THE DESIGN

and

ECONOMICAL COMPARISON

of

REINFORCED CONCRETE and ENCASED STEEL

HIGHWAY BRIDGES

A Thesis

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William Labry Brown

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of the

ADVANCED DEGREE COMMITTEE

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TABLE of CONTENTS

Part I  -  DESIGN of SLAB-BEAM-GIRDER-COLUMN-BRIDGE  
(Reinforced Steel Type)

1. General Data
2. Design of Floor Slab
3. Design of Floor Beams
4. Design of Interior Beams
5. Design of Exterior Beams
6. Design of Girder
7. Design of Exterior Column
8. Design of Interior Column
9. Design of Exterior Column Footings
10. Design of Interior Column Footings

Part II  -  DESIGN of SLAB-BEAM-GIRDER-COLUMN BRIDGE  
(Encased Steel Type)

1. General Data
2. Design of Floor Slab
3. Design of Interior Floor Beams
4. Design of Exterior Floor Beams
5. Design of Girder
6. Design of Exterior Columns
7. Design of Interior Columns
8. Design of Exterior Column Footings
9. Design of Interior Column Footings

Part III  -  ECONOMICAL COMPARISON of REINFORCED CONCRETE TYPE  
WITH ENCASED STEEL TYPE

1. General
2. Assumptions
3. Quantities
4. Unit Cost Analysis
5. Total Cost of Bridges
6. Conclusions
Part I

DESIGN of SLAB-BEAM-GIRDER-COLUMN BRIDGE

Reinforced Steel Type

I. General Data

This bridge is designed for a 20 ton truck load having 80% of load on rear axle and 20% on front; each front wheel is a load of 4 kips, and each rear wheel a load of 16 kips. As this structure is of reinforced concrete the following allowable stresses have been assumed:

\[ f_c = 800\# \]
\[ f_s = 18,000\# \]
\[ n = 15 \]
\[ u = 100\# \]
\[ v = 60\# \text{ without web reinforcement} \]
\[ v = 240\# \text{ with special anchorage} \]
\[ K = 139 \text{ at center} \]
\[ K = 165 \text{ over supports} \]
\[ j = 7/8 \]

II. Design of Floor Slab

In determining the distribution of the wheel loads on the slab, the length of the area of distribution must be known. The width is 5 feet. Referring to Figure 1, let \( X = \frac{L}{2} \), then the effective length for moment equals \( \frac{4}{3}X \) or 3.33 feet. (Fig. 1)

**MOMENT**

\[ W_l = 16000/3.33 = 4800 \text{ lbs. per ft. Live Load} \]
\[ W_d = 170 \text{ lbs. per ft. Dead Load} \]

**SHEAR**: Effective Length

\( X \) equal to or less than 2.5d (d = 10 in., thickness of slab

\( X = \) 25 inches
DISTRIBUTION OF WHEEL LOAD ON FLOOR SLAB

$\frac{1}{3} \times \text{Effective Width}$

$X = \frac{1}{2}$

$X = \frac{1}{2}$
Effective length = \( \frac{4}{3}(25) = 2.75 \text{ feet} \)

\[ W_1 = \frac{16000}{2.75} = 5900\# \]

\[ W_d = 170\# \]

The bending moment of the slab as a continuous beam is determined by a graphical method of moment distribution as shown by Figures 2 and 3.

**MOMENT**

As simple beam: \( P(\text{concentrated load}) = \frac{Wl}{4} = \frac{1x5}{4} = 1.25 \)

As continuous beam: Max. Positive Mom. = 0.62P = 0.775

Max. Negative Mom. = 0.63P = 0.782

Max. Pos. Mom. = 0.775x4800 plus \( \frac{1}{24}(170)(5)(5)= 3398 \text{ ft}\# \)

Max. Neg. Mom. = 0.782x4800 plus \( \frac{1}{12}(170)(5)(5)= 4109 \text{ ft}\# \)

**SHEAR**

\[ V_c = \frac{1}{2}W_1 = \frac{1}{2}(5900) = 2950 \text{ at center of span} \]

\[ V_s = W_1 \text{ plus } \frac{1}{2}W_d = 6325 \text{ over support} \]

**DEPTH of SLAB**

Moment \( d = (\frac{M}{Kb})^\frac{1}{2} = (3898x12^2/139x12)^\frac{1}{2} = 5.3" \text{ at center} \)

\( d = (\frac{M}{Kb})^\frac{1}{2} = (4109/165)^\frac{1}{2} = 5.0" \text{ over support} \)

Shear \( d = V/v_{jb} = 2950x8/12x60x7 = 4.7" \text{ at center} \)

\( d = V/v_{jb} = 2950x8/12x60x7 = 10.0" \text{ over support} \)

Total Depth = \( d \text{ plus } d' = 10 \text{ plus } 2 = 12" \)

**AREA of STEEL**

\[ A_s = \frac{M}{f_{sdj}} = \frac{3898x12^2}{16000x7x10} = 0.293 \text{ sq.in. at center} \]

\[ A_s = \frac{M}{f_{sdj}} = \frac{4105x12^2}{18000x7x10} = 0.313 \text{ sq.in. over support} \]

\[ \frac{7}{12}x0.293 = 0.714 \text{ sq.in. carried into support at bottom.} \]

**BOND**

Summation(0) = perimeter = \( V/uid = 2950x8/100x7x10 = 3.37" \text{ at bottom} \)

Summation(0) = perimeter = \( V/uid = 6325x8/100x7x10 = 7.24" \text{ over support} \)

Steel as noted on Plans
Uniform Load
Maximum Bending Moment of Slab
Graphical Method

W = Weight of Slab

Bending Moment
\[ \text{Neg} = \frac{1}{2} \times \frac{3}{8} W = \frac{3}{16} W \]
\[ \text{Pos} = \frac{1}{3} \times \frac{25}{8} W = \frac{25}{24} W \]
III. Design of Floor Beams.

Weight of Floor = 5x150x1 = 750 lbs. per ft.

" " Paving = 5x20 = 100 " " "

" " Stem = = 170 " " "

" " Uniform Load = 1020 " " "

Moment of Live Load

\[ M_l = 16000(13.6)(13.6)/30 = 98,600 \text{ ft} \cdot \text{lbs.} \]

Moment of Dead Load

\[ M_d = 1020(30)(30)/8 = 11,488 \text{ ft} \cdot \text{lbs.} \]

\[ M = M_l + M_d = 2,560,000 \text{ in} \cdot \text{lbs.} \]

\[ R = \text{reaction of support} = 10933 + \frac{WL}{2} = \frac{10933 + 15300}{2} = 26,233 \]

\[ R = 26,233 \# \text{ (Rear wheel at critical section)} \]

\[ R = 31,300 \# \text{ (Rear wheel at support)} \]

IV. Interior Beams.

The Interior Beams are designed as T-beams. The limiting value of "b" (see Fig. 4) equals one half the spacing of the beams, therefore,

\[ b = 5 \text{ feet} = 60 \text{ inches}. \]

\[ t = 1 \text{ foot} = 12 \text{ inches}. \]

\[ f_c/f_s = 800/18000 = 0.045 \]

\[ M/f_{sgbt^2} = 2560000/18000x60x144 = 0.0145 \]

On Diagram 2, 0.0145 and 0.045 do not intersect, therefore it is necessary to design as rectangular beam.

\[ d = \left(\frac{M}{K_b}\right)^{\frac{1}{2}} = \left(\frac{2560000}{60x139}\right)^{\frac{1}{2}} = 17.5" \]

\[ d' = 19.5" \]

\[ b = 60" \]

\[ b'd = V/V_j = 31300x8/120x7 = 298, \text{ when } d = 20" \]

\[ b' = 15" \]

Therefore (Fig. 4): \[ t = 12"; \ b = 60"; \ d = 20"; \ d' = 3" \]

\[ b' = 15" \]
20 Ton Truck
Point of Maximum Bending Moment
Floor Beam
\[ A_s = \frac{2667000 \times 8}{18000 \times 7 \times 20} = 8.45 \text{ sq.in.} \]

Use 2 layers: 3 - 1\(\frac{1}{6}\) squares = 3.79 sq.in.

