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Renewal of the Milwaukee Avenue viaduct
THESIS
THE RENEWAL
OF THE
MILWAUKEE AVENUE VIADUCT,

Presented By

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to the

PRESIDENT AND FACULTY
of the

ARMOUR INSTITUTE OF TECHNOLOGY

for the degree of

BACHELOR OF SCIENCE IN CIVIL ENGINEERING,

having completed the prescribed course of study

in

CIVIL ENGINEERING

1912

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ILLINOIS INSTITUTE OF TECHNOLOGY
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THE RENEWAL
OF THE
MILWAUKEE AVENUE VIADUCT.
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34. Design of rail fastening in concrete.

35. General Survey of viaduct

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VIEW OF VIADUCT. (Looking East)
VIEW OF NORTH APPROACH. (Looking South)
VIEW OF PONY TRUSSES (Looking South)
The Renewal of the Milwaukee Avenue Viaduct.

Part I.

This viaduct is located along Milwaukee Avenue, its northern terminal being Kinzie Street and its southern, Carroll Avenue. It spans the tracks of the Chicago and Northwestern, the Chicago, Milwaukee and St. Paul, and the Pennsylvania R. R's.

The primary reason for renewal is, that first it is too light a structure to adequately accommodate the street and car traffic to which it is continually subjected; and secondly, it allows too little clearance for the railroad freight traffic. When the present structure was constructed the street and car traffic was, as compared with now, very much smaller. This is especially true of the street cars. The thru structures of the present system are only heavy enough to support one of the cars used at present, at a time. Again, the increased freight traffic has caused the increased size of freight cars, this resulting in decreased clearance under the structure.

The present viaduct, beginning at the north abutment, consists of: eight spans of deck plate girders of various lengths, two of thru Pony trusses
and one of an over-head braced thru truss. To counteract the existing conditions as set forth in the previous paragraph, the proposed structure will be made heavier, the grade will be slightly raised and the floor system made shallow; the effect of the last two being to increase the clearance of the railroad traffic. In selecting a suitable structure for the place the following items will be briefly considered, viz.: (1) the cost and time of construction, the (2) delay to traffic; both freight and street; and (3) its appearance from an aesthetic point of view.

First, the cost of construction. There are numerous types of structures that might be considered. The span of the viaduct is approximately six hundred and seventy feet in length.

(a) A THRU OVERHEAD BRACED STRUCTURE.

(1) In considering a structure of this type there would have to be at least three spans. For example then, consider the span length two hundred and twenty feet long. The ordinary type of floor beams extending from panel point to panel point could not be used on account of the clearance necessary. The only floor system that could be used will be a shallow one, consisting of a
trough system, or I-beam spaced closely on centers. A girder of the thru plate type would be necessary to attach either one of these systems to, the girders being supported at the panel points. Summing up this type of construction then, means that there is a system of thru plate girders superimposed upon an overhead braced structure. It is obvious that this type of construction would be very heavy and consequently costly. A simple thru plate girder could be used to better advantage.

The time of construction of such a structure would be longer than that taken to construct a simple thru girder type. As the time element is one of cost as regards construction and inconvenience and congestion as regards street traffic, a type which would require less time to construct would obviously be cheaper.

(2) The construction of such a type as has been previously considered would require a substructure. A specially designed substructure would have to be designed in order to reduce the delay to freight traffic to a minimum. Even at its best, freight traffic would be considerably delayed, a fact which when considered alone would lead one to consider a type more feasible of construction.
(3) The appearance of such a structure would not be pleasing in such a place. It would look cumbersome and awkward.

B. A DECK PLATE SYSTEM.

(1) To acquire sufficient clearance the depth of girders would have to be small. This means that they will have to be spaced closely together on centers; thus requiring quite a number of girders to a span. Considering the depth, the spans could not economically be made much more than thirty feet. That means about twenty three spans. As the depth varies about one tenth of the span length it will be approximately three feet. Assuming that the minimum spacing of girders to be about six feet, in a roadway of sixty six feet, twelve girders would be necessary or a total of about two hundred and seventy-six girders. The total weight of all this steel would be considerable and costly. Here again a thru plate girder would be more economical.

The time of construction would be less than that required for the type considered under "A" but the cost factor would discard it in view of the fact that another type would be more economical.
(2) The construction of this traffic, especially freight would be slight, because it could be built from the street grade by means of a movable hoist.

(3) It would be a better appearing structure than the former and approaches more closely an open structure, a fact which is being clearly kept in mind in the selection of a suitable structure.

C. A REINFORCED CONCRETE STRUCTURE.

(1) While a reinforced structure would be cheaper than either of the two preceding types there are other things to be considered. This structure would be better and more lasting, the cost of up-keep being reduced to a minimum.

The time of construction would be about the same as that required for the previous ones. This is so, because it could only be built in sections, the reason for which is stated in the following paragraph which treats on the delay to traffic.

(2) The one serious factor against the selection of this type is the delay to freight traffic. A substructure must necessarily be built to construct this type and said structure would not delaying traffic at point of construction until the concrete had thoroughly set. At points on construction the substructure
would tie up the traffic completely and so compromise
the rearrangement of the traffic on the free tracks
that an over amount of congestion would occur.

(3) The appearance of such a structure would be
very pleasing, both at close and long range. It
would be a type that would beautify the place and make
traveling light and easy. Outside of the delay to
freight traffic there is no objection to the choice
of such a structure.

In the construction of these classes the delay of
the street traffic has not been considered very care-
fully, because it was assumed that the majority of it
would be transferred to Desplaines Street at Kinzie
Street.

D. A THRU PLATE GIRDER STRUCTURE COMBINED WITH
REINFORCED CONCRETE.

(1) Here thirteen spans were considered, ten
consisting of thru plate girders and three of rein-
forced concrete. Six of the ten girder spans were
assumed to be fifty-two feet long, and the two remain-
in, sixty feet in length. Three girders were assumed
to each span, making a total of thirty girders. The
floor system was made shallow by the use of I-beams spaced
closely on centers. This type of construction was found to be most economical due to the lesser weight of steel and building materials. The floor system of the concrete spans at the north end was made shallow by spacing the girders five feet on centers and using cross beams five feet centers making a slab five feet square.

The time of construction would be reduced to a minimum because of lesser steel construction in the field. This is so because the construction could be started from both ends and work toward the center, thus cutting the time approximately by half.

(2) The only delay to the railroad traffic would be at the northern terminal where the concrete spans are to be located. This would only stop the traffic on three tracks at the most and this could easily be adjusted for the time being. The girder spans would be constructed from overhead, no substructure therefore being necessary. So as far as delayed traffic is concerned this type of construction would be the most feasible as compared with the previous types.

(3) While the appearance of the steel girders is not exactly prepossessing, a means of obviating
this was in the assumption of a concrete veneer, around each one. This on the whole makes a better appearing structure than the concrete structure would, because the girders would be of less depth.

