THE DESIGN
and
ECONOMICAL COMPARISON
of
REINFORCED CONCRETE and ENCASED STEEL
HIGHWAY BRIDGES
A Thesis
Presented to The Advanced Degree Committee
of the
GEORGIA SCHOOL OF TECHNOLOGY
by
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In part fulfillment
of the requirements
for the degree of
MASTER OF SCIENCE
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APPROVED
By the Sub-Committee
of the
ADVANCED DEGREE COMMITTEE
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SYMBOLS

\( f_c \) = Allowable unit tensile stress in steel (Lbs. per Sq. In.)

\( f_s \) = Allowable unit compressive stress in concrete (Lbs. per square inch).

\( n \) = Modulus of elasticity of steel divided by the modulus of elasticity of concrete.

\( p \) = Percent of steel in tension = \( \frac{A_s}{b \cdot d} \).

\( M \) = External bending moment in inch pounds.

\( b \) = Breadth of beam in inches.

\( d \) = Effective depth of beam in inches (top of beam to center line of tensile steel).

\( d' \) = Fireproofing = Distance from center line of tensile steel to bottom of beam.

\( v \) = Allowable shearing stress in concrete (Lbs. per Sq. In.)

\( V \) = External shear that beam must carry (pounds).

\( V_c \) = Shear that concrete will carry (pounds).

\( V_s \) = Shear steel must carry if web reinforcement is used (Lbs.).

\( u \) = Allowable bond stress between steel and concrete (Lbs. per square inch of steel surface).

Summation \( S \) = Total perimeter of tension steel in inches.

\( L \) = Length of tension bars in inches required to prevent slip.

\( f_c' \) = Crushing strength in pounds per square inch of concrete tested in 6"x12" cylinders after a set of 28 days.
ALLOWABLE STRESSES

This design will be made on data given by the Joint Committee Specifications, the following being used:

\[ f_s = 18,000 \text{ Lbs. per Sq. In. (Intermediate Grade)} \]
\[ f_c = 0.4f_c' \] except over supports where 0.45f_c' is allowed.

\[ n = 10 \text{ if } f_c' \text{ is over 2900 Lbs. per Sq. In.} \]
\[ 12 \text{ if } f_c' \text{ is over 2200 and under 2900 Lbs. Sq. In.} \]
\[ 15 \text{ if } f_c' \text{ is over 1500 and under 2200 Lbs. Sq. In.} \]
\[ v = \]
\[ \text{No web} \quad \text{Web} \]
\[ \text{Reinf.} \quad \text{Reinf.} \]
\[ \text{Longitudinal bars not} \]
\[ \text{specially anchored} \quad 0.02f_c' \quad 0.06f_c' \]
\[ \text{Longitudinal bars} \]
\[ \text{specially anchored} \quad 0.03f_c' \quad 0.12f_c' \]
\[ a = \text{For plain bars not more than } 0.04f_c'. \]
\[ \text{For deformed bars not more than } 0.05f_c'. \]

It is assumed that the concrete as mixed on the job will equal or exceed 2000 Lbs. per square inch compressive strength so \( f_c' \) will be used as 2000.

GENERAL DATA

This bridge will consist of three spans, each 33' long with paved roadway 28' 6" wide. It will be designed for a 20 ton truck with two front wheels of 2 tons each and five feet apart and two rear wheels of 8 tons each and five feet apart. The line of front wheels will be considered as being 14 feet from the line of rear wheels. The beams of the bridge must clear the railroad tracks by at least 22 feet.
FLOOR DESIGN

To obtain maximum moment in floor consider one rear wheel of truck placed half way between two floor supports with axle perpendicular to supports. (see sketch).

\[ W = \text{Effective width} = \frac{4}{3} x \text{ where } x = \frac{1}{2} \text{ span} \]

\[ W = \frac{4}{3}(5/2) = 3.33' \text{ which may be used since it does not exceed 6'}. \]

Considering a 12" strip of floor perpendicular to supports we have for loading:

Concentrated load = \( Q' = \frac{16,000}{3.33} = 4,800 \text{ Lbs.} \)

Uniform load: Assuming \( d = 10" \)

\[ \frac{d + d'(150)}{12} = \frac{10 + 2(150)}{12} = 150 \text{ Lbs.} \]

Assuming pavement to weigh \( 20 \text{ Lbs.} \)

Total uniform load = \( 170 \text{ Lbs.} \)

Max. Pos. Mom. = \( 0.8Q' + \frac{1}{24}(\text{span})^2(170) \)

\[ = (0.80)(4800) + 1/24(170)(5)^2 = 4990 \text{ Ft. Lbs.} \]

Max. Neg. Mom. = \( 0.75Q' + \frac{1}{12}(\text{span})^2(170) \)

\[ = (0.75)(4800) + 1/12(170)(5)^2 = 4000 \text{ Ft. Lbs.} \]

To obtain maximum shear assume same wheel just above edge of support. Effective width for shear \( (W') \) use above method with \( x = 2.5d \) where \( d \) is the effective depth.

\[ W' = (4/3)(2.5)(10/12) = 2.75' \text{ Using 10'' eff. depth.} \]

Then \( Q'' = \frac{16,000}{2.75} = 5900 \text{ Lbs. for concentrated load.} \)

And uniform load = \( 170 \text{ Lbs. per foot as above.} \)
FLOOR DESIGN (Continued)

Shear at edge:
\[ V_s = \frac{Q}{2} + \frac{1}{2} \text{span}(170) = 5900 + \frac{(170)(5)}{2} = 6325 \text{ Lbs.} \]

Shear at center:
\[ V_c = \frac{Q}{2} = \frac{1}{2}(5900) = 2950 \text{ Lbs.} \]

Depth of floor slab for moment:
\[ bd^2 = \frac{M}{K} \]

We have assumed \( b = 12" \)
\[ K = \text{constant (see J. C. S.)} \]
\[ = 139 \text{ at centerline.} \]
\[ = 165 \text{ over supports.} \]

At C. L. \( M = 4000 \text{ Ft. Lbs.} \)
\[
\text{then } d^2 = \frac{4000 \times 12}{139 \times 12} = 28.8 \quad d = 5.4" \\
\overline{\text{Over support } M = 4000 \text{ Ft. Lbs.}}
\[
\text{then } d^2 = \frac{4000 \times 12}{165 \times 12} = 24.2 \quad d = 4.9" \\
\]

Depth of floor slab for shear:
\[ bd = \frac{V}{V_j} \]
\[ j \text{ being that part of the effective depth that may be used as mom. arm.} \]
\[ v = 0.03 \times 2000 = 60 \text{ Lbs. Sq. In.} \]
\[ bd = \frac{V}{60(7/8)} \]
\[ d = \frac{V}{(60)(12)(7/8)} = \frac{V}{630} \]

