.

Temperature Moments and Thrusts of all Points - \( M = \frac{M_0}{H_0} \).

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<thead>
<tr>
<th>Point</th>
<th>For 30 Degree Rise</th>
<th>For 40 Degree Fall</th>
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<td>H</td>
<td>M</td>
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</table>
Shortening of Arch

\[
H_0 = \frac{1}{0.974} \times \frac{240 \times 100 \times 10 \times 144}{2(6758.62 - 2323.24)} = 5,160 \text{ lbs.}
\]

\[
\frac{5160}{7745} = 0.67
\]

H and M for shortening of arch are 67% of those for Temp. Rise of 30 degrees.

<table>
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<th>Thrust</th>
<th>Av. fc</th>
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<td>300,700</td>
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<tr>
<td>3</td>
<td>1188</td>
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<td>254</td>
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<td>307,000</td>
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<td>324,000</td>
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</tr>
<tr>
<td>8</td>
<td>1476</td>
<td>352,500</td>
<td>239</td>
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<tr>
<td>9</td>
<td>1656</td>
<td>352,500</td>
<td>213</td>
</tr>
<tr>
<td>10</td>
<td>1854</td>
<td>394,000</td>
<td>213</td>
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</table>

\[
\text{Av. fc} = 240 \text{ lbs. per sq. in.}
\]

<table>
<thead>
<tr>
<th>Point</th>
<th>M</th>
<th>H</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crown</td>
<td>$24,890$</td>
<td>5,160</td>
</tr>
<tr>
<td>1</td>
<td>$24,890$</td>
<td>5,160</td>
</tr>
<tr>
<td>2</td>
<td>$24,166$</td>
<td>5,160</td>
</tr>
<tr>
<td>3</td>
<td>$22,464$</td>
<td>5,160</td>
</tr>
<tr>
<td>4</td>
<td>$19,621$</td>
<td>5,080</td>
</tr>
<tr>
<td>5</td>
<td>$15,700$</td>
<td>5,050</td>
</tr>
<tr>
<td>6</td>
<td>$10,070$</td>
<td>4,960</td>
</tr>
<tr>
<td>7</td>
<td>$1,760$</td>
<td>4,850</td>
</tr>
<tr>
<td>8</td>
<td>$11,154$</td>
<td>4,630</td>
</tr>
<tr>
<td>9</td>
<td>$33,359$</td>
<td>4,300</td>
</tr>
<tr>
<td>10</td>
<td>$74,050$</td>
<td>3,720</td>
</tr>
<tr>
<td>Spring</td>
<td>$-109,790$</td>
<td>3,270</td>
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</table>
Moments at Crown, Haunch and Springing

for all conditions of loading.

\[ M = m + Mo + Ho y + Vo x \]

<table>
<thead>
<tr>
<th>Point</th>
<th>Loading #1</th>
<th>Loading #2</th>
<th>Loading #3</th>
<th>Loading #4</th>
<th>Loading #5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crown</td>
<td>( \mp 27,200 )</td>
<td>- 17,025</td>
<td>( \mp 1,488 )</td>
<td>( \mp 43,098 )</td>
<td>- 25,429</td>
</tr>
<tr>
<td>7 Left</td>
<td>- 74,850</td>
<td>( \mp 20,910 )</td>
<td>- 1,746</td>
<td>- 90,058</td>
<td>- 17,314</td>
</tr>
<tr>
<td>7 Right</td>
<td>- 74,850</td>
<td>- 83,050</td>
<td>- 105,783</td>
<td>- 90,058</td>
<td>- 17,314</td>
</tr>
<tr>
<td>8 Left</td>
<td>( \mp 8,725 )</td>
<td>( \mp 73,570 )</td>
<td>( \mp 45,873 )</td>
<td>- 18,026</td>
<td>( \mp 80,472 )</td>
</tr>
<tr>
<td>8 Right</td>
<td>( \mp 8,725 )</td>
<td>( \mp 12,245 )</td>
<td>- 15,558</td>
<td>- 18,026</td>
<td>( \mp 80,472 )</td>
</tr>
<tr>
<td>Spring. Lt.</td>
<td>( \mp 459,450 )</td>
<td>( \mp 178,617 )</td>
<td>( \mp 268,824 )</td>
<td>( \mp 492,026 )</td>
<td>( \mp 194,308 )</td>
</tr>
<tr>
<td>Spring. Rt.</td>
<td>( \mp 459,450 )</td>
<td>- 497,904</td>
<td>( \mp 537,759 )</td>
<td>( \mp 492,026 )</td>
<td>( \mp 194,308 )</td>
</tr>
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</table>
### Summary of Maximum Moments and Thrusts at Crown, Haunch and Springing