Therefore, the most suitable type of construction is the last named and the following design will include this type of structure.

PART II.

Design and specifications for a combined thru plate girder and reinforced structure.

The reason for making the northern terminal of reinforced concrete is because the girders cannot be carried to the abutment at said point. The effect of such, i.e. bringing them to the north abutment would be to cut off the approach to Desplaines Street, this fact making that construction impossible.

A comparison of the present system and the proposed system will now be set forth. The present system consists of a ninety foot thru overhead braced structure at the south, then two thru Pony trusses, and a series of deck plate girders, the spans of which vary from fifty to sixty feet. The total is approximately six hundred and sixty feet. The arrangement of
of the proposed system which has been hinted at is in detail as follows. Beginning at the south abutment, there will be two fifty-two foot girders, then two sixty foot girders, the location of Pier #3 being unchanged, then six fifty-two foot spans of girders and finally three concrete spans at the north end, each forty-three feet and eight inches in length.

The total width of the viaduct will be made sixty-nine feet and six inches. Throughout the girder spans, the street and walk will be supported by three girders, one situated between the car tracks and the remaining two between the side walk and street.

On this account two girders will have to be designed for each of the two spans. This is so because the central girder takes a heavier load due to the moving load of the cars than do the side girders and consequently have to be of heavier section. The girders were assumed as nearly uniform in length as possible so as to reduce the cost of shop construction. The floor system which was designed shallow consists of 15"I beams spaced two feet on centers. This part of the structure was made solid and rigid by placing concrete filler between I beams, the surface of which
was made flush with the tops of the I beams. The car rails the depth of which is seven inches are attached directly to the I beams. Upon the remaining surface of the street a three inch sand cushion was figured and firmly tamped. The surface of the street was figured up to grade by using creosote blocks four inches deep.

The width of each roadway is twenty seven feet and each sidewalk six feet clear space. The walk is supported on cantilevers spaced six feet on centers. The walk proper consists of a five inch reinforced slab.

The stresses in the girder were obtained as follows. A special loading shown in plate #1 was used to figure the live load moments and shears for the sixty foot girder. The bending moment and shears were obtained for the same girder from Cooper's E-40 and E-50 loading and ratios were made between the respective moments and shears respectively, i.e. Cooper's two loadings were compared with the special loading used. The moments and shears in the two remaining spans were obtained by multiplying the maximum moments and shears as given in Cooper's loadings by the constant ratios
derived. For the remaining roadway the live loading was assumed to be one hundred pounds per square foot. The equivalent uniform loadings shown on plate II were obtained from the following formulae.

\[
M = \frac{wl^2}{8}
\]

\( M \) maximum live load moments
\( l \) span length in feet
\( w \) weight per lineal foot.

The dead load per lineal foot included the following items: the weight of the creosote block, the sand; the rails; the floor beams; the concrete filler; the cantilevers; the weight of the girders and the weight of the concrete veneer. The dead weight of the girders was figured from the formulae:

\[
w = ab
\]

\( w \) weight per lineal foot
\( a \) a constant 12.5
\( L \) span length in feet
\( b \) a constant 200

Twice this value or \( 2w \) was taken as the weight per lineal foot of the central truss because it supports part of each street and is therefore of heavier cross section. The live and dead loads assumed for the walk were taken as one hundred and seventy five
pounds respectively.

The girders shall be braced laterally against the wind pressures and be supported on columns, the bracing of which shall be at least four times the required section. The columns shall be designed to take the total end shear transmitted from the adjacent girders.

The American Railway Engineering and Maintenance of Way Association specifications, second edition, 1910, governed the design of the steel work and for the reinforced concrete work Turneaure and Maurer's "Principles of Reinforced Concrete" was used.
Loading Used by the City of Chicago Bridge Department

Street Car
35 Tons

Sprinkler
42 Tons

Steam Roller
17 Tons

Heavy Wagons
24 Tons

Specifications: City of Chicago Bridge Department for Single Track
1-35 Ton Street Car, followed by 1-42 Ton Sprinkler Car followed by
1-17 Ton Steam Roller followed by as many 24 Ton Wagons as can be
Placed on Remaining Span.

Scale 1" = 15'-0"
<table>
<thead>
<tr>
<th>NOTATION</th>
<th>60'-0' SPAN</th>
<th>52'-0' SPAN</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>LIVE LOAD MOMENTS AND SHEARS</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>CHIEF 40 BRIDGE LOADING</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Moment</td>
<td>844.4 kF</td>
<td>665.0 kF</td>
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<tr>
<td>End Shear</td>
<td>79.7 k</td>
<td>12.9 k</td>
</tr>
<tr>
<td>1/4 Point Shear</td>
<td>49.2 k</td>
<td>44.75 k</td>
</tr>
<tr>
<td>1/2 Point Shear</td>
<td>22.7 k</td>
<td>20.5 k</td>
</tr>
<tr>
<td><strong>COOPER'S E-50</strong></td>
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<td></td>
</tr>
<tr>
<td>Maximum Moment</td>
<td>3245.0 kF</td>
<td>2540.0 kF</td>
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<tr>
<td>End Shear</td>
<td>244.0 k</td>
<td>222.7 k</td>
</tr>
<tr>
<td>1/4 Point Shear</td>
<td>150.4 k</td>
<td>137.0 k</td>
</tr>
<tr>
<td>1/2 Point Shear</td>
<td>69.4 k</td>
<td>62.6 k</td>
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<tr>
<td><strong>COOPER'S E-40</strong></td>
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<td></td>
</tr>
<tr>
<td>Maximum Moment</td>
<td>2540.0 kF</td>
<td>2030.0 kF</td>
</tr>
<tr>
<td>End Shear</td>
<td>195.2 k</td>
<td>178.5 k</td>
</tr>
<tr>
<td>1/4 Point Shear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1/2 Point Shear</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>EQUIVALENT LIVE LOAD FOR 60'-0' SPAN</strong></td>
<td>1875</td>
<td>1963</td>
</tr>
<tr>
<td><strong>EQUVALENT LIVE LOAD PER FOOT</strong></td>
<td>0.26</td>
<td>0.327</td>
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<tr>
<td><strong>E-40 E-50</strong></td>
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<td></td>
</tr>
<tr>
<td>Maximum Moment</td>
<td>0.325</td>
<td>0.407</td>
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**DATA**