At C. L. \( d = 2950/630 = 4.7" \)

At support \( d = 6325/630 = 10" \)

Since shear at support governs use \( d = 10" \) then \( d + d' = 12" \)

Area of steel required for moment:
\[ A_s = \frac{M}{f_s j d} = \frac{M}{18000(7/8)(10)} = 0.0000635 \text{ M} \]

At C. L. \( A_s = 0.0000635 \times 4000 = 0.254 \text{ Sq. In. (bottom)} \)

At support \( A_s = 0.0000635 \times 4000 = 0.254 \text{ Sq. In. (top)} \)

J. C. S. for special anchorage in continuous beams requires \( 7/12 \) of steel at C. L. to be carried over support in bottom of beam. \( 7/12 \times 0.254 = 14.8 \text{ Sq. In.} \)
FLOOR DESIGN (Continued)

Perimeter of steel for bond:

\[
\text{Summation } \sigma = \frac{V}{\text{ujd}} = \frac{V}{(60)(7/8)(10)} = 0.001143V
\]

At center line:

\[
\text{Sum. } \sigma = 0.001143 \times 2950 = 3.37 \text{ in. (In bottom)}
\]

At support:

\[
\text{Sum. } \sigma = 0.001143 \times 6325 = 7.23 \text{ in. (In top)}
\]

Since more steel is required for shear than for moment we will select enough for shear. Use 1-\(\frac{3}{8}\)" bar in bottom continuous over support in bottom. Use 2-\(\frac{3}{8}\)" round bars in bottom at center, bent up thru quarter points and carried over supports at top.

FLOOR BEAM DESIGN

Since there will be an expansion joint at the ends of beams they will be simple beams.

Each interior beam will carry 5 feet of floor plus one side of a truck.

Each exterior beam will carry 2\(\frac{1}{2}\) feet of floor plus the hand rail and one side of a truck.

Interior beams:

These beams may be designed as "T" beams or as rectangular beams. The former is the more economical and will therefore be used.

Uniform load:

\[
\begin{align*}
\text{Floor} &= (5)(1)(150) = 750 \text{ Lbs.} \\
\text{Pavement (assume)} &= 100 \text{ Lbs.} \\
\text{Beam (below floor)} &= 150 \text{ Lbs.} \\
\text{Total} &= 1000 \text{ Lbs.}
\end{align*}
\]
Computation of shears and moments:

From the above sketch we can see that

\[ R = (P + P') \left[ I - \frac{(z + 2.8)}{L} \right] + 1000 \frac{L}{2} \]

\[ R = (16,000 + 4,000) \left[ I - \frac{z + 2.8}{33} \right] + (1000)(16.5) \]

\[ = 34,800 - 606 z \]

\[ M = R z - (1000 z)(\frac{z}{2}) = R z - 500 z^2 \]

From the above formula we compute the following values of R & M

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<tr>
<th>z</th>
<th>R</th>
<th>M</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>34,800 Lbs.</td>
<td>0</td>
</tr>
<tr>
<td>2.5'</td>
<td>33,285</td>
<td>80,087 Ft. Lbs.</td>
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<tr>
<td>5</td>
<td>31,766</td>
<td>146,325</td>
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<tr>
<td>7.5'</td>
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</tr>
<tr>
<td>15</td>
<td>25,700</td>
<td>273,000</td>
</tr>
<tr>
<td>16.5'</td>
<td>24,800</td>
<td>272,900</td>
</tr>
</tbody>
</table>
At centerline:

\[ M = 273,000 \text{ Maximum moment being used.} \]

\[ \frac{M}{f_{sb} t^2} = \frac{273,000 \times 12}{18,000 \times 60 \times 144} = 0.0211 \]

\[ f_c/f_s = \frac{800}{18,000} = 0.045 \]

By use of tables \( t/d \) is more than \( k (k=.4) \) so the neutral axis is in the flange and the beam is designed as if it were a rectangular section at the center.

\[ bd^2 = \frac{273,000 \times 12}{138.7} \quad d^2 = \frac{273,000 \times 12}{60 \times 138.7} = 394 \]

\[ d = 19.9'' \]

Since \( t = 12'' \) (floor thickness)

\[ t/d = 12/19.9 = 0.6 \text{ which is greater than } k \text{ so the neutral axis is in the flange.} \]

At support:

\[ bd = \sqrt{\frac{V}{VJ}} = \sqrt{\frac{34,800}{120 \times 7/8}} = 332 \text{ which must be minimum area of web. Therefore } 16'' \times 21'' \text{ will be most economical dimension. } d + d' = 23'' \quad t/d = 12/21 = 0.57 \text{ which is greater than } 0.4 \text{ so N.A. is in flange.} \]

Steel required for moment:

\[ A_s = \frac{273,000 \times 12}{18,000 \times 7/8 \times 21} = 9.90'' \text{ Use } 8-13/8'' \text{ square bars.} \]

Steel will have to be put in two layers and will require \( 3 \times \frac{13}{8}'' \times 3 + \frac{13}{8}'' + 3 = 14\frac{1}{2}'' \text{ which is O.K.} \)

Since none of this steel will be needed in top of beam at the supports it will be cut off where not needed for moment in bottom and excess shear will be carried by stirrups. See moment diagram.
Moment & Shear Diagrams for the Interior Beams

39,000 Ft. Lbs.

29,000 Ft. Lbs.

19,000 Ft. Lbs.

Cutting Bars

1 Bar

2 Bars

3 Bars

4 Bars

5 Bars

6 Bars

Spacing of Stirrups
Stirrup Spacing - Floor Beams

Moment Diagram Floor Beams
FLOOR BEAM DESIGN (Continued)

Stirrups:

\[ V_c = vbjd = 40 \times 16 \times 7/8 \times 21 = 11,760 \text{ Lbs.} \]

At support \[ V_s = 34,800 - 11,760 = 23,040 \text{ Lbs.} \]

Maximum allowable spacing (J. G. S.) = \[ \frac{45}{a + 10} \]

where \( a \) = angle in degrees between stirrup and the horizontal

\[ \text{Spacing } s = \frac{45}{90 + 10} \times 21 = 9.5'' \]

Spacing will be determined by use of diagram shown.

Stirrups used to be \( \frac{1}{2}'' \) round bars.