3 - 1\(\frac{3}{4}\) " = 4.68 sq.in.

Checking for value of \(b'\)

\(b' = 3(N - 1)D + D + 3 = 3 \times 2 \times 1\frac{1}{2} + 1\frac{1}{2} + 3 = 11.75\)

Since \(b'\) is 15 inches it is ample.

**STIRRUPS**

\[ V_c = \text{shearing value of concrete} = \frac{V \times d}{2} = 60 \times 15 \times 7 / 8 \times 20 = 15,750\]  

\[ V_c = \text{shear in stirrups} = V - V_c = 31,300 - 15,750 = 15,550\]  

From Diagram 3, spacing of stirrups = 8" for \(\frac{1}{2}\)" round(\(\phi\))

Maximum spacing = \(\frac{45}{(90 + 10)}\) times 20 = 9"  

**BOND**

\[ \frac{V}{V_c} = \frac{31,300 \times 8}{100 \times 7 \times 20} = 17.9'' \]

Requires 3 - 1\(\frac{1}{6}\) squares and 1 - 1\(\frac{1}{8}\) square.

V. Exterior Beam

The Exterior Beam is designed as a Rectangular Beam as shown in Fig. 6.

Weight of Floor per foot of beam = 3 x 150 = 450\#

" Beam = 300\#

" Total = 750\#

The load and moment of the live load for exterior beam is the same as for the interior beam, therefore

\[ bd^2 = \frac{M}{K} = \frac{2560000}{139} = 1835 \]

When \(b = 16.5''\) \(d = 34''\)

\[ V_c = \frac{V \times d}{2} = 60 \times 16.5 \times 7 / 8 \times 34 = 29,500\]  

Shear at edge of support = 31,300 - 1720 = 29,580, as shearing value of concrete is 29,500\#, the entire shear is carried by concrete.
Shear and Moment Diagram
For
Interior and Exterior Beams
Fig 6

Exterior Beam
Steel Layout.
STEEL

\[ A_s = \frac{M}{f_{sij}} = \frac{2560000 \times 3}{18000 \times 7 \times 34} = 4.78 \text{ sq.in.} \text{ Use } 3\frac{1}{4} \text{ sq.} \]

BOND

Summation perimeters = \[ V_{u,jd} = \frac{31300 \times 3}{100 \times 7 \times 34} = 10.55'' \]

Use 2\frac{1}{4} squares at support.

From Fig. 6 cut 1\frac{1}{4} square 7'-6'' from center.

VI. Design of Girder.

Weight of Girder = \( W = 750 \# \)

Concentrated Dead Loads:

Weight of Paving = 5x20x30 = 3000

" " Floor = 5x1x150x30 = 22,500

" " Stem = \( \frac{30 \times 11 \times 15 \times 150}{144} \) = 5150

" " (B) \[ \frac{30,650}{30,650} \]

Concentrated Live Loads:

\( (A) = P \text{ plus } P_1(30-14)/30 \text{ plus } P_1(30-19)/30 \) (Fig. 7)

\( (A) = 16000 \text{ plus } 4000x27/30 = 19,600 \)

<table>
<thead>
<tr>
<th>Load</th>
<th>1st Col.</th>
<th>CL 1st Span</th>
<th>2nd Col.</th>
<th>CL 2nd Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>W</td>
<td>-12.5W</td>
<td>4.17W</td>
<td>-8.33W</td>
<td>4.17W</td>
</tr>
<tr>
<td>B</td>
<td>-5.0B</td>
<td>-0.25B</td>
<td>-0.50B</td>
<td>2.00B</td>
</tr>
<tr>
<td>A</td>
<td>-.50A</td>
<td>2.18A</td>
<td>-1.93A</td>
<td>2.60A</td>
</tr>
</tbody>
</table>

Moments -- (ft. lbs.)

<table>
<thead>
<tr>
<th>Load</th>
<th>1st Col.</th>
<th>CL 1st Span</th>
<th>2nd Col.</th>
<th>CL 2nd Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>W</td>
<td>-9375</td>
<td>3127.5</td>
<td>6255</td>
<td>3127.5</td>
</tr>
<tr>
<td>B</td>
<td>-153250</td>
<td>-7662.0</td>
<td>-15324</td>
<td>61300.0</td>
</tr>
<tr>
<td>A</td>
<td>-98000</td>
<td>42750.0</td>
<td>-37828</td>
<td>50960.0</td>
</tr>
<tr>
<td></td>
<td>-250625</td>
<td>53,539.5</td>
<td>-59407</td>
<td>115397.5</td>
</tr>
</tbody>
</table>

\( b = 20; \quad d = 30.8 \quad 15.2 \quad 14.7 \quad 22.5 \)

Shears \( v = 60 \quad \dot{a} = \frac{V}{u,jb} = \frac{V}{(7/8 \times 60 \times 20)} = \frac{V}{1050} \)
Fig 7

LOADS ON GIRDER

P = WHEEL LOAD

W = WEIGHT OF GIRDER

B = DEAD LOAD
FIG. 8

Uniform Load
Maximum Bending Moment
Graphical Method

W = Weight of Girder

M. Line

\frac{1}{3}W \quad \frac{3}{5}W \quad \frac{2}{3}W
Floor Beam Loads
Maximum Bending Moment of Girder
Concentrated Loads
Maximum Bending Moment of Girder
Graphical Method
End of Cantilever : \( V = A + B = 19600 + 30650 = 50250 \quad d = 47.8 \)

Left of 1st. Col. : \( V = A + B + 5W \quad = 54000 \quad d = 51.4 \)

Right of 1st Col. : \( V = \frac{1}{2}(A + B + 10W) \quad = 28875 \quad d = 27.5 \)

Center 1st Span : \( V = \frac{1}{2}(A + B) \quad = 25125 \quad d = 24.0 \)

Left of 2nd Col. : \( V = \frac{1}{3}(A + B + 10W) \quad = 28875 \quad d = 27.5 \)

Right of 2nd Col. : \( V = \frac{1}{3}(A + B + 10W) \quad = 28875 \quad d = 27.5 \)

Center 2nd Span : \( V = \frac{1}{3}(A + B) \quad = 25125 \quad d = 24.0 \)

Use \( d = 31.0" \); Shear in Cantilever:

\[
V = \frac{ujbd}{120x7/8x20x31} = 65,000
\]

Then cantilever is safe with Stirrups.

**STEEL**

\[
A_s = \frac{M}{f_{sg}d} = 0.00076M/d
\]

Summation Perimeter = \( V/ujd = 0.0114V/d \)

<table>
<thead>
<tr>
<th></th>
<th>1st Col.</th>
<th></th>
<th>2nd Col.</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>As (top)</td>
<td>6.14</td>
<td></td>
<td>1.32</td>
<td>1.46</td>
</tr>
<tr>
<td>Summation (c)</td>
<td>19.90</td>
<td>9.24</td>
<td>10.65</td>
<td>9.24</td>
</tr>
<tr>
<td>Bars</td>
<td>4-1(\frac{1}{2}) sq.</td>
<td>1-1(\frac{1}{8}) sq.</td>
<td>2-1(\frac{1}{2}) sq.</td>
<td>1-1(\frac{1}{8}) sq.</td>
</tr>
</tbody>
</table>

**STIRRUPS**

Use 3/8" Round "U"

Left of 1st Col. : \( V_c = vbjd = 60x20x7/8x31 = 32550 \)

\[
V_s = V - V_c = 54000 - 32550 = 21450
\]

Diagram 3 : Spacing = 6"

End of Cantilever : \( V_s = V - V_c = 50250 - 32550 = 17700 \)