<table>
<thead>
<tr>
<th></th>
<th>Crown</th>
<th>Point 7</th>
<th>Point 8</th>
<th>Springing</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M</td>
<td>H</td>
<td>M</td>
<td>H</td>
</tr>
<tr>
<td><strong>Loads</strong></td>
<td>43,098</td>
<td>289,947</td>
<td>20,910</td>
<td>298,500</td>
</tr>
<tr>
<td><strong>Temp.</strong></td>
<td>49,780</td>
<td>10,300</td>
<td>3,510</td>
<td>9,600</td>
</tr>
<tr>
<td><strong>Arch Short.</strong></td>
<td>24,890</td>
<td>5,160</td>
<td>1,780</td>
<td>4,830</td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td>117,768</td>
<td>305,407</td>
<td>26,180</td>
<td>312,950</td>
</tr>
<tr>
<td><strong>Loads</strong></td>
<td>-25,429</td>
<td>272,931</td>
<td>-105,763</td>
<td>313,000</td>
</tr>
<tr>
<td><strong>Temp.</strong></td>
<td>-37,336</td>
<td>7,740</td>
<td>-2,634</td>
<td>7,220</td>
</tr>
<tr>
<td><strong>Arch Short.</strong></td>
<td>-24,890</td>
<td>5,160</td>
<td>1,780</td>
<td>4,830</td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td>-37,875</td>
<td>285,831</td>
<td>-106,637</td>
<td>325,050</td>
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</table>
### Maximum Moments, Thrusts and Eccentric Distances

<table>
<thead>
<tr>
<th>Point</th>
<th>Positive M</th>
<th>Negative M</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>M</td>
<td>K</td>
</tr>
<tr>
<td>Crown</td>
<td>117,768</td>
<td>305,407</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>86,049</td>
<td>331,540</td>
</tr>
<tr>
<td>Spring</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Stresses in extreme fibers are determined by Hoole and Johnson's equations and diagrams for k and L. Tension is developed only at the springing as shown by the values of x₀ for zero f₀'.

### Steel in Arch Ring

5 - 1 inch square bars in each face from springing to haunch - length 38 ft.
5 - 1 inch round bars in each face from haunch to crown - length 25 ft.

118 - 5/8 inch ø hoops to each ring spaced 24 inches C. to C.

average length - 13 ft.

4 - 1 inch square dowels in each abutment through skewback - length 12 ft.
Properties of Sections

<table>
<thead>
<tr>
<th></th>
<th>Crown</th>
<th>Div. No. 7</th>
<th>Div. No. 8</th>
<th>Springing</th>
</tr>
</thead>
<tbody>
<tr>
<td>t</td>
<td>2.17</td>
<td>2.84</td>
<td>3.89</td>
<td>3.89</td>
</tr>
<tr>
<td>p</td>
<td>.0042</td>
<td>.0041</td>
<td>.0030</td>
<td></td>
</tr>
<tr>
<td>r</td>
<td>.88</td>
<td>1.22</td>
<td>1.74</td>
<td></td>
</tr>
<tr>
<td>d'</td>
<td>.21</td>
<td>.21</td>
<td>.21</td>
<td></td>
</tr>
<tr>
<td>d</td>
<td>1.96</td>
<td>2.63</td>
<td>3.68</td>
<td></td>
</tr>
<tr>
<td>n</td>
<td>15.00</td>
<td>15.00</td>
<td>15.00</td>
<td></td>
</tr>
<tr>
<td>2p</td>
<td>.0083</td>
<td>.0062</td>
<td>.006</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>936.00</td>
<td>1227.00</td>
<td>1679.00</td>
<td></td>
</tr>
</tbody>
</table>

xo for zero σ0

\[ xo = \frac{t^2}{24np} \frac{p^2}{x} \frac{1}{1 + \frac{p}{2p}} \]