<table>
<thead>
<tr>
<th>Width of Bridge</th>
<th>Walk Width</th>
<th>Street Width</th>
<th>Width Outside Track</th>
<th>Lbs/ft. on Street</th>
<th>Lbs/ft. on Walk</th>
<th>Load on Street</th>
<th>Load on Walk</th>
<th>Weight to Central</th>
<th>Weight to Side</th>
<th>Span Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>69'6&quot;</td>
<td>6'0&quot;</td>
<td>27'0&quot;</td>
<td>22'0&quot;</td>
<td>100</td>
<td>100</td>
<td>132 k</td>
<td>36 k</td>
<td>80365</td>
<td>27465</td>
<td>60'0&quot;</td>
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<tr>
<td>69'6&quot;</td>
<td>6'0&quot;</td>
<td>27'0&quot;</td>
<td>22'0&quot;</td>
<td>100</td>
<td>100</td>
<td>110 k</td>
<td>30 k</td>
<td>80365</td>
<td>27465</td>
<td>52'0&quot;</td>
</tr>
</tbody>
</table>

**Typical Cross-section of Street.**

**Scale: 1" = 12'-0"**

<table>
<thead>
<tr>
<th>Spans</th>
<th>Central Girder</th>
<th>Side Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total Live Load</td>
<td>Total Live Load</td>
</tr>
<tr>
<td></td>
<td>Moment</td>
<td>Shear</td>
</tr>
<tr>
<td></td>
<td>1/8 ft. Shear</td>
<td>1/8 ft. Shear</td>
</tr>
<tr>
<td>60'-0&quot;</td>
<td>20368 kN</td>
<td>172.15 kN</td>
</tr>
<tr>
<td>52'-0&quot;</td>
<td>15788 kN</td>
<td>155.2 kN</td>
</tr>
</tbody>
</table>
Weights of street materials, loading and clearances.

Creosote Block (depth) 4"
Sand Cushion ("") 3"
I-beam-floor beam ("") 15"
Girder Flange ("") 8"

Total ..... 30" = 2'6"

The rails are attached directly to the floorbeams.
The concrete filler is flush with the top flange of the floorbeam.

Weight of sand/cubic foot = 118 Lbs.
Weight of creosote block = 12 x 4.5 + 10 (for treatment) = 64 Lbs.

Dead loading on central truss.
Concrete filler = 27 x 12.5 = 3380 Lbs.

Weight of rails (to center) = 75 x 18.6 x 75 = 232" 27
Weight of sand = 27 x 0.25 x 118 = 800"
Weight of creosote block = 27 x 0.33 x 64 = 575"

Assumed weight of concrete veneer = 6 x 150 = 900" Total = 5882"

M = W1² = 5882 x 3600 = 2,640,000 ft. lbs.

End shear = 60 x 5880 = 176,000 Lbs.
1/4 point shear = 176,000 = 68,000 Lbs.
1/2 point shear < 0000.00

Dead load per foot of girder figured from formulae
W = AL + B.

W = weight per foot of girder
A = 12.5 (constant)
B = 200 ("")

\[ \text{Deflection per foot of girder} = \left( \frac{1}{18} \right) \times \text{D} \]

\[ \text{Deflection at girder} = \left( \frac{1}{18} \right) \times 27'0" \]

\[ \text{Deflection at track 15} = \left( \frac{1}{18} \right) \times 23'6" \]

\[ \text{Deflection at track 15} = \left( \frac{1}{18} \right) \times 18'6" \]

\[ \text{Deflection at track 15} = \left( \frac{1}{18} \right) \times 27'0" \]
Floor Beam Computations.

Total L.L. Diagram for 21'-0"

Centre of Gravity: \[ \frac{3750 \times 17.75 + 937.5 \times 12 + 175 \times 17.75}{5800} = 13.45' \]

\[ R = \frac{78052}{27} = 2890' \]

\[ M = 2890 \times 13.56 - 3750 \times 4.5 = 25,090 \text{ ft}^2 \]

Assume floor beams 2'-0" on 4

This will increase moment by a multiple of two.

\[ M_{LL} = 2 \times 25090 = 46,180 \text{ ft}^2 \]

Assume Concrete area = 12"

Concrete weight per lineal foot = \( 1 \times 2 \times 0.832 \times 150 = 250' \)

Moment: \[ \frac{Wl^2}{8} \]

\[ W = 250' \]

\[ l = 27'0" \]

\[ M = \frac{250 \times 27 \times 27}{8} = 27400 \text{ ft}^2 \]

Assume Impact = 100% of Live Load Moment

Total Moment:

L.L. Moment = 46180

Impact = 46180

Dead Load = \[ \frac{27400}{119760} \text{ ft}^2 \]

A 15" I Beam @ 75' will satisfy.
DESIGN OF SIDEWALK CANTILEVER

Design of Web

Sidewalk To Rest On Cantilevers Spaced 6'-0" On Centre. Sidewalk 6'-0" Wide.

Moment = \( \frac{Wl}{2} + Pl \)

- \( W = 6300 \) lb
- \( l = 6'-0" \) P = 5400 lb
- \( M = \frac{6300 \times 6}{2} + 3400 \times 5.25 = 425 \) kip ft

\[ M = \frac{SI}{c} \]

- \( S = 5 \times 16000 \)
- \( I = bd^2 \)
- \( b = \pi \)
- \( d = \text{Average Depth} \)

\[ d = \frac{6 \times 423,000 \times 12}{16000 \times 900} = 2.12" \] Total Thickness of Flange

Shear = \( 6300 + 5400 = 12700 \) ft-

Area Req'd = \( \frac{12700}{10000} = 1.27" \)

Thick of Web Req'd = \( \frac{1.27}{30} = 0.0423" \)

Use Web 8" Thick.

Bending Along X-Axis = 452,000 ft-

Stress = \( \frac{452,000 \times 12}{18} = 301,000 \) ft-

Area Req'd = \( \frac{301,000}{18,000} = 16.8\) "

Use 2-6" x 6" Flanges

Connection Angles 2-6" x 6" Rivets - 7 - Field

Scale: 3" = 1'-0"

Note: Gage Lines 24"
### Stress Sheet for Plate Girders

#### 60'-0" Central Girder

<table>
<thead>
<tr>
<th>Notation</th>
<th>LL Track</th>
<th>Tr Impact</th>
<th>LL Uniform</th>
<th>D.L Truss</th>
<th>D.L Street</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moments</td>
<td>1313.8</td>
<td>1120.0</td>
<td>725.0</td>
<td>857.0</td>
<td>3080.0</td>
<td>7093.8</td>
</tr>
<tr>
<td>End Shear</td>
<td>124.0</td>
<td>106.0</td>
<td>48.5</td>
<td>57.0</td>
<td>205.4</td>
<td>540.9</td>
</tr>
<tr>
<td>Shear 1/ft</td>
<td>76.6</td>
<td>65.5</td>
<td>24.1</td>
<td>28.5</td>
<td>102.7</td>
<td>397.4</td>
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<tr>
<td>Shear 2/ft</td>
<td>35.0</td>
<td>35.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>70.0</td>
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#### 60'-0" Side Girder