Diagram 3 : Spacing = 6"

**VII. Design of Exterior Column.**

Case 1. Eccentric Loading - Reference Fig. 11

\[
A = 2x19600 + 0.5x19600 = 49000
\]

\[
B = 2.5x30650 = 76625
\]

\[
W = 10x750 = 7500
\]

\[
N = 133125
\]
Eccentric Moment

\[ -5 \times 53500 = -267500 \text{ ft. lbs.} \]
\[ 0.5 \times 5 \times 53500 = \underline{133750} \quad " " \]

Total Moment

\[ -133750 \quad " " \]

Eccentricity = \( X_c = \frac{M}{N} = \frac{133750}{133325} = 1.004' = 12" \)

Case 2. Eccentric Loading - Reference Fig. 12

\[ A = 2 \times 19600 = 39200 \]
\[ B = 2.5 \times 30650 = 76625 \]
\[ W = 10 \times 750 = \underline{7500} \]
\[ N = \underline{123325} \]

Eccentric Moment

\[ -5(19600 \text{ plus } 30650) = -251250 \text{ ft. lbs.} \]
\[ 5 \times 0.5 \times 30650 = \underline{76625} \quad " " \]
\[ -174625 \quad " " \]

Eccentricity = \( X_c = \frac{174625}{123325} = 1.416' = 17" \)

Maximum Eccentricity = 17"

\( f_c = 600 \); \( f_s = 16000 \); \( n = 15 \); \( t = 24" \); \( \frac{d'}{t} = 0.10 \)

\( N = 123325 \); \( X_c = 17" \); \( p = 0.04 \)

Diagram 4 fails to give intersection.

Assume \( t = 26" \); \( \frac{t}{X_c} = \frac{26}{17} = 1.53 \)

\( np = 15 \times 0.04 = 0.60 \); with \( \frac{t}{X_c} \) as arguments, Dia. 5 gives:

\( \frac{N X_c}{f_c b t^2} = 0.251 \); \( b = 123325 \times 17/600 \times 0.251 \times 26 \times 26 = 20" \)

\( A_g = p b t = 0.04 \times 20 \times 26 = 20.8 \text{ sq. in.} \)

Use 8-1\(\frac{1}{8}\) sq. Bars in each b face.

Check for \( b : D(N-1)3 \text{ plus } D \text{ plus } 4 = 28\frac{3}{4} \)

Since \( b = 20" \), column is too small.

Assume \( t = 30" \); \( \frac{t}{X_c} = \frac{30}{17} = 1.77 \); \( p = 0.025 \)

\( np = 15 \times 0.025 = 0.375 \); \( \frac{N X_c}{f_c b t^2} = 0.189 \)
Eccentric Loading
of Exterior Columns
Case 1
Eccentric Loading of Exterior Columns

Case 2.
\[ b = \frac{123315 \times 17}{600 \times 0.189 \times 30 \times 30} = 20.2 \]

\[ A_s = \text{pbt} = 0.025 \times 20 \times 30 = \text{sq.in.} \]

Use 5-1\(\frac{3}{4}\) sq. bars in each face (b). \((10 \times 1.563) = 15.63\)

Check for \(b\): \(D(N-1)D\) plus \(D\) plus 4 = 20\(\frac{3}{4}\)

Use \(\frac{3}{4}\)" round ties - 8 inches on center.

**VIII. Design of Interior Column.**

Case 2. Eccentric Loading - Reference Fig. 13

\[ A = 1.5 \times 19600 = 29400 \]
\[ B = 2.0 \times 20850 = 61300 \]
\[ C = 10 \times 750 = 7500 \]
\[ N = 98200 \]

Eccentric Moment

\[-0.5(30625 \text{ plus } 19600)5 = -125625 \]
\[0.5(30625)5 = \frac{7500}{49000} \]

Eccentricity = \(X_c = \frac{M}{N} = \frac{49000}{98200} = 0.50' = 6\"

Assume \(t = 15''\); \(t/X_c = 2.5\); \(p = 0.4\)

\[ np = 0.4 \times 15 = 0.60 ; \]
\[ NX_c/f_{cgt}^2 = 0.225 \]

\[ b = 98200x5/600 \times 0.225x15x15 = 19.5 \]

Try \(b = 20''\)

\[ A_s = \text{pbt} = 0.04 \times 20 \times 15 = 12 \text{ sq.in.} \]

Use 8-1\(\frac{3}{4}\) sq. bars \((8 \times 1.563 = 12.5)\)

Check for "b": \(3(N-1)D\) plus \(D\) plus 4 = 15\(\frac{3}{4}\)

Since \(b\) is 20, this is ample.

**IX. Design of Exterior Column Footings.** (Fig. 14)

Moment of Column = 174625 ft.lbs.

Load on Column = 123300 lbs.

Weight of Column = 12500 "

\[ N " " = 135800 " \]
\[ X_c \text{ (footing)} = 174625/135800 = 1.285 \]

T less than 6x1.285 = 7.71 ; \( W = 4200/9 \) (assumed)

\[ L = 5 \text{ Tons per sq.ft.} \]

\[ B(T/2 - X_c) = (N + N)/30000 = 140000/15000 = 9.33 \]

\( B = b + 3.13 = 5' \); \( 5(T/2 - 1.285) = 9.33 \); \( T = 6.3 \)

\[ a = (0.001)/LB = 0.001x135800/5x5 = 5.4' \]

\[ y = 2N/aB = 2x135800/5.4x5 = 10050' \]

\[ y_2 = y(1-T-t) = 10000(1-3.8/10.8) = 5480' \]

\[ P_1 = \text{punching shear} = \frac{T-t}{12}(8y_2 + 2y) + b(y + 2yg) \]

\[ P_1 = (3.8/12)(5x36480 + 1.67x22960) = 54000' \]

\[ M_1 = 3/gP_1(T-t) = 3x54000x3.3/8 = 77000 \text{ ft.lbs.} \]

\[ P_2 = \text{punching shear along } t = (T + t)/6 \times (B-b)y \]

\[ P_2 = (7.8/6)x3.13x10000 = 40700' \]

\[ M_2 = 3x40700x3.13/8 = 47800 \text{ ft.lbs.} \]

\[ d_1 = (\frac{M}{bk})^{1/3} = (77000/1.57x139)^{1/3} = 13.5'' \]

\[ d_1 = \frac{V}{by} = 54000/1.67x120x12 = 22.5'' \]

\[ d_2 = (\frac{M}{bk})^{1/3} = (47800/2.5x139)^{1/3} = 11.3'' \]

\[ d_2 = \frac{V}{by} = 40700/120x12x2.5 = 11.3'' \]

\[ y_4 = y(1 - 0.5(T-t-\delta)) = 10000(1-0.36/5.4) = 9330' \]

\[ HI = 1.67 \text{ plus } 1.54x2 = 4.75' \]

\[ HC = 2.5 \text{ plus } 3.03 = 5.53' \]

\[ V_1 = \text{shear on trapezoid } \text{MIG} \]

\[ V_1 = \frac{0.36(5 \text{ plus } 4.75)}{2} \times 10000 \text{ plus } 9330 = 17000' \]

\[ V_2 = \text{shear on trapezoid } \text{CHD2} \]

\[ V_2 = \frac{0.36(6.3 \text{ plus } 5.53)}{2} \times 10000/2 = 10600' \]

\[ v = V/bjd = 17000x8/7.5x5.58x18.5x12 = 13.4 \text{ lbs. per sq.in.} \]

\[ v = V/bjd = 10600x8/7.5x5.58x18.5x16 = 9.77 " " " \]

As the allowable shearing stress of concrete is 40 lbs. per sq. in., these stresses are safe.
Fig. 14

Exterior Column footing

Diagram showing exterior column footing with dimensions labeled as follows:
- N
- d
- t
- y
- y_1
- y_2
- B
- d
- t
- d
- a
- T