J. S. G. requires that one fourth of shear at end of beam be assumed at center.

The spacing may be calculated by the following method:

\[ V_s = \frac{Asf_sjd}{s \sin(a)} \]

and since \( a = 90 \text{ degrees} \)

\[ \sin a = 1 \]

Then spacing \[ s = \frac{Asf_sjd}{V_s} = 2 \times 0.196 \times 16,000 \times 7/8 \times 21 = 23,040 \]

\[ s = 5.7'' \] and since spacing will start not at end of beam but at edge of girder where \( V_s \) will naturally be smaller \( 6'' \) will be used.

To find where the spacing may be changed to \( 8'' \):

\[ V_s = \frac{122,000}{8} = 15,250 \text{ Lbs.} \] and this point may be calculated or scaled from the diagram. It was scaled and found to be six feet from end of beam.

Placing first stirrup six inches from end and using thirteen spaces of six inches there will be left twelve spaces of eight inches. Therefore 26 stirrups will be needed in each end of each beam.
Exterior beams:

These beams are designed as simple rectangular beams.

**Uniform load:**
- **Floor** = $3 \times 1 \times 150 = 450$ Lbs.
- **Pavement (assume)** = 50
- **Hand rail (average)** = 250
- **Beam (assume)** = 250

**Total** = 1000 Lbs. per Ft.

This is same load per foot as for interior beams and since this beam will carry one row of truck wheels the moment and shear will be the same.

**Size for moment:**
\[
bd^2 = \frac{273,000 \times 12}{138.7} = 20,840 \text{ Ins.}^3
\]

Making $b = \frac{1}{3}d$, $d^2 = 41,680 \quad d = 34.5"$ use 35"

**Size for shear:** Use same size with stirrups for excess shear.

\[
V_s = 34,800 - 40 \times 17 \times \frac{7}{8} \times 35 = 14,000 \text{ Lbs.}
\]

\[
s = \frac{2 \times 0.196 \times 18,000 \times 7/8 \times 35}{14,000} = 21".
\]

**Maximum spacing using $\frac{3}{8}"$ round stirrups**

\[
s = \frac{45}{90 + \frac{10}{9}} = 15.8" \quad \text{Use 15"}
\]

Stirrups must be used thru 9' from each end, starting six inches from end. This is obtained by drawing the $V_s$ line on the shear diagram and scaling or calculating the distance from the end of the beam to the intersection on the curve. Seven spaces or eight stirrups are required.

**Steel for moment:**
\[
A_s = \frac{273,000 \times 12}{18,000 \times 7/8 \times 35} = 5.95 \text{ Sq. In.}
\]

Use four $1\frac{1}{2}"$ square bars. In this case one bar can be cut off at each point where two bars are cut off in the interior beams since one quarter of the total steel area is cut in each case and the moment is the same.
GIRDER DESIGN

Girders will be supported by three columns spaced ten feet on center and will overhang the outer columns about five and one half feet.

Loading:

Assume size of girder $24'' \times 30''$

Weight of girder $2 \times 2\frac{1}{2} \times 150 = 750 \text{ Lbs. per foot.}$

Floor $= 1 \times 5 \times 33 \times 150 = 24,750$

Stem $= 16/12 \times 11/12 \times 33 \times 150 = 6050$

Pavement $= 5 \times 33 \times 20 = 3300$

Dead load (Int. beams) $34,100 \text{ Lbs. per beam.}$

For exterior beams:

Floor $= 1 \times 2 \times 33 \times 150 = 9,900$

Beam $= 17/12 \times 35/12 \times 33 \times 150 = 20,500$

Pavement $= 2 \times 33 \times 20 = 1,320$

Hand rail $= 33 \times 300 = 9,900$

Dead load (Ext. beams) $= 31,620 \text{ Lbs. per beam.}$

Truck loadings will be considered as being concentrated on wheels as already shown, and distance from front wheels of one truck to rear wheels of next truck ahead taken as 19 feet (J.C.S.). Since front and rear wheels are 14 feet apart the total distance from one set of rear wheels to rear wheels of truck ahead is 33', which is the same as the span, then maximum loading on a girder will occur when a set of rear wheels is over the girder and each beam load on the girder (in addition to dead load) will be: $P_1 + \frac{14}{33} P_2 + \frac{19}{33} P_2 = 16000 + 4000 = 20,000 \text{ Lbs.}$
These loads occur in pairs five feet apart and a space of five feet will be assumed between trucks parallel on the same section of the floor, therefore each beam when loaded to its maximum will carry one side of one truck.

To obtain the moment and shear in the girders the slope deflection method will be used.

Assuming girders to be 24" x 30" and columns to be 24" x 24"

\[
\frac{EI}{L} = \frac{I/12 \cdot 2 \cdot (2)^3}{20} = 0.067 \text{ for columns.}
\]

\[
\frac{EI}{L} = \frac{I/12 \cdot 2 \cdot (2.5)^3}{10} = 0.26 \text{ for girders.}
\]

Moments calculated on cantilever and simple beam basis:

Due to dead loads:

Cantilever overhangs:

\[31,600 \times 5 + 2{1/2} \times 5 \times 750 = 168,000 \text{ Ft. Lbs.}\]

Continuous spans:

\[
\frac{34,100 \times 8 \times 750 \times (8)^2}{12} = 72,200 \text{ Ft. Lbs.}
\]

Due to truck loads:

Cantilever overhangs:

\[20,000 \times 4 = 80,000 \text{ Ft. Lbs.}\]

Interior spans:

\[
\frac{20,000 \times 8}{8} = 20,000 \text{ Ft. Lbs.}
\]
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|        | 0    | 0   | 0     | 0    | 0   | 0     | 0    | 0    |

|        | 0    | 0   | 0     | 0    | 0   | 0     | 0    | 0    |

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|        | 0    | 0   |       | 0    | 0   |       | 0    | 0    |

|        | -38.5 | 38.5 |       | -38.5 | 38.5 |       | 0    | 0    |

|        | 0    | 0   |       | 0    | 0   |       | 0    | 0    |

|        | -149 | 149 |       | -149 | 149 |       | 0    | 0    |

MOMENT DUE TO UNIFORM LOAD (weight of beam)
and MOMENT DUE TO CONCENTRATED LOADS
(dead weight of floor etc.)
MOMENTS DUE TO CONCENTRATED WHEEL LOADS
PLACED AS SHOWN

-80  0  0  20  -20
64   -8.9  -8.9  16
-4.5  32    8.0  -4.5
-18.3  -17.8  3.6
-8.9  1.8   1.8
-0.8  3.6   3.6
0.8   -3.2  -3.2
61.1  5.9   1.9
-6.9