Crown - \( xo = \frac{2.17^2}{24 \times 0.0042 \times 2.17} \times \frac{1}{\frac{1}{0.125}} \times \frac{1}{6 \times 2.17} \times 0.401 \)

No. 7 - \( xo = \frac{2.57^2}{24 \times 0.0035 \times 2.57} \times \frac{1}{\frac{1}{0.105}} \times \frac{1}{6 \times 2.57} \times 0.474 \)

No. 8 - \( xo = \frac{2.84^2}{24 \times 0.0041 \times 2.84} \times \frac{1}{\frac{1}{0.123}} \times \frac{1}{6 \times 2.84} \times 0.536 \)

Spring. - \( xo = \frac{3.89^2}{24 \times 0.003 \times 3.89} \times \frac{1}{\frac{1}{0.09}} \times \frac{1}{6 \times 3.89} \times 0.713 \)
Fibre Stresses of Sprung

\[ \frac{k}{L} = \frac{2p}{t} = 0.56 \quad (\text{from diagram}) = 0.437 \]

\[ \frac{\Delta}{t} = \frac{0.21}{3.89} = 0.054 \quad L (\text{from diagram}) = 0.14 \]

\[ \frac{f_0}{L} = \frac{627.264 \times 12}{0.14 \times 36 \times 46.75^2} = 900 \text{ lbs. per sq. in} \]

\[ \frac{fa}{L} = \frac{15 \times 900}{\left( \frac{3.68}{3.89 \times 0.437} \right) - 1} = 15,660 \text{ lbs. per sq. in.} \]

Fibre Stresses at Crown and Points No. 7 and No. 8

\[ \frac{f_0}{bt} = \frac{H}{\left( 1 + n \frac{2p}{t^2} \right) \frac{6 \times x_0 \frac{t}{2}}{12 + 2n \frac{p}{r^2}}} \]

Crown - \( f_0 = \frac{305.407}{36 \times 26} \left( \frac{1}{1.126} + \frac{6 \times 4.56 \times 26}{676 \times (180 \times .0083 \times .774 \times 144)} \right) = 564 \text{ lbs. per sq. in.} \)

Point No. 7 - \( f_0 = \frac{325.080}{36 \times 30.94} \left( \frac{1}{1.166} + \frac{6 \times 4 \times 30.94}{953 \times (180 \times .0097 \times 1.166 \times 144)} \right) = 449 \text{ lbs. per sq. in.} \)

Point No. 8 - \( f_0 = \frac{331.540}{36 \times 34.12} \left( \frac{1}{1.123} + \frac{6 \times 3.12 \times 34.12}{1163 \times (180 \times .0082 \times 1.488 \times 144)} \right) = 356 \text{ lbs. per sq. in.} \)

Point No. 6 - \( f_0 = \frac{352.010}{36 \times 34.12} \left( \frac{1}{1.123} + \frac{6 \times 1.8 \times 34.12}{1163 \times (180 \times .0082 \times 1.488 \times 144)} \right) = 324 \text{ lbs. per sq. in.} \)

The stresses at all points except the springing are very low. This is

due to the fact that the line of pressure follows the arch axis so closely

that the stress is entirely compressive. As a very slight reduction in the

arch section would cause a considerable change in eccentricity and a pro-

portionately higher \( f_0 \), the present section will be held. The saving in

concrete effected by such a change would be negligible.
### Quantities

**Design No. 2 - Concrete**

- **Slabs and Curbs**  
  \[11.6 \times 9 \times \left(23 / 3\right) \times \frac{1}{27} = 100.53 \text{ C.Y.}\]

- **Beams**  
  \[8 \times \left[\left(24 \times 20 \times 2.5\right) - \left(1 \times 2 \times 20\right)\right] \times \frac{1}{27} = 28.60 \text{ C.Y.}\]