Dimensions and labels indicate the structural design of the column footing.
STEEL

\[ A_s \text{ (in T direction)} = \frac{M_1}{f_sjd} = \frac{77000 \times 12 \times 8}{7 \times 18000 \times 18.5} = 3.95 \text{ sq. in.} \]

Steel in T direction is distributed over a width B unless B is greater than b plus 2d, in which case the width over which steel is distributed is 0.5(B plus b) plus d. Outside of this width the same size bars are used with double spacing.

\[ b \text{ plus } 2d = 1.67 \text{ plus } 3.08 = 4.75 \text{ which is less than } 5' \]

Distribute Steel: 0.5(5 plus 1.67) plus 1.54 = 4.88

\[ A_s \text{ (per foot)} = \frac{3.95}{4.88} = 0.81 \text{ sq. in.} \]

\[ (S)(O) \text{ (per foot)} = \frac{V1}{u_{jd}} = \frac{17000 \times 8}{7 \times 75 \times 18.5} = 14.0 \text{ in.} \]

\[ (S)(O) \text{ (per foot)} = \frac{14}{4.88} = 2.87 \text{ in.} \]

Use 1" rounds - 12" O.C.

\[ A_s \text{ (in B direction)} = \frac{M_2}{f_sjd} = \frac{47800 \times 8 \times 12}{7 \times 18000 \times 18.5} = 1.97 \text{ sq. in.} \]

\[ t \text{ plus } 2d = 2.5 \text{ plus } 3.04 = 5.54 \text{ which is less than } T=6.3 \]

Distribute Steel: 0.5(T plus t) plus d = 0.5(6.3 plus 2.5) plus 1.54 = 5.94

\[ A_s \text{ (per foot)} = \frac{1.97}{5.94} = 0.33 \text{ sq. in.} \]

\[ (S)(O) = \frac{V2}{u_{jd}} = \frac{10600 \times 8}{75 \times 75 \times 18.5} = 3.73 \text{ in.} \]

\[ (S)(O) \text{ (per foot)} = \frac{3.73}{5.94} = 1.47 \text{ in.} \]

Use 3/4" round - 12" O.C.

X. Design of Interior Column Footing (Fig. 15)

Moment of Column = 49000 ft. lbs.

Load on " = 108000 lbs.

Weight of " = 6250 "

N of " = 114260 "

\[ X_c \text{ (footing)} = \frac{49000}{114260} = 0.43' \]

\[ T = 3.39 \text{ is greater than } 6 \times 0.43 = 2.58 \]
\[ BT = \frac{0.005N}{L} + W(1 + \frac{2.58}{T}) = 11.75 + 30.3/T \]
\[ BT^2 = 11.75T - 30.3 = 0; \quad B = 5; \quad T = 3.89 \]
\[ Y = \left( \frac{N}{BT} \right) + \left( \frac{6NY}{BT^2} \right) \]
\[ = (10800/5x3.89) + (6x108000x0.43/5x3.89x3.89) = 9130 \]
\[ Y_1 = \left( \frac{N}{BT} \right) - \left( \frac{6NY}{BT^2} \right) = 5450 - 3680 = 1770 \]
\[ Y_2 = y - 0.5(y - y_1)(1 + \frac{t}{T}) \]
\[ = 9130 - 3680(1 + \frac{1.25}{3.89}) = 4270 \]
\[ Y_2 = y - 0.5(y - y_1)(1 - \frac{t}{T}) \]
\[ = 9130 - 2500 = 7000 \]

\[ P_1 = \text{upward pressure on trapezoid ABFD} \]
\[ P_1 = \left( \frac{T-t}{12} \right)(B(y_2 + 2y) + b(y + 2y_2)) \]
\[ = (2.64/12)(5x12000 + 1.67x223130) = 25900 \text{ lbs.} \]

\[ M_1 \text{ acts along the line AB} \]
\[ M_1 = \frac{3}{8}P_1(T-t) = 3x359000x2.64/8 = 35600 \]

\[ P_2 = \text{upward pressure on trapezoid ACED} \]
\[ P_2 = \left( \frac{B-b}{12} \right)(0.5T(y + y_1 + y_2) + 0.5T(y_1 + y_2 + y_3)) \]
\[ = \frac{3.33}{12}(1.99x17900 + 0.625x13050) = 12200 \text{ lbs.} \]

\[ M_2 = \frac{3}{8}P_2(B-b) = 3x122000x3.33/8 = 15250 \text{ ft.lbs.} \]

\[ d_1 = \left( \frac{M_1}{K_b} \right)^{\frac{1}{3}} = (35000/139x1.67)^{\frac{1}{3}} = 12.3'' \]

\[ d_2 = \left( \frac{M_2}{K_t} \right)^{\frac{1}{3}} = (14750/139x1.25)^{\frac{1}{3}} = 9.54'' \]

\[ d_1 = \frac{P_1}{v_b} = 35400/120x20 = 14.75'' \]

\[ d_2 = \frac{P_2}{v_t} = 11800/120x15 = 6.55'' \]

Use \( d = 15'' \)

\[ Y_4 = y - (y - y_1)(T - \frac{0.5(T-t) - d}{T}) \]
\[ = 9130 - 7360x1.32/3.89 = 9130 - 170 = 8960 \]

\[ v_1 = \frac{3.89 + 3.75}{2} \left( \frac{2}{0.42(9130 + 7000)} \right) = 12900 \]

\[ v_2 = \frac{5 + 4.17}{2}(0.07)(10900) = 0.236x10900 = 3120 \]

\[ v = 12900x8/7x75x15 = 13.15 \text{ lbs, per sq.in.} \]
\[ v = \frac{3120x8}{7x75x15} = 3.17 \text{ lbs. per sq. in.} \]

\[ A_s \text{ (in T direction)} = \frac{M}{f_{sd}} = \frac{35000x3x12}{16000x7x15} = 1.243 \text{ sq. in.} \]

\[ b + 2d = 1.67 + 2.50 = 4.17 \quad \text{Less than B = 5} \]

Distribute Steel: 0.5(5 plus 1.67) plus 1.25 = 4.58

\[ A_s \text{ (per foot)} = \frac{1.243}{4.58} = 0.272 \text{ sq. in.} \]

\( (S)0 = \frac{12900x8}{75x7x15} = 13.15 \)

\[ (S)0 \text{ (per foot)} = \frac{13.15}{4.58} = 2.85 \]

Use \( \frac{7}{8}'' \) rounds - 12" 0.3.

\[ A_s \text{ (in B direction)} = \frac{15250x12x8}{16000x7x15} = 0.774 \text{ sq. in.} \]

\[ t + 2d = 1.25 + 2.50 = 3.75 \quad \text{Less than T = 3.39} \]

Distribute Steel: 0.5(3.89 plus 1.25) plus 1.25 = 3.82

\[ A_s \text{ (per foot)} = \frac{0.774}{3.82} = 0.203 \]

\( (S)0 = \frac{3120x8}{75x7x15} = 3.18 \)

\[ (S)0 \text{ (per foot)} = \frac{3.18}{3.82} = 0.83 \]

Use \( \frac{3}{8}'' \) rounds - 12" 0.3.
Part II

DESIGN of SLAB-BEAM-GIRDER COLUMN BRIDGE
Encased Steel Type

I. General Data

This bridge designed for 20 ton truck load similar to bridge designed in Part I.

Stresses:

\[ f_c = 900 \]
\[ f_s = 18000 \]
\[ n = 15 \]
\[ h = \text{height of steel beam} \]
\[ a = \text{thickness of concrete above steel} \]
\[ I_s = \text{moment of inertia of steel section} \]
\[ I = " \quad " \quad " \quad " \quad \text{compound section} \]
\[ k = 0.4 \]

II. Design of Floor Slab

The floor slab for this bridge is similar to the slab used for the bridge designed in Part I. For this reason, that design is used and made an integral part of the plans for this type bridge. For all dimensions and other data, reference is made to "Design of Floor Slab," Part I.