-0.5
-0.6
-0.7
-0.8
-0.9
-1.0
-1.1
-1.2
-1.3
-1.4
-1.5
-1.6
-1.7
-1.8
-1.9
-2.0
MOMENTS DUE TO CONCENTRATED WHEEL LOADS PLACED AS SHOWN
MOMENTS DUE TO CONCENTRATED WHEEL LOADS
PLACED AS SHOWN

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<td>-19.8</td>
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<td>-59.7</td>
<td>-39.7</td>
<td>-19.7</td>
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</tr>
<tr>
<td>4</td>
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<td>-59.6</td>
<td>-39.6</td>
<td>-19.6</td>
<td>0</td>
</tr>
</tbody>
</table>

Diagram showing moments due to concentrated wheel loads.
## Summation of Moments in Girder

<table>
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<tr>
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<th>149</th>
<th>-23.5</th>
<th>33.5</th>
<th>-149</th>
<th>80</th>
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<td>-69.5</td>
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<tr>
<td></td>
<td>-248</td>
<td>218.5</td>
<td>-13.7</td>
<td>13.7</td>
<td>-218.5</td>
<td>248</td>
</tr>
</tbody>
</table>

-61.5  61.5

-28    28
Moments:

Over 1st. support = 248,000 Ft. Lbs.
At C. L. of 1st span = \(-218,500 + 49,800 - \frac{750 \times 16}{2}\)
\[= -25,300 \text{ Ft. Lbs.}\]

Over second support = 61,500 Ft. Lbs.
At C. L. of 2nd span = \(-61,500 + 10,300 \times 4 - \frac{750 \times 16}{2}\)
\[= 26,300 \text{ Ft. Lbs.}\]

Since the girder will be symmetrical about the center line no other moments need to be calculated.

Shears:

At end = 31,600 Lbs. due to exterior beam.
At one foot from end = 31,600 + 20,000 = 51,600
At left of first support = 750 \times 5 + 51,600 = 55,370 Lbs.
At right of first support = 750 \times 4 + \frac{54,100 + 20,000}{2} + \frac{218,500 - 61,500}{8} = 49,800 Lbs.
At C. L. 1st span = 17,050 - 750 \times 4 + 19,625 = 33675 Lbs.
At left of second support = 49,800 Lbs. (same as other end of span)
At right of second support = 750 \times 4 + 17,050 + 10,000
\[-19,625 = 10,300\]

(This will be discarded and the girder designed symmetric about the center)

Solving for \(d\):

\(b = 24''\) assumed. \(bd^2 = \frac{M}{K}\)
Over first col.: \(bd^2 = \frac{248,000 \times 12}{165} = 18,000\) \(d = 27.4''\)
At C. L. span: \(bd^2 = \frac{26,300 \times 12}{139} = 2270\) \(d = 9.7''\)

Use \(d = 28''\) then \(d + d' = 30''\) as assumed.
GIRDER DESIGN (Continued.)

Steel for moment:  \[ A_s = \frac{M}{f_s J_d} = \frac{M}{18000 \times \frac{7}{8} \times 28} = 0.0000272M \]

Over 1st support = \( 248,000 \times 0.0000272 = 6.7 \text{ Sq. In.} \)

At C. L. span = \( 26,300 \times 0.0000272 = 0.72 \text{ Sq. In.} \)

Over center support = \( 61,500 \times 0.0000272 = 1.7 \text{ Sq. In.} \)

Length that bars must be carried beyond point of maximum moment before cutting = \( \frac{18,000 \times 9/8}{400} = 51'' \) for \( \frac{1}{2}'' \) bars. Since the length of the cantilevers is only 48" from edge of supports these bars will have to be bent. Since there is only 48" from the C. L. of spans to edges of supports the steel in the bottom will have to be carried from the C. L. of one outside column to C. L. of column at opposite end of girder.

See plans for complete steel details.

Steel for shear:  \[ \text{Perimeter} = \frac{V}{100 \times 7/8 \times 28} = 0.000408 V \]

At end:  \( V = 31,600 \)  Perimeter = 12.5"

At I' :  \( V = 51,600 \)  " = 21"

1st col.  \( V = 55,370 \)  " = 22.5"

C. L. span:  \( V = 33,675 \)  " = 13.5"

2nd Col.  \( V = 49,800 \)  " = 20.3"

In most cases the shear requirement for steel exceeded that for moment.

Stirrups:  Use \( \frac{1}{2}'' \) round "U" stirrups.  \( A_s = 2 \times 0.196 = .392 \text{ Sq. In.} \)

\( V_o \) (shear that the concrete will carry) = \( 40 \times 24 \times 30 \times 7/8 = 25,200 \text{ Lbs.} \)

At end of cantilever:  \( V_s = 51,600 - 25,200 = 26,400 \text{ Lbs.} \)

Spacing \( s = \frac{A_s f_s j_d}{V_s} = 0.392 \times 18000 \times 7/8 \times 28 \)

= 6.5"

At left of Ist Col.:  \( V_s = 55,370 - 25,200 = 30,170 \text{ Lbs.} \)

\[ s = \frac{0.392 \times 18,000 \times 7/8 \times 28}{30,170} = 5.7'' \]
GIRDER DESIGN (Continued)

Starting 6\text{\frac{1}{2}}\text{in.} inches from end use five spaces of 6\text{in.} and four spaces of 4\text{in.} in cantilever.

No stirrups will be needed over the columns.

At right of first column:

\[ V_s = 49,800 - 25,200 = 24,600 \text{ Lbs.} \quad s = 7\text{in.} \]

At C. L. span:

\[ V_s = 33,675 - 25,200 = 8475 \text{ Lbs.} \quad s = 20\text{in.} \]

But Max. \( s = \frac{45}{100}d = 12.5\text{in.} \)

For interior spans use two spaces 7\text{in.}; two 9\text{in.} and one 10\text{in.}.

This spacing starts at edge of column and leaves a 12\text{in.} space at the center.

COLUMN DESIGN

Special symbols:

- \( A \) = Cross section area of column.
- \( b \) and \( t \) = dimensions.
- \( A_s \) = Area of vertical steel.
- \( p \) = Percent of steel in a cross section area.
- \( A_c = A - A_s \) or cross section area of concrete.
- \( N \) = Maximum load parallel to long axis.
- \( h \) = Height of column in inches.
- \( R \) = Least radius of gyration.
- \( x_0 \) = Excentricity = \frac{M}{N}

J. C. S. requires that ties be not more than eight inches C. C., that \( p \) must be between .005 and .02, that fireproofing be not less than 2\text{in.} and that \( h/R \) be less than 40.