- **Pilasters No. 1**  
  \[4 \times 17 \left[\left(20 \times 20\right) / \left(28.5 \times 28.5\right)\right] \times \frac{1}{27} = 10.60 \text{ C.Y.}\]

- **Pilasters No. 2**  
  \[4 \times 10 \left[\left(20 \times 20\right) / \left(25 \times 25\right)\right] \times \frac{1}{27} = 5.27 \text{ C.Y.}\]

- **Pilasters No. 3**  
  \[4 \times 5.8 \left[\left(20 \times 20\right) / \left(23 \times 23\right)\right] \times \frac{1}{27} = 2.76 \text{ C.Y.}\]

- **Pilasters No. 4**  
  \[4 \times 3.8 \left[\left(20 \times 20\right) / \left(22 \times 22\right)\right] \times \frac{1}{27} = 1.75 \text{ C.Y.}\]

**Subtotal**  
48.98 C.Y.

- **Pedestal Bent Cols.**  
  \[4 \times 20.8 \left[\left(24 \times 24\right) / \left(35 \times 35\right)\right] \times \frac{1}{27} = 19.27 \text{ C.Y.}\]

- **Pedestal Bent Caps.**  
  \[\left[\left(2 \times 24 \times 3 \times 2\right) / \left(2 \times 4\right)\right] \times \frac{1}{27} = 10.95 \text{ C.Y.}\]

**Subtotal**  
30.23 C.Y.

- **Arch Abutments**  
  \[\text{Area} = \left(21 \times 4.33\right) / \left(18 \times 3\right) / \left(10.5 \times 13.66\right) = 288.4 \text{ sq. ft.}\]
  \[\text{Width} = 5.5 \text{ ft.}\]
  \[\text{Deduction for skewback} = 3.2 \times 5 \times 3 = 57.6 \text{ Cu. Ft.}\]
  \[\text{Vol.} = 4 \times \left\{288.4 \times 5.5\right\} - 57.6 \times \frac{1}{27} = 226.5 \text{ Cu. Yds.}\]

- **Arch Rings**  
  \[\text{Area} = 354 \text{ sq. ft.}\]
  \[\text{Width} = 3 \text{ ft.}\]
  \[\text{Vol.} = 2 \times 354 \times 3 \times \frac{1}{27} = 78.67 \text{ Cu. Yds.}\]
<table>
<thead>
<tr>
<th>Quantities</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Design No. 2 - Steel</strong></td>
</tr>
<tr>
<td>Slabs</td>
</tr>
<tr>
<td></td>
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<tr>
<td></td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
</tr>
<tr>
<td>Beams</td>
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<tr>
<td></td>
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<tr>
<td><strong>Subtotal</strong></td>
</tr>
<tr>
<td>Pilasters</td>
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</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
</tr>
<tr>
<td>Pedestal Elements</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
</tr>
<tr>
<td>Col. - 16 - 1&quot; sq. bars @ 25' = 400 x 3.43 = 1372 lbs.</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
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<tr>
<td>Arch Ringes</td>
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<tr>
<td></td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
</tr>
<tr>
<td>Abutments</td>
</tr>
</tbody>
</table>
Quantities

Design No. 2

Excavation for Abutments -

Average ground elevation - 460.0

Foundation elevation - 445.5

Depth - 14.3

Area = 23 x 7.6 = 172.5 sq. ft.

Vol. = 4 x 14.7 x 172.5 = 10,143 cu. ft. = 375.7 cu. yds.
Summary of Total Quantities - Design No. 2