III. Design of Interior Floor Beam.

\[ R_1 = \text{left reaction (Refer to Fig. 16)} \]
\[ R_1 = (P + P_1)(1 - (x + 2.8)/L) + 1050(L/2) \]
\[ P_1 + P_2 = 20,000 \text{#} \]
\[ L = 30' \quad ; \quad L/2 = 15' \]
\[ R_1 = 20000 - (20000X + 56000)/30 + 1050x15 \]
\[ R_1 = 33880 - 667x \]
Fig 16

T-Beam
Encased Steel

\[ \text{Width} \times \text{Depth} \times \text{Length} \]
\[ M = R_1X - 525X^2 = 33880X - 667X^2 - 525X^2 = 33880X - 1199X^2 \]

<table>
<thead>
<tr>
<th>( X (\text{ft}) )</th>
<th>( R ) (lbs)</th>
<th>( M ) (ft-lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>33890</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>31220</td>
<td>116480</td>
</tr>
<tr>
<td>8</td>
<td>28554</td>
<td>194830</td>
</tr>
<tr>
<td>12</td>
<td>25894</td>
<td>233200</td>
</tr>
<tr>
<td>14.1</td>
<td>24490</td>
<td>240500</td>
</tr>
<tr>
<td>15.0</td>
<td>23880</td>
<td>239300</td>
</tr>
</tbody>
</table>

For encased steel shape use 18" - 62# I.

\[ A_s = 18.91 \quad I_s = 1045 \quad h \quad plus \quad a = d = 21 \quad kd = 8.4 \]

\[ I = I_s \quad plus \quad A_s(h/2 \quad plus \quad a - kd)^2 \quad plus \quad (b_1(kd)^2/3n) \]

\[ plus \quad \frac{b_0^2}{n}(t^2/12 \quad plus \quad (kd - t/2)^2) \]

\[ I = 1045 \quad plus \quad (18.81x3.5x3.5) \quad plus \quad (15x8.4^3)/3x15 \]

\[ plus \quad ((12.75x12)/15)(12 \quad plus \quad 2.4x2.4) \]

\[ I = 1045 \quad plus \quad 244 \quad plus \quad 198 \quad plus \quad 131 = 1668 \]

\[ b = \frac{2nA_s}{(kd)^2}(h/2 \quad plus \quad a - kd) = (2x15x1831/8.4x8.4)(3.6) = 28.8 \]

\[ b_o = (28.8-15)/8.4x8.4x8.4/(12 \quad plus \quad 2.4x2.4)36 = 12.75 \]

\[ M = 15x800x1668/8.4 = 2,400,000 \]

\[ v = (aV/nI)(kd - a/2) = 3x33890x6.9/15x1668 = 28\% \text{ per sq.in} \]

As the Moment and Shear are safe, the design is correct.

\[ b_1 \quad plus \quad b_o = 15 \quad plus \quad 12.75 = 27.75 \]

IV. Design of Exterior Floor Beam (Fig. 17)

**LOAD**

Weight of Floor = 3x150 = 450#

" Railing = 300

750#

Maximum Moment Dead and Live Load

<table>
<thead>
<tr>
<th>( X (\text{ft}) )</th>
<th>( R ) (lbs)</th>
<th>( M ) (ft-lbs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>14.1</td>
<td>24490</td>
<td>240500</td>
</tr>
</tbody>
</table>
Exterior Beam
Encased Steel

Fig. 17
Use 21\"-56#/ I.

\[ A_s = 16.9 \; ; \; I_s = 1337 \; ; \; kd = 0.4 \times 30 = 12 \]
\[ d = 30 \; ; \; h = 21 \; ; (h/2 \; plus \; a - kd) = 11 \; plus \; 7 - 11.6 = 6.4 \]
\[ b = 2 \times 15 \times 17.06 \times 6.4 / 11.6 \times 11.6 = 24" \]
\[ I = 1337 \; plus \; (17.06 \times 6.4) \; plus \; (2.4 \times 11.6^3) / 4.5 = 2867 \]
\[ v = 7 \times 33890 \times 8.1 / 15 \times 2867 = 45 \]
\[ M = n f_s I / kd = 15 \times 800 \times 2867 / 11.6 = 2,950,000 \; \text{in.} \cdot \text{lbs.} \]
\[ M = 246,000 \; \text{ft.} \cdot \text{lbs.} \]

V. Design of Girder

1. Uniform load \[ W = 735 \]

2. Concentrated dead loads:
   
   Weight of Paving = 55x20x30 = 3000
   
   " " Floor = 5x1x30x150 = 22500
   
   " " Stem = 30x11x15 / 144 = 5150

   B = 30650

3. Concentrated live load:

   A = 16000 plus 4000x16 / 30 plus 4000x11 / 30 = 19600

   The bending moments in the bent consisting of four columns and a girder are calculated by the Hardy Cross method as shown in Figures 18 and 19.


<table>
<thead>
<tr>
<th>Load</th>
<th>M (ft. lbs) 1st Col.</th>
<th>M (ft. lbs) 2nd Col.</th>
</tr>
</thead>
<tbody>
<tr>
<td>W</td>
<td>-9180</td>
<td>-5690</td>
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<tr>
<td>B</td>
<td>-76500</td>
<td>-29400</td>
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<tr>
<td>A</td>
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<td>-67790</td>
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</table>

From Fig. 17: a = 4; h = 22; d = 26; k = 0.4; kd = 10.4

Use 22"-62#/ I; \[ A_s = 16.19 \; ; \; I_s = 1465.7 \]
Maximum Bending Moment of Dead Loads
by
Cross Method of Moment Distribution.

\[
\begin{array}{cccccccc}
M = 76.5^2 & M = 19.2 & M = 19.2 & M = 19.2 & M = 19.2 & M = 19.2 & M = 76.5^2 \\
15^2 & 15^2 & 15^2 & 15^2 & 15^2 & 15^2 & 15^2 \\
1.2 & 1.2 & 1.2 & 1.2 & 1.2 & 1.2 & 1.2 \\
-76.5 & +19.2 & 0.0 & 0.0 & 0.0 & 0.0 & +76.5 \\
0.0 & 0.0 & 0.0 & 0.0 & 0.0 & 0.0 & 0.0 \\
0.0 & 0.0 & 0.0 & 0.0 & 0.0 & 0.0 & 0.0 \\
0.0 & -5.4 & 0.0 & 0.0 & 0.0 & 0.0 & 0.0 \\
0.0 & +4.6 & 0.0 & 0.0 & 0.0 & 0.0 & 0.0 \\
-76.5 & +65.1 & -29.4 & +17.1 & -27.1 & +24.4 & -65.1 \\
\end{array}
\]
### Maximum Bending Moment in Uniform Load
Cross Method of Moment Distribution

<table>
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<tr>
<th>L/2</th>
<th>1.0</th>
<th>2.0</th>
<th>3.0</th>
<th>4.0</th>
<th>5.0</th>
<th>6.0</th>
<th>7.0</th>
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<tr>
<td>W</td>
<td>7.35^2</td>
<td>per ft</td>
<td></td>
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</table>

<table>
<thead>
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<th>L/2</th>
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<th>6.0</th>
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<td>0.3</td>
<td>0.4</td>
<td>0.5</td>
<td>0.6</td>
<td>0.7</td>
</tr>
<tr>
<td>M</td>
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<td>0.4</td>
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<td>1.2</td>
<td>1.6</td>
<td>2.0</td>
<td>2.4</td>
<td>2.8</td>
</tr>
</tbody>
</table>

Legend: Moments (in ft-lb)
### Fig. 20

**Exterior Column Maximum Bending Moment of Live Loads by Cross Method of Moment Distribution**

\[
M = 19.6 \times 10^6 \cdot 98
\]

\[
M = 18.6 \times 10^6 = 245
\]

\[
M = 19.6 \times 10^6 = 98
\]