Exterior columns:

\[ N = \text{Sum of Max. shears on right and left plus beam load with} \]
\[ \text{truck plus weight of girder} = 49,800 + 55,370 + 54,100 + 750 \times 10 = 166,770 \text{ Lbs.} \]
COLUMN DESIGN (Continued)

\[ L_0 = \frac{37,900}{166,000} = 0.23' = 2.8'' \quad M = (19 + 18.9) \times 1000 = 37,900 \text{ Ft. Lbs.} \]

Since girders are 24" thick try columns 24" square.

Height = 20' = 240" \quad R = 24/\sqrt{E} = 6.95

\[ h/R = 240/6.95 = 34 \quad \text{which is less than 40 so O.K.} \]

Try \(p = 0.005\)

\[ \frac{d'}{t} = \frac{2}{24} = 0.09 \quad \frac{X_0}{t} = \frac{2.8}{24} = 0.117 \]

\[(n - 1)p = 14 \times 0.005 = 0.07\]

Knowing the above values the value of \(\frac{f_{bot}}{N}\) can be determined from tables based on J. C. S. and experiment.

\[ \frac{f_{bot}}{N} = 1.48 \quad f_c = \frac{1.48 \times 166,000}{24 \times 24} = 427 \text{ Lbs./Sq. In.} \quad \text{which is allowable.} \]

\[ A_s = .005 \times 24 \times 24 = 2.88 \text{ Sq. In.} \quad \text{Use 2-1" round bars in each face.} \]

Since all columns were assumed to have the same stiffness and since the interior columns will have a smaller loading than the exterior columns the interior columns will be identical with the above design.

FOOTING DESIGN

Loading:

\[ N = 166,000 + 2 \times 2 \times 20 \times 150 = 178,770 \text{ Lbs.} \]

\[ M = (9.5 + 9.4)1000 = 18,900 \text{ Ft. Lbs.} \]

\[ X_0 = \frac{18,900}{178,770} = 0.106' = 1.3'' \]

Values of \(b\) and \(t = 24''\)

Since \(X_0\) is less than \(1/6 \times t\) the footing will be in compression over the entire area.

The bearing will be on clay or possibly on rock. Assume the bearing of foundation to be 5 tons per square foot.

Assume weight of footing to be 3750 Lbs.

Minimum area = \(\frac{N + W}{2000 \times 5} = 18.25 \text{ Sq. Ft.} \quad B \text{ and } T = 4.3'\)
FOOTING DESIGN (Continued)

B = Dimension of footing at right angle to direction of girder.
T = Dimension of footing parallel to direction of girder.

The fact that the loading is eccentric will make the size of the girder somewhat larger than the minimum. Try 5' x 5' x 1' which will weigh 3750 pounds.

See sketch.

\[
y = \frac{N}{BT} + \frac{6hX_0}{BT^2} = \frac{178,770}{25} + \frac{6 \times 178,770 \times 0.106}{125} = 8,080 \text{ Lbs. per Sq. Ft.}
\]

\[
y_1 = \frac{178,770}{25} - \frac{6 \times 178,770 \times 0.106}{125} = 6,260 \text{ Lbs. per Sq. Ft.}
\]

\[
BT = \frac{N + W}{L} (I + \frac{6X_0}{T}) = \frac{178,770 + 3750}{10,000} (I + \frac{636}{5}) = 20.6 \text{ Sq. In.}
\]

B = T = 4.55' which is O.K. since less than 5'

The pressure will be a straight line variation from one B edge to the opposite. Therefore \( y_2 = 7534 \) and \( y_3 = 6806 \) pounds/Sq. Ft.

The upward thrust on \( y_2y = (\frac{y + 2y_2}{2})(\frac{B}{2})(\frac{T - t}{2}) = 29,590 \text{ Lbs.} \)

Thrust on \( y_2y_2' = (\frac{y + 2y_2}{2})(\frac{B}{2})(\frac{T - t}{2}) = 11,483 \text{ Lbs.} \)

Punching shear along \( b = P_b = 29,590 + 11,483 = 41,073 \text{ Lbs.} \)

Moment arm = \( \frac{3}{4}(\frac{T - t}{2}) = 1.125 \)

\( M_b = 41,073 \times 1.125 = 46,213 \text{ Ft. Lbs.} \)

\( d^2 = \frac{M_b}{K_d} = 166 \quad d = 12.9'' \) so 13" will be used.

\( d + b' = 15'' \) This extra weight due to increase in thickness of footing will not affect \( W \) appreciably.

\( A_s = \frac{M}{f_{A3}d} = \frac{46,213 \times 12}{18,000 \times 7/8 \times 13} = 2.7 \text{ Sq. In.} \) Use 3 - 1" Sq. bars

\( V = \frac{M}{M_d} = 46,297/(50 \times \frac{7}{8} \times 13) = 29 \text{ Lbs. per Sq. In.} \)

which is O.K. since 40# is allowed.
Since the distance from the floor beams to bed rock, at the ends of the bridge is only 7' (approximately), they will be supported by a gravity wall 18" wide at the top and 2.5' wide at 7' from the top. The height of this wall will depend on the amount of rock necessary to remove, at each section, in order to clear all loose and broken fragments.

This wall will also serve as a retaining wall for the fill at the ends of the bridge.

At the ends of the outside girders the gravity walls will make an angle of 45 degrees with the longitudinal axis of the bridge and slope down away from the bridge with a twenty percent slope. See diagram below.
## STEEL REQUIRED

### FLOOR

<table>
<thead>
<tr>
<th>Mark</th>
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<th>Length</th>
<th>Weight</th>
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<tbody>
<tr>
<td>F-1</td>
<td>198</td>
<td>1/8&quot; Square</td>
<td>31'-6&quot;</td>
<td>5300 Lbs.</td>
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<tr>
<td>F-2</td>
<td>198</td>
<td>1/8&quot; Round</td>
<td>33'-6&quot;</td>
<td>4425</td>
</tr>
<tr>
<td>F-3</td>
<td>99</td>
<td>1/8&quot; Round</td>
<td>31'-6&quot;</td>
<td>2075</td>
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### INTERIOR BEAMS

<table>
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<th>Size and Type</th>
<th>Length</th>
<th>Weight</th>
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</thead>
<tbody>
<tr>
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<tr>
<td>B-4</td>
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<td>1-1/8&quot; Square</td>
<td>32'-6&quot;</td>
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<td>B-5</td>
<td>750</td>
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<td>4'-6&quot;</td>
<td>2254</td>
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### EXTERIOR BEAMS

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</thead>
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<td>E-5</td>
<td>96</td>
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### HAND RAIL