<table>
<thead>
<tr>
<th>Notation</th>
<th>LL Track</th>
<th>Tr Impact</th>
<th>LL Uniform</th>
<th>D.L Truss</th>
<th>D.L Street Sidewalk</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moments</td>
<td>187.6</td>
<td>160.0</td>
<td>835.0</td>
<td>423.5</td>
<td>850</td>
<td>3986.0</td>
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<tr>
<td>End Shear</td>
<td>17.7</td>
<td>15.1</td>
<td>55.5</td>
<td>28.5</td>
<td>159.1</td>
<td>275.9</td>
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<tr>
<td>Shear 1/ft</td>
<td>10.9</td>
<td>10.9</td>
<td>21.7</td>
<td>14.2</td>
<td>79.5</td>
<td>143.5</td>
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<td>Shear 2/ft</td>
<td>5.1</td>
<td>5.1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>10.2</td>
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#### 52'-0" Central Girder

<table>
<thead>
<tr>
<th>Notation</th>
<th>LL Track</th>
<th>Tr Impact</th>
<th>LL Uniform</th>
<th>D.L Truss</th>
<th>D.L Street</th>
<th>Total</th>
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<tbody>
<tr>
<td>Moments</td>
<td>1034.0</td>
<td>825.0</td>
<td>544.0</td>
<td>568.0</td>
<td>231.3</td>
<td>5284.0</td>
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<tr>
<td>End Shear</td>
<td>113.4</td>
<td>96.5</td>
<td>41.8</td>
<td>45.6</td>
<td>178.5</td>
<td>475.6</td>
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<tr>
<td>Shear 1/ft</td>
<td>69.64</td>
<td>59.4</td>
<td>20.9</td>
<td>21.8</td>
<td>89.2</td>
<td>261.0</td>
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<tr>
<td>Shear 2/ft</td>
<td>31.9</td>
<td>31.9</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>63.8</td>
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#### 52'-0" Side Girder

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<thead>
<tr>
<th>Notation</th>
<th>LL Track</th>
<th>Tr Impact</th>
<th>LL Uniform</th>
<th>D.L Truss</th>
<th>D.L Street Sidewalk</th>
<th>Total</th>
</tr>
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<tbody>
<tr>
<td>Moments</td>
<td>142.6</td>
<td>127.0</td>
<td>635.0</td>
<td>284.0</td>
<td>1890.5</td>
<td>2682.1</td>
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<tr>
<td>End Shear</td>
<td>16.2</td>
<td>13.8</td>
<td>46.0</td>
<td>21.8</td>
<td>144.1</td>
<td>241.9</td>
</tr>
<tr>
<td>Shear 1/ft</td>
<td>9.93</td>
<td>8.46</td>
<td>25.0</td>
<td>10.9</td>
<td>72.0</td>
<td>123.75</td>
</tr>
<tr>
<td>Shear 2/ft</td>
<td>4.55</td>
<td>4.55</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>9.10</td>
</tr>
</tbody>
</table>

Note:

- Bending Moments are in Kip-feet.
- Shears are in Kips.
### Design of 60°0' Central Girder

**Approximate Flange Stress**

\[
\frac{65,000,000}{72} = 1180,000\text{#}
\]

**Approximate Area**

\[
\frac{1180,000}{16000} = 74\text{"}
\]

**Web Area**

\[
\frac{540,400}{10000} = 54\text{"}
\]

**Thickness**

\[
\frac{54}{72} = 0.75\text{" Use Web 72}\times\frac{3}{16}
\]

**Flange Section**

<table>
<thead>
<tr>
<th>2 Angles 8\times\frac{1}{8}&quot;</th>
<th>Gross Area</th>
<th>Net Area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>33.48\d&quot;</td>
<td>28.96\d&quot;</td>
</tr>
</tbody>
</table>

| 1/8 Web Area                  | 6.75\"     | 6.75\"   |
| 2 Plates 18\times\frac{1}{2}\"| 18.00\"    | 16.00\"  |
| 2 Cover Plates 18\times\frac{3}{4}\"| 27.00\"    | 24.00\"  |
|                               | 9.00\"     | 8.00\"   |

**Total**

94.23\d" 83.71\d"

**Center of Gravity**

- Top cover plate: -9 x 9.75 = 178
- Angles: 33.48 x 15.59: -522.2nd
- 2 Side Plates: 18\times162.3rd
- C. of G.: 1369.25: 14.52

**Effective**

94.23 depth: 72-2\times5.48: 65.04

**Actual Stress**

\[
\frac{85,000,000}{1,305,000} = \frac{700,000}{16000}
\]

**60°0" Side Girder**

**Approximate Stress**

\[
\frac{47,800,000}{72} = 665,000\text{#}
\]

**Area**

\[
\frac{665,000}{16000} = 41.5\text{"}
\]

**Web Area**

\[
\frac{275900}{10000} = 27.6\text{"}
\]

**Thickness**

\[
\frac{27.6}{72} = 0.383\text{" Use Web 72}\times\frac{7}{16}
\]

**Flange Section**

<table>
<thead>
<tr>
<th>2 Angles 8\times\frac{7}{8}&quot;</th>
<th>Gross Area</th>
<th>Net Area</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>26.48\d&quot;</td>
<td>22.96\d&quot;</td>
</tr>
</tbody>
</table>

| 2 Cover Plates 18\times\frac{1}{2}\"| 18.00\"    | 16.00\"  |

| 1/8 Web Area                     | 4.00\"     | 4.00\"   |

**Total**

48.48\d" 42.96\d"

**Center of Gravity**

- Top cover plate: -9 x 8.75: 78.75
- Angles: 26.48 x 54.7: 145.2nd
- C. of G.: 298: 6.15

**Effective**

72-2\times1.85: 68.8"
Stiffeners in 60°0" Central Girder.

End Shear Transmitted to Abutment by Stiffeners,
\[
\frac{540,900}{14,420} = 38 \text{ Rivets; plus } 50\% = 57.
\]
Use 19 Rivets in 3 Pair of Stiffeners.

Stress per Pair = \[\frac{540,900}{180,300} \#\]
Assume Angles 6"x4\(\frac{3}{4}\)"
\[S = 16,000 - \frac{70 \times 69.75}{0.86} = 10,325 \#\]

Req'd. Area \[= \frac{180,300}{10,325} = 17.5 \text{" Actual Area } = 18 \text{"}\]
Minimum size of Intermediate stiffeners: \[72 + 2 = 44\"
Use Angles 5"x3\(\frac{3}{4}\)\"x\(\frac{3}{4}\)"

Spacing of stiffeners,
\[d = \frac{3}{40} (12000 - 5) \quad \text{(See Above)}\]

End Spacing = \[d = \frac{3}{160} (12000 - \frac{540,900}{54}) = 38\"

Intermediate Spacing = \[d = \frac{3}{160} (12000 - \frac{597,400}{34}) = 87\"

Spacing not to exceed 6\(\frac{1}{2}\)"

Stiffeners in 60°0" Side Girder.