<table>
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<th>1/4</th>
<th>3/4</th>
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<table>
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<th>1/8</th>
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<tr>
<td>+9.6</td>
<td>-5.7</td>
<td>-10.6</td>
<td>+10.6</td>
<td>+5.7</td>
<td>-11.3</td>
<td>-11.3</td>
</tr>
</tbody>
</table>
Fig. 21

**Interior Column**

**Maximum Bending Moment of Live Loads**

by

**Cross Method of Moment Distribution.**

\[ M = 24^\circ \]

\[ M = 24^\circ \]
\[ b = \frac{2nA_s}{(kd)^2}(h/2 \text{ plus } a - kd) = (2 \times 15 \times 13.19/10.4 \times 10.4)(4.5) = 23.0 \]
\[ I = 1466 \text{ plus } (18.19 \times 4.6 \times 4.6) \text{ plus } 23 \times (10.4)^3/45 = 2425 \]
\[ f_c = Mkd/nI = 183680 \times 12 \times 10.4/15 \times 2425 = 630 \]
\[ f_s = M(1-k)d/I = 183680 \times 12 \times 0.6 \times 10.4/2425 = 14750 \]
\[ v = (aV/nI)(kd-a/2) = 4 \times 57500(10.4-2)/15 \times 2425 = 53 \]

VI. Design of Exterior Column

**LOAD**

A = 2 \times 19600 = 39200

B = 2.5 \times 30650 = 76625

W = 10 \times 750 = 7500

N = 123325

**MOMENTS**

Fig. 19: Uniform Dead Load: \[ M = 610 \text{ ft.lbs.} \]

Fig. 18: Concentrated " " : \[ M = 11400 \]

Fig. 20: " Live Load : \[ M = 20600 \]

\[ M = 32610 \text{ ft.lbs.} \]

\[ X_c = M/N = 32610 \times 12/123325 = 3.17" \]

Assume: \[ p = 0.01 \; ; \; t = 16" \; ; \text{ since } n = 15 \; , \text{ then} \]

\[ np = 0.01 \times 15 = 0.15 \; ; \; t/X_c = 16/3.17 = 5.09 \]

From Dia. 5, \[ N_{x_c}/f_c b t^2 = 0.110 \]

\[ b = 123325 \times 3.17/600 \times 0.110 \times 16 \times 16 = 23" \]

\[ A_s = pt = 0.01 \times 23 \times 16 = 3.68 \]

Use 3-1\(1/8\)" squares in each face

Size: \[ b = 23" \; ; \; t = 16 \text{ plus } 4 = 20" \]

VII. Design of Interior Column.

The maximum bending moment of this column is shown in Fig. 21. However the design of the column concentrically loaded requires a larger section than the eccentric moment shown in Fig. 21. Therefore the column is loaded with live and dead loads directly
over column and five feet right and left respectively.

**LOAD**

\[
A = 2 \times 19600 = 39200
\]

\[
B = 2 \times 30650 = 61300
\]

\[
W = 10 \times 750 = 7500
\]

\[
N = 108000
\]

Assume: \( b = 23; t = 14 \)

\[
A = 14 \times 23 = 322
\]

\[
N = 400 \times 23 \times 14 \times 1.07 = 137500
\]

\[
R = 4
\]

\[
N' = N(1.33 - h/120R) = N(1.33 - 0.5) = 0.83N
\]

\[
N = 108000/0.83 = 130000
\]

Assume: \( b = 23; t = 15 \)

\[
A = 15 \times 23 = 345
\]

\[
N = 400 \times 23 \times 15 \times 1.07 = 143000
\]

\[
A_s = 0.005 \times 345 = 1.725
\]

Use 4-\( \frac{3}{8}'' \) round in each face

**VIII. Design of Exterior Footing (Fig. 22)**

Load = \( N = 123325 + 20(23 \times 16/144)150 = 132300 \)

Moment = \( M = 32610 \text{ ft.} \text{lbs.} \) (See Design of Exterior Col.)

Eccentricity = \( X_c = 32610/132300 = 0.246 \text{ ft.} = 3 \text{ in.} \)

\( W \) (assumed) = \( Wt. \) of Footing = 4700\#

\[
BT^2 = 0.0005\left(\frac{N + W}{L}\right)(1 + 6X_c/T)
\]

\[
= 13.7T \text{ plus 20.25; } B = 5'\]

\[
T^2 = 2.74T \text{ plus 4.04}
\]

\[
T = 4.0'
\]

\[
y = N/BT + 6NX_c/BT^2 = 132300/5x4 \text{ plus } \frac{6 \times 132300 \times 0.246}{5 \times 4 \times 4}
\]

\[
= 9040\#
\]
\[ y_1 = \frac{N}{BT} - \frac{6NX_c}{BT^2} = 6600 - 2440 = 4160 \# \]
\[ y_3 = y - \left( \frac{y-y_1}{z} \right)(1 + \frac{t}{T}) = 9040 - \left( \frac{9040-4160}{2} \right)(1 + \frac{1.33}{4.0}) \]
\[ \quad = 9040 - 3250 = 5790 \# \]
\[ y_2 = 9040 - 2440 \times 0.57 = 7410 \# \quad (P_1= \text{punching shear along"b"}) \]
\[ P_1 = \frac{T-t}{12}(B(y_2 \text{ plus } 2y) \text{ plus } b(y \text{ plus } 2y_2)) \]
\[ \quad = \frac{4.0 \times 1.33}{12}(5(7410 \text{ plus } 18080) \text{ plus } 1.92(9040 \text{ plus } 14420)) \]
\[ \quad = (2.57/12)(102000 \text{ plus } 45100) = 32800 \# \]
\[ M_1 = \text{moment about "b"} = \frac{3}{8}P_1(T-t) \]
\[ = 3 \times 32800 \times 2.57/3 = 32800 \text{ ft.lbs.} \]
\[ P_2 = \text{punching shear along "t"} \]
\[ = \frac{B-b}{12}(y_1 \text{ plus } y_2 \text{ plus } y_3) \text{ plus } \frac{t}{2}(y_1 \text{ plus } y_2 \text{ plus } y_3) \]
\[ = (3.08/12)(2(4160 \text{ plus } 7410 \text{ plus } 5790)) \text{ plus } (0.67)(4160 \text{ plus } 7410 \text{ plus } 5790) \]
\[ = 3.08 \times 46300/12 = 11900 \# \]
\[ M_2 = \text{moment about "t"} = \frac{3}{8}P_2(B-b) \]
\[ = 3 \times 11900 \times 3.08/3 = 13750 \text{ ft.lbs.} \]
\[ d_1 = (M_1/kb)^{1/3} = (32800/139 \times 1.92)^{1/3} = 11.1" \]
\[ d_2 = (M_2/kt)^{1/3} = (13750/139 \times 1.33)^{1/3} = 8.6" \]
\[ d_1 = P_1/vb = 32800/120 \times 23 = 12.0" \]
\[ d_2 = P_2/vt = 13750/120 \times 16 = 7.2" \]
Use: \( d = 12" \)