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<td>143</td>
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<td>R-3</td>
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<td>2'-6&quot;</td>
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<td>R-4</td>
<td>54</td>
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<td>2'-6&quot;</td>
<td>51</td>
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### GIRDER

<table>
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</thead>
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<tr>
<td>G-1</td>
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<td>1-1/8&quot; Square</td>
<td>20'-0&quot;</td>
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<td>G-2</td>
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<td>34'-6&quot;</td>
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<tr>
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<td>92</td>
<td>3/8&quot; Round</td>
<td>6'-6&quot;</td>
<td>400</td>
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</table>

### COLUMNS AND FOOTINGS

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<th>Size and Type</th>
<th>Length</th>
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</tr>
</thead>
<tbody>
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<td>24'-0&quot;</td>
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<tr>
<td>G-2</td>
<td>210</td>
<td>1&quot; Round</td>
<td>7'-0&quot;</td>
<td>245</td>
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<tr>
<td>G-3</td>
<td>18</td>
<td>1&quot; Square</td>
<td>4'-8&quot;</td>
<td>285</td>
</tr>
</tbody>
</table>

TOTAL WEIGHT OF STEEL = 38,166 Pounds or 19.08 Tons
Chamfer 1/2 on ballusters & panels, all other 1/2.
Expansion joints in one end of each rail. A rail expansion joint must be at each deck expansion joint.
PART II

Design of Slab-Beam-Girder-Column Type Bridge by Encased Beam Method.

Symbols and allowable stresses identical to those in Part I

Additional data:

\[ K = 0.4 \]
\[ n = 15 \]
\[ f_s = 18,000 \text{ pounds per Sq. In.} \]
\[ a = \text{Thickness of concrete outside of steel.} \]
\[ h = \text{Depth of steel 'I' beam.} \]
\[ I = \text{Moment of inertia of compound section about N.A.} \]
\[ I_s = \text{Moment of inertia of steel section about N.A.} \]
\[ A_s = \text{Area of compressive steel.} \]
\[ A_s = \text{Area of steel section.} \]
\[ t = \text{Thickness of floor slab.} \]
\[ b' = \text{Thickness of concrete web.} \]
\[ d = h + a. \]

**INTERIOR BEAM DESIGN**

\[ a = 3'' \] Try 18'' I Beam 70# per Ft.  \( A = 20.46 \text{ Sq. In.} \)

\[ I_s = 917.5 \text{ In.}^4 \]

Flange width = 6.25"

\[ I = 917.5 + 20.46 (9 + 3 - .4 \times 21)^2 + \frac{60 \times (84)^3}{3.15} \]
\[ = 917.5 + 20.46 (12 - 6.4 x 21)^2 + 1648.5 \]
\[ = 917.5 + 20.46 (4.8)^2 + 1648.5 \]
\[ = 917.5 + 20.46 \times 23.04 + 1648.5 \]
\[ = 917.5 + 471.2 + 1648.5 \]
\[ = 2937.2 \]

\[ M = \frac{F_1}{I_s} = \frac{2,020,000}{(1-k)d} \]
\[ 2,020,000 = 18,000 \times 1648.5 \times \frac{.6 \times 21}{d} \]
\[ = 2,200,000 \]

This is O.K. Therefore the 18'' Beam will be used.
CONCRETE ENCASED STEEL BEAMS

RECTANGULAR TYPE

\[ h = \text{height of steel beam in inches} \]
\[ a = \text{thickness of concrete above top edge of steel in inches} \]
\[ A_s = \text{Area of steel section in square inches} \]
\[ I_s = \text{moment of inertia of steel section about its C.ofG. inches}^4 \]

\[ I = I_s + A_s \left( \frac{h}{2} + a - kd \right)^2 + \frac{h(a - kd)^3}{3n} \]

\[ b = \frac{2nA_s}{(kd)^2} \left( \frac{h}{2} + a - kd \right) \]

\[ k = \frac{1}{1 + \frac{I_s}{I_c}} \]
INTERIOR BEAM DESIGN (CONTINUED)

Width of stem = 6.25" + 4" = 10.25" Therefore 11" will be used

\[ \frac{1}{8} \times W L^2 = 2,020,000 = \frac{1}{8} \times W (33)^2 \quad W = 2300\# \]

\[ V = \frac{1}{6} W L = 0.5 \times 2300 \times 33 = 37,900\# \]

\[ V = \frac{3 \times 37,900}{15 \times 1648} = \frac{8.4 - 3/2}{3} = 36 \quad \text{And since it is less than 40 it may be used.} \]

EXTERIOR BEAM DESIGN

\[ a = 8" \quad \text{Try 20" I Beam 65.4\# per Ft.} \quad \text{Assume } D = 32" \]

\[ 12 \times 168,000 = \frac{18,000}{.4 \times 32} (1169.5 + 19.08(10 + 8 - .4(32)^2)) \]

\[ 2,020,000 = 2,080,000 \quad \text{Therefore the above beam will be used} \]

\[ b = \frac{2 \times 15 \times 19.08}{(.4(32))^2} = 23" \]

DESIGN OF GIRDER

Assume \( d = 22" \quad \text{Try 18" Bethlehem "G" I Beam } a = 4" \)

\[ 12 \times 248,000 = \frac{18,000}{.6 \times 22} (I_s + A_s(h/2 + a - Kd))^2 + \frac{2nA_s}{(Kd)^2} \frac{(h/2 + a - Kd)Kd}{3n} \]

\[ 2,980,000 = \frac{18,000}{.6 \times 22} (1380+23.6(9+4-8)^2 + 2/3A_sKd(h/2+a-Kd)) \]

\[ = \frac{18,000}{.6 \times 22} (1380+23.6 \times 9 + 2/3 \times 23.6 \times 8 \times 3) \]

\[ = 2,960,000 \quad \text{Therefore the beam may be used.} \]

\[ B = \frac{2nA_s}{(Kd)^2}(h/2 + a - Kd) = \frac{2 \times 15 \times 23.6}{(.4 \times 22)^2} (9+4+4 \times 24) = 22" \]
COMPARISON OF COST OF INCASED "I" BEAM AND REINFORCED
CONCRETE FOR GIRDERS AND "T" FLOOR BEAMS.

The following cost data obtained from Gilletts Handbook
on Cost Data and from the Georgia State Highway Department.

REINFORCED CONCRETE DESIGN

Howe Truss or equivalent design must be used on center
span so as not to interfere with traffic during construction.

Cost of Howe Truss per linear foot is $37.50 including
erection costs.

Three Howe Trusses required - placed as shown in diagram.
Assuming the life of the trusses to be six installations we will
use 1/6 of cost new for the job.