Req'd. Rivets = \[\frac{275,900}{9188} = 30; \text{ plus } 50\% = 45\]
Use 15 Rivets in 3 Pair of Stiffeners.

Stress per Pair = \[\frac{275,900}{91,934} \#\]
Assume Angles 6"x4"x\(\frac{1}{2}\)"
\[S = 16,000 - \frac{70 \times 69.75}{0.87} = 10,385 \#\]

Req'd. Area = \[\frac{91,934}{10,385} = 8.85\" Actual Area = 9.5\"
Minimum size of Intermediate stiffeners: \[72 + 2 = 44\"
Use Angles 5"x3\(\frac{1}{2}\)"x\(\frac{1}{2}\)"

Spacing of stiffeners,
\[d = \frac{1}{40} (12000 - 5) \quad \text{(See Above)}\]

End Spacing = \[d = \frac{1}{640} (12000 - \frac{275,900}{31.5}) = 22\"

Intermediate Spacing = \[d = \frac{1}{640} (12000 - \frac{143,500}{31.5}) = 82\"
Web Splice for 60'-0" Central Girder.

Splice to occur at or near the 1/3 point. Moment resisted by web: 
\[ M = 6.75 \times 85,000,000 = 10,650,000 \]
\[ \frac{M}{A} = 0.75 \times 106,500,000 = 79,725,000 \text{# taken by straps. Assume 30 rivets.} \]

Stress per rivet: 
\[ S = \frac{79,725,000}{12,900} = 6,245.2 \text{# allowed} = 14,432 \text{#} \]

\[ \frac{1}{4} \text{ strap plates } 30 \times 10\frac{1}{2} \]

\[ 0.75 \times 106,500,000 = 26,57,500 \text{# taken by vertical plate. Assume 36 rivets} \]

\[ S = 16.5 \times 26,57,500 = 11,450 \]
\[ 6(63+63+63+63+63) = 875 \text{ result: } \sqrt{11560+5525} = 12,630 \text{#} \]

Stress at outer fibre: 
\[ I_{p} (plates) = 2 \times 125 \times \frac{3}{6} = 875 \]
\[ I_{R} (rivets) = 4 \times 125 + 125 (643) = 2601 \]

\[ 1.15 - 1.25 \]

Use 2 vertical splice plates. 36\times18\times1/4".

Web Splice for 60'-0" Side Girder.

Moment resisted by web: 
\[ M = \frac{47,800,000 \times 3.85 \times 6,675,000}{27.6} \]

Assume plate on 7/8 fillers and 132 rivets. Stress per rivet: 
\[ S = \frac{35 \times 6,675,000}{6(64+64+64+64+64)} = 9186 \text{#} \]

Total rivets: 132+30% = 198.

Use 2 plates 70\frac{1}{2} \times 18\frac{1}{2}".

2 fillers 36 \times 30\frac{3}{8}".

Stress due to shear: 
\[ I_{p} = 2 \times 0.5 \times 70.25 = 26,000 \]
\[ I_{R} = 4(11 \times 0.48) + 0.5 (4169.5) = 8304 \]
\[ 1.15 - 1.25 \]

\[ I_{p} - I_{R} = 20559.6 \]

\[ S = \frac{6675,000 \times 35.125}{20559.6} = 11,450 \text{#} \]
**Design of 52'-0" Central Girder**

**Moment:** 5,284,000 * 12 = 63,408,000"*

**Stress:** \( \frac{63,408,000}{72} = 880,000 \) *

**Area Req'd:** \( \frac{880,000}{16000} = 55\frac{y}{"} \)

**For Flanges**

**Section:** Effective

2 Cover Plates 18\"x\"\]
1 Cover Plate 18\"x\"\]
8 Web Area

2.5 8\"x8\" 11" **Gross Area** = 67.70"

**Net Area** = 60.20"

**Centre of Gravity of Section:**
\[ \frac{9 \times 18 + 9 \times 8 + 12 \times 8.75 + 30 \times 5.63}{67.7} = \frac{448.6}{67.7} = 6.62\"

8.62 = 138° from Top of Angle

**Effective Depth** = 72\" - 2.7\" = 69.34"

**Actual Stress:** \( \frac{63,408,000}{69.34} = 916,000 \) *

**Actual Area Needed:** \( \frac{916,000}{16000} = 57.20\"

**Section Safe**

**Web Section:** Shear = 473,000#  
Area Req'd = \( \frac{473,000}{16000} = 47.60\"

Use Web 72\"x1\" 11"

**Design of 52'-0" Side Girders**

**Moment:** 2,682,100\"* = 32,200,000\"*

**Stress:** \( \frac{32,200,000}{72} = 447,500 \) *

**Area Req'd:** \( \frac{447,500}{16000} = 27.95\"

**Section:** Effective

2 Cover Plates 18\"x\"
8 Web = 3.4"
2 Ls 8\"x8\"\]
4"

**Gross Area** = 44.28"

**Net Area** = 59.28"

**Centre of Gravity:**
\[ \frac{9 \times 8.75 + 9 \times 8.25 + 23.88 \times 5.72}{44.28} = 8.67\"

**Effective Depth** = 73.24"  
**Stress:** \( \frac{32,200,000}{73.34} = 440,000 \) *

**Actual Area** \( \frac{440,000}{16000} = 28.5" Section Safe**

**Web Section:** Shear = 241900#  
Area Req'd = \( \frac{241900}{16000} = 24.2"\]

Use 24.2 \( \frac{72}{72} = 0.366\"  
**Web Section** 72\"x5\"
**Stiffeners in 52' 0" Central Girder.**

Reqd. Rivets = \( \frac{473,600}{14432} = 34 \text{; plus } 50\% = 51 \)

Use 17 Rivets in 3 pair of stiffeners.

Stress per pair = \( \frac{473,600 \cdot 15,7867}{10325} \)

Assume angles \( 6" \times 4\frac{3}{8} \times \frac{7}{8} \).

\[ S = 16,000 - \frac{70 \times 69.75}{0.86} = 10,525 \]

\[ \text{Reg'd. Area} = \frac{15,7867}{10325} = 15.780" \text{ Actual area} = 17.880" \]

Minimum size of intermediate stiffeners = \( 72 \div 2 = 44" \)

Use angles \( 5" \times 3\frac{1}{2} \times \frac{3}{16} \).

Spacing of stiffeners.

\[ d = \frac{1}{40} (12,000 - 5) \]

End spacing = \( d = \frac{11}{640} (12,000 - 473600) = 35" \)

Intermediate spacing.

\[ d = \frac{11}{640} (12,000 - 261,000) = 11.5" \]

Spacing not to exceed 6' 0".

**Stiffeners in 52' 0" Side Girder.**

Reqd. Rivets = \( \frac{241,900}{7876} = 32 \text{; plus } 50\% = 48 \)

Use 16 Rivets in 3 pair of stiffeners.