Checking for value of \( W = 5 \times 4 \times 1.46 \times 150 = 4370 \# \)
\[ y_4 = y - (y-y_1)(\frac{T-t}{T} - d) = 9040 - (9040-4160)(\frac{4-1.33}{4} - 1) \]
\[ \quad = 9040 - 390 = 8650 \]
\[ V_1 = \text{Area}(\text{ENIF})x(y \text{ plus } y_4)/2 \]
\[ IN = 47" = 3.92' \]
\[ \text{Area (ENIF)} = \frac{3.92 \times 5 \times 0.33}{2} = 1.48 \]
\[ (y \text{ plus } y_4)/2 = (9040 \text{ plus } 8650)/2 = 8845 \]
\[ V_1 = 1.48 \times 8845 = 13100 \]
\[ V_2 = \text{area}(\text{GIFJ}) \times (y \text{ plus } y_1)/2 \]
\[ GI = 40'' = 3.33' \]
\[ \text{Area (GIFJ)} = 0.33(3.33 \text{ plus } 4)/2 = 1.22 \]
\[ (y \text{ plus } y_1)/2 = (9040 \text{ plus } 4160)/2 = 6600 \]
\[ V_2 = 1.22 \times 6600 = 8060 \text{ (} \]
\[ v_1 = \text{vertical shear along line NI} = V_1/Bjd \]
\[ = 13100x8/5 \times 12 \times 7 \times 12 = 21 \text{#}/\text{sq. in.} \]
\[ v_2 = \text{vertical shear along line GI} = V_2/Tjd \]
\[ = 8060x8/4 \times 12 \times 7 \times 12 = 16 \text{#}/\text{sq. in.} \]
\[ A_s (\text{in T direction}) = M/f_{s,jd} = 32800x8 \times 12/18000x7 \times 12 = 2.08 \text{ b plus } 2d = 1.91 \text{ plus } 2 = 3.91' \text{ which is less than 5'} \]
\[ \text{Distribute steel: } 0.5(1.91 \text{ plus } 5) \text{ plus } 1 = 4.96' \text{ (Appr. 5')} \]
\[ A_s (\text{per foot}) = 2.08/5 = 0.416 \text{ sq. in.} \]
\[ S(0) = V_1/ujd = 13100x8/75 \times 7 \times 12 = 15.7 \]
\[ S(0) = 15.7/5 = 3.14'' \text{ per foot.} \]
\[ \text{Use 1'' round-12'' OC.} \]
\[ \text{Distribute steel: } 0.5(1.33 \text{ plus } 4) \text{ plus } 1 = 3.67 \]
\[ A_s = 13750x8x12/18000x7x12 = 0.873 \text{ sq. in.} \text{ (in B direction)} \]
\[ A_s (\text{per foot}) = 0.873/3.67 = 0.238 \text{ sq. in.} \]
\[ S(0) = V_2/ujd = 8060x8/75 \times 7 \times 12 = 9.57 \]
\[ S(0) = 9.57/3.67 = 2.61 \text{ per foot} \]
\[ \text{Use 7/8'' round-12'' OC.} \]

**IX. Design of Interior Footing**

\[ \text{Load} = N = 108000 \text{ plus } 150(23 \times 15)/144 = 111750# \]
\[ \text{Weight of Footing} = W = 4000# \]
\[ BT = (N \text{ plus } W)/2000 = 115750/2000 \times 5 = 11.58 \]
\[ B = 4 \text{ ft.}; T = 3.25 \]
\[ y = N/BT = 111750/4 \times 3.25 = 9050 \]
\[ P_1 = \text{punching shear along "b"} \]
$M_1 =$ moment about line "b"

$P_1 = (B \text{ plus } b)(T-t)y/4 = 28150\#$

$W_1 = (T-t)(2B \text{ plus } b)y/24 = 15100 \text{ ft.} \text{lbs.}$

$P_2 =$ punching shear along "t"

$M_2 =$ moment about line "t"

$P_2 = (T \text{ plus } t)(B-b)y/4 = 20400$

$W_2 = (B-b)(2T \text{ plus } t)y/24 = 11700 \text{ ft.} \text{lbs.}$

$d_1 = (M_1/K_b)^{1/3} = (15100x12/139x23)^{1/3} = (53)^{1/3} = 7.27 \text{ in.}$

$d_1 = P_1/v_b = 28150/120x23 = 10.5$

$d_2 = (M_2/K_t)^{1/3} = (11700x12/139x15)^{1/3} = 8.05$

$d_2 = P_2/v_t = 20400/120x16 = 10.65$

Use $d = 11 \text{ in.}$

$V_1 =$ vertical shear along IJ

$V_2 =$ " " " " IK

$V_1 = 0.125x9050(3 \text{ plus } 4)/2 = 3960\#$

$V_2 = (1/12)9050(2.25 \text{ plus } 3.25)/2 = 2075\#$

$v_1 = 3960x8/12x3x7x11 = 11.5 \text{ lbs. per sq.in.}$

$v_2 = 2075x8/12x2.5x7x11 = 8.0 \text{ lbs. per sq.in.}$

$A_s(\text{in B direction}) = M_2/f_{s,jd} = 11700x12x8/18000x7x11 = 0.779$

Distribute steel: 0.5(4 plus 2) plus 0.91 = 3.91 \text{ ft.}$

$A_s(\text{per foot}) = 0.779/3.91 = 0.193 \text{ sq.in.}$

$S(C) = V_2/u_{jd} = 2075x8/75x7x11 = 3.81 \text{ in.}$

$S(C) = 3.81/3.91 = 0.974 \text{ in. per ft.}$

Use $\frac{1}{2}\" \text{ round-12" OC.}$

$A_s(\text{in T direction}) = M_1/f_{s,jd} = 151000x12x8/18000x7x11 = 1.045$

Distribute steel: 0.5(3.25 plus 1.25) plus 0.91 = 3.11 \text{ ft.}$

$A_s(\text{per foot}) = 1.045/3.11 = 0.336$

$S(C) = V_1/u_{jd} = 3960x3/75x7x11 = 5.48 \text{ in.}$

$S(C) = 5.48/3.11 = 1.765 \text{ in. per foot}$

Use $\frac{3}{4}\" \text{ round-12" OC.}$
Part III

ECONOMICAL COMPARISON OF

REINFORCED CONCRETE TYPE WITH ENCASED STEEL TYPE

I. In this comparison of cost, the unit on which the cost is based is the span of thirty feet consisting of two bents with corresponding beams spanning the interval between bents. The bent is composed of two exterior columns and footings, two interior columns and footings, and the girder supported by these columns. The beams which span the distance between bents are: the two exterior beams and the seven interior beams. The two exterior beams support railings as shown on the accompanying plans.

II. The determination of the cost of these two type bridges is based upon the following assumption:

1. Mixture of concrete
   a. Railing: 1-1\(\frac{1}{2}\)-3
   b. Remaining parts of structure: 1-2-4

2. Prices
   a. Gravel delivered per ton $2.50
   b. Sand " " " 2.25
   c. Cement " " bbl. 1.95
   d. Lumber " " M 40.00
   e. Steel shapes delivered per 100# 3.50
   f. " bars " " " 3.25
   g. Labor (common) per day of 10 hrs. 2.00
   h. " (skilled) " " " " 3.50
   i. Carpenters " " " 5.00
   j. Superintendent " " " 6.00
### III. Quantities; reinforced type.