Cost of Howe Trusses - 3 required, each 33' long
\[ 3 \times 33 \times \$37.50 \times \frac{1}{6} \approx \$620.00 \]

Assuming timber suitable for columns to be $30/M
Cost of 4" x 6" columns for center span =
\[ \frac{30 \times (4 \times 6)}{1000 \times \frac{12}{12} \times (34) \times \frac{23}{4}} \approx 16.90 \]

Cost of column material for outside spans
\[ 2 \times 16.90 \approx 33.80 \]

Cost of 4" x 4" Column timber for outside span
at $30/M (Columns 5' apart)
\[ \frac{.020 \times 4 \times 4 \times 7 \times 7 \times 20 \times 2}{12} \approx 26.16 \]

Cost of forms for exterior beams, interior beams,
floor. Based on flat rate of .25 per Sq. Ft.

Total area:
\[ 2 \left( \frac{26+18+26}{12} \right) 99 + 5 \left( \frac{16+21+21}{12} \right) 99 + \left( \frac{30-2x18-5x16}{12} \right) 99 \]
\[ \approx 6960 \text{ Square Feet of forms at } \$0.25 \text{ Per S.F.} \]
\[ \approx 1740.00 \]

Cost of forms for girders and columns.
Total area:
\[ 2x6x20 \left( \frac{26+26}{12} \right) + (30x26x32) \approx 400 \text{ Sq.Ft.} \]
\[ 400 \times .25 = 400.00 \]

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REINFORCED CONCRETE DESIGN (CONTINUED)

Cost of concrete assuming concrete in place to cost $25.00 per cubic yard.
Volume:
\[1 \times 30 \times 99 + 3 \times 1\frac{1}{2} \times 2 \times 99 + 2 \times 1\frac{1}{2} \times 5 \times 99 + 2 \times 2 \times 2 	imes 30 + 2 \times 2 \times 6 \times 20 = 62,928 \text{ Cu.Ft.} = 304 \text{ Cu.Yds.} \]

\[304 \times 25 = 7600.00 \]

Cost of reinforcing steel assuming $0.06 per pound in place.

\[38,166 \times .06 = 2283.96 \]

Total for reinforced concrete design = $12,330.82

ENCASED "I" BEAM DESIGN - COSTS

If encased I Beam construction is used, forms are fastened to the beams thus eliminating need for Howe Truss.

The following costs include erection costs.

- Cost of 4"x 6" timber - center span 16.90
- Cost of 4"x 6" timber - outside spans 33.80
- Cost of forms for floors 1740.00
- Cost of Girders and columns (forms) 100.00
- Cost of concrete (in place) 7410.00
- Cost of steel for floor: 12,860 Lbs. x .06 = 771.60

Cost of beams (steel I beams)

2 - 18" Beams 80# per Ft., 30' Long )
15 - 15" Beams 70# per Ft., 33' Long )$0.04 per
6 - 20" Beams 65.4# per Ft. 33'Long ) Pound 2095.00

Total for encased beam design $12,167.30

COMPARISON OF COSTS

$12,330.82 - 12,167.30 = $163.52 difference in favor of encased I beams.
The comparison of costs shows that the encased I beam type costs less by $163.52. The main reason for this saving is that by using encased beam construction there is no need for the Howe Truss, the forms being tied to the I beams. The fact that more time is required to tie and place reinforcing bars than to place the I beams is also a factor in favor of the encased beams. The time saved by using I beam construction on this bridge would be twelve to sixteen days less than that required if reinforced concrete construction were used.
### Table Number 2: Areas and summations of perimeters also weights of bars.

#### Summation of areas = A. Summation of perimeters = 0.

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<thead>
<tr>
<th>Bar Size</th>
<th>Wt. Lbs.</th>
<th>Number of bars</th>
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</thead>
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<td>3/8&quot; 0</td>
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<td>1</td>
</tr>
<tr>
<td>1/2&quot; 0</td>
<td>0.633</td>
<td>2</td>
</tr>
<tr>
<td>1/2&quot; S</td>
<td>0.850</td>
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<tr>
<td>5/16&quot; 0</td>
<td>1.043</td>
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<td>1.502</td>
<td>5</td>
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<td>2.044</td>
<td>6</td>
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<td>1&quot; 0</td>
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<tr>
<td>1 1/2&quot; S</td>
<td>4.303</td>
<td>9</td>
</tr>
<tr>
<td>1 1/4&quot; S</td>
<td>5.313</td>
<td>10</td>
</tr>
</tbody>
</table>

#### Areas and summations of perimeters for various spacings.

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Spacing of bars in inches</th>
</tr>
</thead>
<tbody>
<tr>
<td>3/8&quot; 0</td>
<td>3&quot; 4&quot; 5&quot; 6&quot; 7&quot; 10&quot; 12&quot;</td>
</tr>
<tr>
<td>1/2&quot; 0</td>
<td>4.04 4.71 5.53 2.14 3.17 2.72 1.20</td>
</tr>
<tr>
<td>1/2&quot; S</td>
<td>5.60 6.60 7.60 3.14 4.17 3.72 2.20</td>
</tr>
<tr>
<td>5/16&quot; 0</td>
<td>7.20 8.20 9.20 4.17 5.17 4.72 3.20</td>
</tr>
<tr>
<td>3/16&quot; 0</td>
<td>8.80 9.80 10.80 5.17 6.17 5.72 4.20</td>
</tr>
<tr>
<td>7/16&quot; 0</td>
<td>10.40 11.40 12.40 6.17 7.17 6.72 5.20</td>
</tr>
<tr>
<td>1&quot; 0</td>
<td>12.00 13.00 14.00 7.17 8.17 7.72 6.20</td>
</tr>
<tr>
<td>1 1/2&quot; S</td>
<td>13.60 14.60 15.60 8.17 9.17 8.72 7.20</td>
</tr>
<tr>
<td>1 1/4&quot; S</td>
<td>15.20 16.20 17.20 9.17 10.17 9.72 8.20</td>
</tr>
</tbody>
</table>
**Table No. 3. Values of k, j, p and K.**