Stress per pair = \( \frac{241,900 \cdot 80.634}{10385} \)

Assume angles \( 6" \times 4\frac{3}{2} \times \frac{3}{4} \).

\[ S = 16,000 - \frac{70 \times 69.75}{0.87} = 10,385 \]

\[ \text{Reg'd. Area} = \frac{80.634}{10385} = 7.750" \text{ Actual area} = 9.50" \]

Minimum size of intermediate stiffeners = \( 72 \div 30 = 44" \)

Use angles \( 5" \times 3\frac{1}{2} \times \frac{3}{16} \).

Spacing of stiffeners.

\[ d = \frac{1}{40} (12,000 - 6) \]

End spacing = \( d = \frac{3}{520} (12,000 - \frac{241900}{27}) = 19" \)

Intermediate spacing = \( d = \frac{3}{520} (12,000 - \frac{123730}{27}) = 70" \)
DESIGN OF SPLICE PLATES FOR 52'-0" CENTRAL AND SIDE GIRDERS.

CENTRAL GIRDER

MAX. SHEAR AT 1/2 PT. = 31,900 ft-
MAX. BENDING = 63,400,000 in-

BENDING MOM. TAKEN BY 6 WEB AS ABOVE
2 WEB x 63,400,000 = 61,620,000
FLANGE AREA
60.2 = 65,500,000

M = 500 C
S = 15 x 3 = 45

I = \( \frac{150 \times 50}{12} = 2 \times (49.1 + 1.5 + 36.4 + 6.4) \)

= 1,100 - 2,550 = 850,

\( \frac{850}{C} = 50,000 \times 28 \)

S = 50,000 X 1,100 = 56 X 50,000

SAFE AS MAX. STRESS ALLOWED AT THIS POINT = 14,200 ft-
2 SPICE PLATES 50% O.K.

FOR SHEAR STRESSES,

\( S = \frac{\text{No. rivets} \times \text{stress in each rivet}}{\text{stress in rivet}} \)

\( 6 \times 4 = 258 \)

\( F = (1443^2 + 850^2)^{1/2} = 1452 \)

\( S = 6 \times 4 \times 1452 = 7,600,000 \) ft-

AS STRESS RIVETS WILL CARRY LOAD ON RIVETS = 6,500,000 ft-

:: NO RIVETS SAFE

SIDE GIRDER.

MAX. SHEAR AT 1/2 PT. = 31,900 ft-
MAX. BENDING = 32,200,000 in-

BENDING MOM. TAKEN BY 6 WEB AS ABOVE
32,200,000 \times 3.28 = 2,780,000 in-

I = BY SAME METHOD AS ABOVE = 4763

S = 4763 \times 2 = 3000

SECTION SAFE 2-PLATES 56% O.K.

RIVET STRESSES,-

\( S = 4 \times 4 = 258 \)

\( F = 4 \times 4 \times 258 = 1452 \)

\( S = 4 \times 4 \times 258 = 5,200,000 \) ft-

ACTUAL LOAD = 2,780,000 ft-

:: NO RIVETS SAFE

S.C.A.
LATERAL BRACING FOR 60'-0" GIRDER.

Θ = 29° 4'  Sec. Θ = 1.144

Heaviest Load following C.B.L. = 48,000 / 10 = 4,800

Loading per foot = 200 + 200 + 480 = 880

880 x 15 = 13,200

Length of diagonals = \( \sqrt{27^2 + 15^2} = 34.875' = 34' 10\frac{1}{2}" \)

Stress in AB = \( \frac{1}{4} (1 + 2 + 3) \times \text{Sec.} \ Θ = \frac{13,200 \times 6 \times 114}{4} = 22,400 \)

Stress in BC = \( \frac{1}{4} (1 + 2) \times \text{Sec.} \ Θ = \frac{13,200 \times 3 \times 114}{4} = 11,800 \)

Reqd. area = \( \frac{22,400}{16,000} = 1.4 \) " Use 2 Angles 3 1/2" x 3 1/8".

" Rivets = \( \frac{22,400}{726} \) Use same angles for 52'-0"

LATERAL BRACING FOR 52'-0" GIRDER.
### Length of Cover Plates

#### 60'-0" Central Girder

<table>
<thead>
<tr>
<th>Plate</th>
<th>Cover 18&quot;3/4&quot;</th>
<th>Cover 18&quot;1/4&quot;</th>
<th>Cover 18&quot;1/2&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2&quot;</td>
<td>30'-0&quot;</td>
<td>20'-0&quot;</td>
<td>10'-0&quot;</td>
</tr>
</tbody>
</table>

#### 60'-0" Side Girder

<table>
<thead>
<tr>
<th>Angle</th>
<th>Cover 18&quot;1/2&quot;</th>
<th>Cover 18&quot;1/4&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>8\times8\times1/2&quot;</td>
<td>30'-0&quot;</td>
<td>15'-0&quot;</td>
</tr>
<tr>
<td>8\times8\times7/8&quot;</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### 52'-0" Central Girder

<table>
<thead>
<tr>
<th>Angle</th>
<th>Cover 18&quot;3/4&quot;</th>
<th>Cover 18&quot;1/2&quot;</th>
<th>Cover 18&quot;1/4&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>8\times8\times1&quot;</td>
<td>26'-0&quot;</td>
<td>18'-0&quot;</td>
<td>10'-0&quot;</td>
</tr>
<tr>
<td>8\times8\times1/2&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

#### 52'-0" Side Girder

<table>
<thead>
<tr>
<th>Angle</th>
<th>Cover 18&quot;3/4&quot;</th>
<th>Cover 18&quot;1/2&quot;</th>
<th>Cover 18&quot;1/4&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>8\times8\times11/4&quot;</td>
<td>26'-0&quot;</td>
<td>17'-0&quot;</td>
<td></td>
</tr>
<tr>
<td>8\times8\times13/16&quot;</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Scale:** 1" = 1'-0"
Columns for Central Girder.
The greatest reaction occurs at the junction of the 60°0' and 52°0' girders, and = 540,900 + 473,600 = 1,014,500.

Least radius of gyration occurs about the vertical axis: \( R = \sqrt{\frac{I}{A}} \)

Section: \( I = I_1 + I_2 \)

Area

1. Cover plate 14\(\times\)1\(\frac{\text{in}}{\text{in}}\) 229.0 14.0'
2. 7/4\(\times\)2\(\frac{\text{in}}{\text{in}}\) 34.0 0 6.78

Half Total 787.86 55.12

\( R = \sqrt{\frac{787.86}{55.12}} = 3.79 \)

Using a safety factor of 4, unit stress \( S = 11,825 \)

Reqd. area = \( \frac{1,014,500}{11,825} = 85.6 \) 0" Actual area: 110.24 0"

10 of these columns are req'd.