<table>
<thead>
<tr>
<th>Member</th>
<th>Concrete</th>
<th>Bars</th>
</tr>
</thead>
<tbody>
<tr>
<td>Railing</td>
<td>6.46 cu. yds.</td>
<td>206.7#</td>
</tr>
<tr>
<td>Slab</td>
<td>18.12</td>
<td>4603.9</td>
</tr>
<tr>
<td>Interior Beam stems</td>
<td>8.92</td>
<td>6806.3</td>
</tr>
<tr>
<td>Exterior Beams</td>
<td>9.16</td>
<td>1414.6</td>
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<tr>
<td>Girders</td>
<td>13.08</td>
<td>2339.5</td>
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<tr>
<td>Exterior Columns</td>
<td>12.35</td>
<td>5578.0</td>
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<tr>
<td>Interior Columns</td>
<td>6.58</td>
<td>4621.4</td>
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<tr>
<td>Exterior Footings</td>
<td>8.22</td>
<td>527.0</td>
</tr>
<tr>
<td>Interior Footings</td>
<td>3.89</td>
<td>196.6</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>6.46</td>
<td>25794.0</td>
</tr>
</tbody>
</table>

### Quantities; encased steel type.

<table>
<thead>
<tr>
<th>Member</th>
<th>Concrete</th>
<th>Bars</th>
<th>Shapes</th>
</tr>
</thead>
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<tr>
<td>Railing</td>
<td>6.46 cu. yds.</td>
<td>206.7#</td>
<td></td>
</tr>
<tr>
<td>Slab</td>
<td>18.12</td>
<td>4603.9</td>
<td></td>
</tr>
<tr>
<td>Interior Beam stems</td>
<td>8.92</td>
<td>18100.0#</td>
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<tr>
<td>Exterior Beams</td>
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<td>3485.0</td>
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<td>Girders</td>
<td>13.95</td>
<td>5175.0</td>
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<tr>
<td>Exterior Columns</td>
<td>9.84</td>
<td>2512.4</td>
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<tr>
<td>Interior Columns</td>
<td>7.10</td>
<td>1184.6</td>
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</tr>
<tr>
<td>Exterior Footings</td>
<td>4.70</td>
<td>398.4</td>
<td></td>
</tr>
<tr>
<td>Interior Footings</td>
<td>2.30</td>
<td>136.9</td>
<td></td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>6.46</td>
<td>9042.9</td>
<td>26760.0</td>
</tr>
</tbody>
</table>
IV. Unit cost analysis.

1. Concrete (1-1\(\frac{1}{2}\)-3)  
   Per Cu.Yd.  
   - Cement  
     1.9 bbls. @ $2.00 = $3.80  
   - Sand  
     0.44 cu. yds. @ 2.80 = 1.25  
   - Gravel  
     0.84 cu. yds. @ 3.40 = 2.85  
   - Mixing and Placing  
     3.10  
   - Forms  
     5.00  
   - Misc. Expense  
     .50  
   - Profit  
     3.30  
   - Total per cu. yd.  
     $19.80

2. Concrete (1-2-4)  
   - Cement  
     1.3 bbls. @ $2.00 = $2.60  
   - Sand  
     0.43 cu. yd. @ 2.80 = 1.20  
   - Gravel  
     0.86 cu. yd. @ 3.40 = 2.90  
   - Mixing and Placing  
     2.80  
   - Forms  
     2.75  
   - Misc. Expense  
     .50  
   - Profit  
     2.75  
   - Total per cu. yd.  
     $15.50

3. Steel Bars  
   Per lb.  
   - Bars delivered  
     $0.0325  
   - Bending and Placing  
     0.0160  
   - Profit  
     0.0115  
   - Total per lb.  
     $0.0600

4. Structural Sections  
   Per lb.  
   - Struct. sections delivered  
     $0.0350  
   - Erecting  
     0.0125  
   - Profit  
     0.0105  
   - Total per lb.  
     $0.0580
V. Total Cost of Bridges.

1. Reinforced Type.

RAILING
6.46 cu.yd. concrete  @ $19.80 = $127.91
2067 lbs. bars  @ 0.06 = 124.00

SLAB
13.12 cu.yd. concrete  @ 15.50 = 280.86
4603.9 lbs. bars  @ 0.06 = 276.23

INT. BEAM STEMS
8.92 cu.yd. concrete  @ 15.50 = 138.26
6806.3 lbs. bars  @ 0.06 = 408.39

EXT. BEAMS
9.16 cu.yd. concrete  @ 15.50 = 141.98
1414.6 lbs. bars  @ 0.06 = 84.88

GIRDERS
13.08 cu.yd. concrete  @ 15.50 = 202.74
2839.5 lbs bars  @ 0.06 = 170.37

EXT. COLUMNS
12.35 cu.yd. concrete  @ 15.50 = 191.43
5578 lbs bars  @ 0.06 = 334.68

INT. COLUMNS
6.58 cu.yd. concrete  @ 15.50 = 101.99
4621.4 lbs. bars  @ 0.06 = 277.28

EXT. FOOTINGS
6.22 cu.yd. concrete  @ 15.50 = 96.41
527 lbs. bars  @ 0.06 = 31.62

INT. FOOTINGS
3.89 cu.yd. concrete  @ 15.50 = 60.30
196.6 lbs. bars  @ 0.06 = 11.80

Total  $ 2949.72
2. **Encased Steel Type**

**RAILING**
- 6.46 cu.yd. concrete @ $19.80 = $127.91
- 206.7 lbs. bars @ 0.06 = 12.40

**SLAB**
- 18.12 cu.yd. concrete @ 15.50 = 280.86
- 4603.9 lbs. bars @ 0.06 = 276.23

**INT. BEAMS**
- 8.92 cu.yd. concrete @ 15.50 = 138.26
- 18100 lbs. str. sec. @ 0.058 = 1049.30

**EXT. BEAMS**
- 11.85 cu. yd. concrete @ 15.50 = 183.57
- 3485 lbs. str. sec. @ 0.058 = 202.13

**GIRDERS**
- 13.95 cu. yd. concrete @ 15.50 = 216.23
- 5175 lbs. str. sec. @ 0.058 = 300.15

**EXT. COLUMNS**
- 9.84 cu.yd. concrete @ 15.50 = 152.52
- 2512.4 lbs. bars @ 0.06 = 150.74

**INT. COLUMNS**
- 7.10 cu.yd. concrete @ 15.50 = 110.05
- 1184.6 lbs. bars @ 0.06 = 71.08

**EXT. FOOTINGS**
- 4.70 cu.yd. concrete @ 15.50 = 72.85
- 398.4 lbs. bars @ 0.06 = 23.90

**INT. FOOTINGS**
- 2.30 cu.yd. concrete @ 15.50 = 35.65
- 136.9 lbs. bars @ 0.06 = 8.21

**Total** $3412.74
VI. Conclusions.

The cost of these two bridges show that the encased steel type is less economical than the reinforced concrete type. This is true because of the waste of steel in the encased steel type, for the steel cannot be distributed to the best advantage as can be done with single bars. The difference in cost is approximately $500, for the single span of thirty feet. For longer spans or for bridges of several spans, this difference certainly makes the steel encased type uneconomical.

In this cost analysis no attempt has been made to include the cost of the abutments at the end of the bridge, or the cost of any excavation. The addition of these costs would affect both bridges equally, and therefore would not change the difference in cost. For this reason only those parts have been designed and compared that show differences due to their respective type of reinforcing.
"T" BEAMS  Neutral Axis in Web

Values of \( p \)

\( N = 10 \)

\[
\frac{t}{d} = R_2 - \sqrt{R_2^2 - \frac{12 f_c}{f_s}} R_1
\]

\[
P = \frac{t f_c}{d f_s} - \left( \frac{t}{d} \right)^2 \left( \frac{f_c}{f_s} + \frac{1}{n} \right)
\]

Values of \( p \)

\( N = 12 \)

\[
R_1 = \frac{f_c}{f_s} + \frac{1}{n}
\]

\[
R_2 = 3 \left( \frac{M}{f_s b t^2} + \frac{f_c}{f_s} + \frac{1}{2n} \right)
\]

Values of \( p \)

\( N = 15 \)

DIAGRAM 2.
**STIRRUP SPACING.**

\[ V_c = V - V_s \]

\[ s = \frac{A_s f_s d}{V_s} \]

Max Spacing

\[ s = \frac{45}{e + 10} d = 0.45d. \]

Stirrups must be hooked.

\[ f_s = 18000, \ \frac{1}{2}'' \Phi U \]

\[ f_s = 16000, \ \frac{3}{4}'' \Phi U \]

\[ f_s = 16000, \ \frac{1}{2}'' \Phi U \]
Rectangular Eccentric Columns
Compression over the Entire Section.
RECTANGULAR ECCENTRIC COLUMNS
Tension over Part of the Section.

\[ \frac{d'}{t} = 0.10 \]
\[ f_s = nE \left[ \frac{1}{k} - 1 \right] \]