| n  | f_s | k   | j   | p   | K   | n  | f_s | k   | j   | p   | K   |
|----|-----|-----|-----|-----|-----|----|-----|-----|-----|-----|-----|-----|-----|-----|
| 10 | 1200| 0.423|0.067|0.0161|220.4|1100|12000|0.409|0.067|0.0133|206.3|
|    | 1250| 0.430|0.064|0.0171|254.6|1125|12000|0.410|0.063|0.0142|221.3|
|    | 1300| 0.441|0.061|0.0182|247.0|1150|12000|0.419|0.060|0.0157|234.5|
|    | 1350| 0.457|0.058|0.0193|261.0|1200|12000|0.420|0.057|0.0161|247.6|
|    | 1400| 0.467|0.055|0.0204|276.0|1250|12000|0.421|0.054|0.0170|261.4|
| 12 | 1500| 0.332|0.070|0.0103|143.2|1300|12000|0.329|0.067|0.0089|133.4|
|    | 2000| 0.379|0.066|0.0113|150.0|1350|12000|0.375|0.075|0.0093|146.8|
|    | 2500| 0.415|0.063|0.0123|170.0|1400|12000|0.400|0.071|0.0102|150.0|
|    | 3000| 0.457|0.059|0.0134|193.5|1450|12000|0.441|0.066|0.0111|172.3|
|    | 3500| 0.491|0.055|0.0146|217.3|1500|12000|0.432|0.063|0.0120|182.3|
|    | 4000| 0.534|0.051|0.0158|241.5|1550|12000|0.423|0.060|0.0129|192.4|
|    | 4500| 0.577|0.047|0.0170|265.5|1600|12000|0.444|0.055|0.0138|212.7|

**Table No. 3. Values of f_a = 20000**

<table>
<thead>
<tr>
<th>n</th>
<th>f_a</th>
<th>k</th>
<th>j</th>
<th>p</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>1200</td>
<td>0.365</td>
<td>0.075</td>
<td>0.0113</td>
<td>197.0</td>
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<tr>
<td></td>
<td>1250</td>
<td>0.378</td>
<td>0.072</td>
<td>0.0120</td>
<td>200.2</td>
</tr>
<tr>
<td></td>
<td>1300</td>
<td>0.394</td>
<td>0.069</td>
<td>0.0120</td>
<td>222.5</td>
</tr>
<tr>
<td></td>
<td>1350</td>
<td>0.403</td>
<td>0.066</td>
<td>0.0123</td>
<td>235.1</td>
</tr>
<tr>
<td></td>
<td>1400</td>
<td>0.412</td>
<td>0.063</td>
<td>0.0144</td>
<td>246.2</td>
</tr>
<tr>
<td>12</td>
<td>1500</td>
<td>0.379</td>
<td>0.066</td>
<td>0.0070</td>
<td>197.4</td>
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<tr>
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<td>1600</td>
<td>0.391</td>
<td>0.063</td>
<td>0.0070</td>
<td>196.4</td>
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<tr>
<td></td>
<td>1700</td>
<td>0.393</td>
<td>0.066</td>
<td>0.0067</td>
<td>196.4</td>
</tr>
<tr>
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<td>1800</td>
<td>0.390</td>
<td>0.066</td>
<td>0.0067</td>
<td>188.4</td>
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<tr>
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<td>1900</td>
<td>0.392</td>
<td>0.069</td>
<td>0.0079</td>
<td>191.6</td>
</tr>
<tr>
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<td>2000</td>
<td>0.403</td>
<td>0.064</td>
<td>0.0113</td>
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<tr>
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<td>2100</td>
<td>0.410</td>
<td>0.063</td>
<td>0.0123</td>
<td>210.1</td>
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<tr>
<td>15</td>
<td>2200</td>
<td>0.310</td>
<td>0.077</td>
<td>0.0047</td>
<td>67.50</td>
</tr>
<tr>
<td></td>
<td>2300</td>
<td>0.329</td>
<td>0.071</td>
<td>0.0053</td>
<td>64.40</td>
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<tr>
<td></td>
<td>2400</td>
<td>0.344</td>
<td>0.065</td>
<td>0.0060</td>
<td>106.3</td>
</tr>
<tr>
<td></td>
<td>2500</td>
<td>0.360</td>
<td>0.063</td>
<td>0.0060</td>
<td>110.0</td>
</tr>
<tr>
<td></td>
<td>2600</td>
<td>0.375</td>
<td>0.075</td>
<td>0.0075</td>
<td>131.2</td>
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<tr>
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<td>2700</td>
<td>0.390</td>
<td>0.083</td>
<td>0.0083</td>
<td>144.0</td>
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<tr>
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<td>2800</td>
<td>0.403</td>
<td>0.081</td>
<td>0.0091</td>
<td>157.0</td>
</tr>
</tbody>
</table>

Based on the formula:

\[
\begin{align*}
    n &= \frac{E_s}{E_c} \\
    k &= \frac{1}{1 + \frac{f_s}{(pn)^2}} \\
    p &= \frac{f_c k}{2f_s} \\
    j &= 1 - \frac{k}{3} \\
    K &= f_s p j = 0.5 f_c k j
\end{align*}
\]
RECTANGULAR BEAMS.
Tension Reinforced

Formulae

\[ k d^2 = \frac{2M}{f_c k j} = \frac{M}{f_s p j}, \quad j = \frac{1}{3} k, \quad k = \sqrt{2pn + (pn)^2 - pn} \]

\[ M = \frac{M}{k d^2} \]

Values of \( p = 10 \)

Values of \( p = 12 \)

Values of \( p = 15 \)

DIAGRAM I.
"T" BEAMS  Neutral Axis in Web

Diagram 2.

Formulae:

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

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

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

\[ R_2 = 3 \left( \frac{M}{f_t b t^2} + \frac{f_s}{f_t} + \frac{1}{2n} \right) \]
Values of $V_s$ for $f_s = 18000, \frac{3}{8} \phi U$

**STIRRUP SPACING.**

$V_c = \frac{vb}{d} = $ Shear Carried by Concrete  
$V_s = $ Shear Carried by Stirrups  
$s = \frac{A_s f_s j d}{V_s}$  
Max Spacing  
$s = \frac{450}{a+10} d = 0.45d$  

Stirrups must be hooked.

Formaloe

Effective depth = $d$

Values of $f_s$

16000  20000  30000  40000  50000  60000  70000  80000  90000  100000

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

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Values of $V_c$

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Values of $V_s$

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Values of $V_c$

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Values of $V_s$

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Values of $V_c$

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Values of $V_s$

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Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$

10  20  30  40  50  60  70  80  90  100

Values of $V_s$

10  20  30  40  50  60  70  80  90  100

Values of $V_c$
RECTANGULAR
ECCENTRIC COLUMNS
Compression over the
Entire Section.

Values of \( \frac{d'}{t} \):
- \( d' = 0.05 \)
- \( d' = 0.10 \)
- \( d' = 0.15 \)

Values of \( \frac{x_0}{t} \):
- 0.0
- 0.2
- 0.4
- 0.6
- 0.8
- 1.0
- 1.2
- 1.4
- 1.6
- 1.8
- 2.0

DIAGRAM 4