Columns for Side Girder.
Total load = 275,900 + 241,900 = 517,800.

A standard plate and channel column of the following section is necessary.

2 - 35\# channels.
2 - cover plates 20\(\frac{\text{in}}{\text{in}}\) \(\frac{1}{2}\).

Size of top and bottom plates on these columns; 22\(\frac{\text{in}}{\text{in}}\) \(\times\)20\(\frac{\text{in}}{\text{in}}\) 40 req'd.

Size of said plates on central columns are; 17\(\frac{\text{in}}{\text{in}}\) \(\times\)16\(\frac{\text{in}}{\text{in}}\) 20 req'd.

Connection angle for both sets; 6\(\times\)4\(\times\)\(\frac{1}{2}\) \(\times\)14" used on central set.
40 - 6\(\times\)4\(\times\)\(\frac{1}{2}\) \(\times\)10"
80 - 6\(\times\)4\(\times\)\(\frac{1}{2}\) \(\times\)20" Side
80 - 6\(\times\)4\(\times\)\(\frac{1}{2}\) \(\times\)12"
Typical Column Footings

20 reqd. for side set. 10 reqd. for cen. set.

<table>
<thead>
<tr>
<th>Used</th>
<th>Base Plate</th>
<th>HUB</th>
<th>Cap Plate</th>
<th>Ribs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Under</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
</tr>
<tr>
<td>CEN. GIRL</td>
<td>5&quot;</td>
<td>2&quot;</td>
<td>2.5&quot;</td>
<td>13&quot;</td>
</tr>
<tr>
<td>SIDE GIRL</td>
<td>5&quot;</td>
<td>2&quot;</td>
<td>2.5&quot;</td>
<td>13&quot;</td>
</tr>
</tbody>
</table>

Diagram for Column Bracing

$\theta = 33^\circ 50'$ Sec $\theta = 1.205$

Stress in diagonal = $\frac{26600 \times 32000}{16000}$

Reqd. area = $32000 = 2.0''

Use 1 angle $3\frac{1}{2}'' \times 13\frac{3}{8}'' = 40$ reed

Use $\frac{7}{8}$ rivets. Gusset plate

Reqd. no. = $\frac{32000}{6} = 6.175'' \times 15\frac{3}{8}''$

Shop connection; use 3. 240 reed.

480 connection angles reed. $10\frac{1}{2}'' \times 3\frac{1}{2}'' \times 3\frac{3}{8}''$
Assume Live Load = 100 #/ft.
" Dead " = 75 #/ft.

From Turnaire and Maurer, Table 21 4
5" slab will be necessary.

Reinforcement consists of 1/2" rods 8" o.c.
used both ways. The slab is hung on steel cantilevers 6'-0" o.c.
TYPICAL CROSS SECTION
— VIADUCT —

The drawing gives general dimensions only for a more complete design of parts see overall drawing.

Note: Concrete Veneering on Reinforced To Be Worked into Area In Body, Not Cut as Shown.

Reinforcement: See Overall Drawing for Details.

ARMOUR INSTITUTE OF TECHNOLOGY
ARCHITECTURAL DEPARTMENT
The Reconstruction of the Michigan Ave Viaduct — Chicago Il.
TYPICAL CROSS SECTION OF VIADUCT
DRAFTED BY: [Signature]
Design of Slabs for Concrete Section North End.

5'-0" on $s$: Slab = $5'-0" \times 5'-0"

Live Load on Slab = 5' x 6' x 100 = 3000$

Assume Dead Load = 75 #/ft', Live Load = 100 #/ft'

Impact = 95 #/ft', Total Load = 271 #/ft'

From T&M Page 297 Table 6: $f_s = 15000$, $f_c = 000$

Slab = 3' Thick, Weight = 42.7 #/ft', % Steel = 0.2470 ft'

Use 1" Rods - 3/4" on center lines for middle half and 5" on center line toward supports

Scale 1" = 1'-0"
Design of Beams for Concrete
Section North End.

- 2 - 1/4" Rods - 6'-0"
  Turned at 60° for Stirrup

Scale: 1" = 1'-0"

Stirrup Diagram

Each Beam Takes One Panel Load - One 1/4" Panel Load Acting through Third Points as per Loading Diagram

Total Slab Load:
\[ DL \text{ of Slab} = 427 \times 25 = 11700 \]
\[ LL \text{ of Slab} = 100 \times 25 = 2600 \]
\[ \text{Impact} = \frac{2460}{2} = 1230 \text{ kips} \]

Total = 6130 kips

\[ R = \frac{6130}{2} = 3065 \text{ kips} \]

\[ M = 3065 \times 1.0 = 3065 \text{ kips-feet} \]

Assume: \( b = 6 \) in, \( d = 18 \) in, \( f = 10 \) in, \( t = 3.5 \) in
\[
A = \frac{3}{8} (d-f) = \frac{3}{8} (18-10) = 3.4375 \text{ sq in}
\]

Use: 2 - \( \frac{3}{8} \)" Rods at 2'-0" on Center

Shear: \( \frac{3065}{6.1} = 501 \text{ kips} \)

From Stirrup Diagram

\[ P = 2 \times 0.5 \cos 60° = 0.5 \text{ kips} \]
\[ P = 0.5 \times \frac{3065}{10} \times 12 = 1840 \text{ kips} \]

Area Needed: \( \frac{1840}{3065} = 0.60 \text{ in}^2 \)

Use: 1 - 1/4" Rod 6'-0"

S.C.A.
Design of Girders for Concrete Section North End

Beam Reaction at O and Girder Reaction at E

Track Load: End Shear for 32'-0" Girder (E-40) = 128,800 \#/track. Total per track. For Chgo. Bridge Load $\frac{128,800}{2} \times 0.325$ (see Plate 2) = 20,900\# \text{ Impact} = 18,950\#

End Shear due to Beam Loads = 7 x 3065 = 21,455\#
End Shear due to Slabs = 6 x 3065 = 18,390\#
End Shear due to Weight of Slabs $\frac{8.5 \times 6 \times 150 \times 5}{144}$ = 80,491\#

Total = 80,491\# \text{ End Reaction.}

Moment at Center from Loading as per diagram = 12,114,600 \#.