Concrete

<table>
<thead>
<tr>
<th>Description</th>
<th><strong>Class &quot;A&quot;</strong></th>
<th><strong>Class &quot;B&quot;</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>2 End Bents</td>
<td>19.36 C.Y.</td>
<td>97.76 C.Y.</td>
</tr>
<tr>
<td>2 Intermediate Bents</td>
<td>14.22 C.Y.</td>
<td>77.15 C.Y.</td>
</tr>
<tr>
<td>2 Pedestal Bents</td>
<td>20.23 C.Y.</td>
<td></td>
</tr>
<tr>
<td>4 Arch Abutments</td>
<td></td>
<td>226.50 C.Y.</td>
</tr>
<tr>
<td>2 Arch Rings</td>
<td>78.67 C.Y.</td>
<td></td>
</tr>
<tr>
<td>8 Pilasters and Beams</td>
<td>48.96 C.Y.</td>
<td></td>
</tr>
<tr>
<td>4 - 40' Decks</td>
<td>111.58 C.Y.</td>
<td></td>
</tr>
<tr>
<td>9 Slabs</td>
<td>100.53 C.Y.</td>
<td></td>
</tr>
<tr>
<td><strong>Totals</strong></td>
<td>404.17 C.Y.</td>
<td>401.41 C.Y.</td>
</tr>
</tbody>
</table>

Steel

<table>
<thead>
<tr>
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<th><strong>Steel</strong></th>
</tr>
</thead>
<tbody>
<tr>
<td>2 End Bents</td>
<td>7,138 lbs.</td>
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<tr>
<td>2 Intermediate Bents</td>
<td>8,220 lbs.</td>
</tr>
<tr>
<td>2 Pedestal Bents</td>
<td>3,464 lbs.</td>
</tr>
<tr>
<td>4 Arch Abutments</td>
<td>659 lbs.</td>
</tr>
<tr>
<td>2 Arch Rings</td>
<td>9,496 lbs.</td>
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<tr>
<td>8 Pilasters &amp; Beams</td>
<td>7,999 lbs.</td>
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<tr>
<td>4 - 40' Decks</td>
<td>48,912 lbs.</td>
</tr>
<tr>
<td>9 Slabs</td>
<td>9,729 lbs.</td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td>95,619 lbs.</td>
</tr>
<tr>
<td>Bearing Plates and Bolts</td>
<td>1,355 lbs.</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>96,954 lbs.</td>
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Excavation

<table>
<thead>
<tr>
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<th><strong>Excavation</strong></th>
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</thead>
<tbody>
<tr>
<td>2 End Bents</td>
<td>149.1 C.Y.</td>
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<tr>
<td>2 Intermediate Bents</td>
<td>185.1 C.Y.</td>
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<td>4 Abutments</td>
<td>375.7 C.Y.</td>
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<tr>
<td><strong>Total</strong></td>
<td>709.9 C.Y.</td>
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</table>
## Estimates - Designs No. 1 and No. 2

<table>
<thead>
<tr>
<th>Item</th>
<th>Design No. 1</th>
<th>Design No. 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Class &quot;A&quot; Concrete</td>
<td>428.20 C.Y.</td>
<td>404.17 C.Y.</td>
</tr>
<tr>
<td>Class &quot;B&quot; Concrete</td>
<td>356.22 C.Y.</td>
<td>401.41 C.Y.</td>
</tr>
<tr>
<td>Reinforcing Steel</td>
<td>114,750.00 Lbs.</td>
<td>96,354.00 Lbs.</td>
</tr>
<tr>
<td>4&quot; Concrete Paving</td>
<td>587.00 Sq. Yds.</td>
<td>587.00 Sq. Yds.</td>
</tr>
<tr>
<td>Hand Rail</td>
<td>528.00 Lin. Ft.</td>
<td>528.00 Lin. Ft.</td>
</tr>
<tr>
<td>Excavation</td>
<td>470.10 C.Y.</td>
<td>709.90 C.Y.</td>
</tr>
</tbody>
</table>
Conclusions

Obviously the slight difference in cost between the two designs is in favor of the deck girders as constructed. A considerable saving could be effected by lowering the spring line in Design No. 2, but it is not considered advisable to place the spring line below water level at the location of the abutments in this design.

At the present time a steel I beam design would probably be more economical than either of those considered here, but at the time the original design was made the present deep sections were not being rolled and for this reason such a design has not been considered in the preparation of this paper.
STRUCTURE AND WEST APPROACH
RIP RAP - WEST END
STRUCTURE AND APPROACH HILLS