Assume $b = 72''$ \text{ $d = 24$} \text{ $P = 3\%$}

Plate 9 - T8, M - Page 203 \text{ $K = 0.325$} \text{ $J = 0.95$}

For dimensions of beam see Plate 26

S.C.A
Design of Girders Continued
From Plate 24

\[ A = \frac{M}{f_s \times Jd} = \frac{12,140,000}{15,000 \times 0.95 \times 24} = 35.40'' \]
\[ f_s = h \left( \frac{1-K}{k} \right) f_{c'c} \quad \Rightarrow \quad f_{c'} = \frac{f_s \times 15}{h \left( \frac{1-K}{k} \right)} = \frac{35.40 \times 15}{0.325} = 481 \]
\[ f_s' = h \left( \frac{K-d}{k} \right) f_{c'c} = 15 \times \frac{0.325 - 0.224}{0.325} = 3500 \]
\[ \frac{f_s - f_s'}{f_s} = 1 - \frac{f_s'}{f_s} = 1 - \frac{3500}{3500} = 0.87 \]
\[ : 37\% \text{ REDUCTION NECESSARY.} \]

Area of Compressive Steel = \( 0.37 \times 35.4 = 13.5'' \)
Area of Tensile Steel = 35.4 - 13.5 = 21.9''

Compressive Steel = 8 - 0.14'' Ø Rods - 2'' on C.
Tensile Steel = 14 - 0.14'' Ø Rods 2'' on C.
Spaced 7 in Upper Row - 7 Lower Row.

Shear:

\[ V = \frac{80 + 81}{24 \times 24} = 139 \text{ lb} \]

See Stirrup Diagram (28) For Following

Data on Stirrups
80 = \( \frac{X}{11.2} \)
X = 2.6''
P = 0.7 \times \frac{80}{24} = 0.7 \times 3.33 \times 0.321
P = 0.753'' Area Required = \( \frac{0.753}{1500} = 0.50'' \)

Turn Up Center Rod Top Group of Tensile
5'' 0.3'' From C at 45°
P = 0.7 \times \frac{80}{24} \times 4 = 9575'' Area = \( \frac{9575}{1500} = 0.638'' \)

Turn Up 2 Rods of Top Row of Tensile Steel Adjoining Middle 8.2'' From C of Girder.
P = 0.7 \times \frac{80}{24} \times 5 = 11700'' Area = \( \frac{11700}{1500} = 0.780'' \)

Turn Up 2 Outside Top Row of Tensile Steel 15.4'' From C.
LOADING DIAGRAM SIDE GIRDERS

DEAD LOAD: \( \frac{304 \times 31.54 \times 42.7}{2} = 2030 \) kips

LIVE LOAD: \( \frac{304 \times 31.54 \times 100}{2} = 4760 \) kips

GIRDER WEIGHT: \( \frac{304 \times 6.1 \times 1150}{144} = 2410 \) kips

BEAM WEIGHT: \( \frac{362 \times 150 \times 8.5 \times 9}{144} = 1870 \) kips

TOTAL: 11,070 kips

\( R = \frac{117 \times 25.675}{43.675} = 6500 \) kips

\( M = 117,000 \times 12 = 1,405,000 \) in-kips

\( A = \frac{6M}{7.50} = \frac{8 \times 1405000}{7.50} = 445 \) in

USE 5-18' Ø RODS 2' ON 4

BEAM SECTION — 26'2" x 14" —

THEORETICAL

\( \nu = \frac{6500}{14 \times 24} = 19.6 \). NO STIRRUPS NEEDED
GIRDERS BETWEEN COLUMNS

**LOAD AT EACH POINT = 80491#**

\[ P = 80491 + 49 + 45 + 195 = 169008 \]

\[ C = 80491 X 47 = 3,751,950 \]

\[ M = 169000 X 11.75 = 80491 X (2.875 + 6) \]

\[ M = 16250000 \text{ in}^2 \]

\[ A = \frac{16250000}{11.75} = 20,650 \text{ in}^2 \]

**USE 12-1/2" RODS 6" ON C.**

**BEAM = 69" x 24"**

\[ v = \frac{V}{69000} = \frac{78.25}{60132} \]

**STIRRUPS**

FOR DETAILS SEE PLATE 20

\[ 80 - \frac{x}{78.25} = \frac{7.83}{X} = 5 \]

\[ P = 0.7 \sqrt{15} = 0.7 \times 169000 \times 24 = 47,300 \text{#} \]

\[ H = \frac{47300}{80} = 3,162 \text{ in}^2 \]

\[ P' = 0.7 \times 169000 \times 58.25 = 65600 \text{#} \]

\[ A = \frac{65600}{15000} = 4.38 \text{ in}^2 \]

**FIRST SET OF STIRRUPS TURNED UP 5-3/8"**

FROM CENTRE - USE 2 OUTSIDE TENSILE STEEL BARS IN LOWEST ROW

**SECOND SET - USE 2 MIDDLE TENSILE STEEL BARS MIDDLE ROW TURNED UP 7-1/2" FROM CENTER.**

**THIRD SET - USE 2 OUTSIDE TENSILE STEEL BARS IN TOP Row - TURN UP AT 9-1/8" FROM THE CENTER.**
STIRRUP DIAGRAM FOR COLUMN

FOR CALCULATIONS SEE PLATE 28

SCALE 3'/1-0'

S.C.A
**Column Design for Concrete Section North End.**

Assume $f_c = 400$  $f_s = 15f_c$  $P = 8\%$

$$f_s = \frac{15 \cdot 400}{P} = 6000$$

$$R = f_c \left(1 + \frac{P-1}{P}\right) = \frac{338000}{400 \cdot (1+1/2)} = 4000$$

358000 = Load Supported by a Central Column

$$A_s = 0.08 \cdot 400 = 32$$

$$A_c = \frac{P - A_s}{f_c} = \frac{338000 - 6000 \cdot 32}{400}$$

$$A_c = 3650 \text{ needed, use } 5760$$

$$P_c = \frac{338000}{1+(P-1)} = 153500$$

**Column Section:**

Concrete = 24" x 24"

Reinforcing = 4 LS - 3\% 5/16

Tied by Plates 24\% 3" - 1/2"

Spaced 2'-0" on Center Line.
Stirrups Design for Girder Columns - Foundation

For calculations for stirrups see Plate 51.
Concrete Column Foundations

Load thru columns: \( P = 338000 \) ft-

Bearing of foundation strata: 14,400 sq ft

Area rod: \( \frac{338000}{14400} = 23.0 \) sq ft

Base: 5'-0" x 5'-0"

Center of gravity of trap ABCD: \( \frac{15}{3} \times \frac{2}{2+5} = 0.645 \)

Material: \( 169000 \times 121.85 = 1720,000 \) in-

Steel necessary:

Area rods: \( \frac{1720000 \times 8}{7 \times 15000 \times 60} = 2.72 \) in-

Use 1" rods 4'-2" on 8" - 4'-6" long.

Shear:

\( V = \frac{139000}{60} = 47 \) ft-

Allowed = 30 ft-

Turn up every fourth rod 60° - 1'-0" from center.
RAIL FASTENING IN CONCRETE

5" PIPES PLACED IN CONCRETE 10" LONG - 2" ON CENTER.
1" BURNT BOART SUNK INTO PIPES AND BROUGHT TO GRADE. FILL PIPES WITH NEAT CEMENT AND ADJUST TO CONFORM WITH TRACK ELEVATION.

SCALE 1/2" = 1'-0"