Standard Notations

(a) Rectangular Beams.
- \( f_s \) = tensile unit stress in longitudinal reinforcement.
- \( f_c \) = compressive unit stress in extreme fiber of concrete.
- \( E_s \) = modulus of elasticity of steel.
- \( E_c \) = modulus of elasticity of concrete.

\( \frac{n}{E_c} = M \) = bending moment, or moment of resistance in general.

\( A_s \) = effective cross sectional area of tension reinforcement.

\( b \) = width of beam.

\( d \) = effective depth, or depth from compression surface of beam to center of tension reinforcement.

\( k \) = ratio of depth of neutral axis to effective depth, \( d \).

\( j \) = ratio of lever arm of resisting couple to depth, \( d \).

\( j_d \) = depth from compression surface of beam to resultant of compressive stresses.

\( p \) = ratio of effective area of tension reinforcement to effective area of concrete in beam.

\( z \) = depth from compression surface of beam to resultant of compressive stresses.

(b) T-Beams.

\( b \) = width of flange.

\( b' \) = width of stem.

\( t \) = thickness of flange.

(c) Beams Reinforced for Compression.

\( A' \) = area of compressive steel.

\( f'e \) = compressive unit stress in longitudinal reinforcement.

\( C \) = total compressive stress in concrete.

\( C' \) = total compressive stress in steel.

\( d' \) = depth from compression surface of beam to center of compression reinforcement.

\( z \) = depth from compression surface of beam to resultant of compressive stresses.

(d) Shear, Bond and Web Reinforcement.

\( V \) = total shear.

\( V' \) = external shear on any section after deducting that carried by the concrete.

\( v \) = shearing unit stress.

\( u \) = bond stress per unit of area of surface of bar.

\( o \) = perimeter of bar.

\( Z_o \) = sum of perimeters of bars in one set.

\( a \) = spacing of web reinforcement bars, measured perpendicular to their direction.

\( s \) = spacing of web reinforcement bars, measured at the neutral axis and in the direction of the longitudinal axis of the beam.

\( A_w \) = total area of web reinforcement in tension within a distance, \( a \), of the total area of all bars bent up in any one plane.

\( a \) = angle between web bars and longitudinal bars.

\( f_v \) = tensile unit stress in web reinforcement.

Design Formulas

(a) Flexure of Rectangular Reinforced Concrete Beams and Slabs.

Computations of flexure in rectangular reinforced concrete beams and slabs shall be based on the following formulas:

1. Reinforced for tension only.

Position of neutral axis,

\[ k = \sqrt{\frac{2pn + (pn)^2}{p_n}} \]

Arm of resisting couple,

\[ j = 1 - \frac{k}{3} \]

Compressive unit stress in extreme fiber of concrete,

\[ f_c = \frac{2M}{jkb^2} = \frac{2f_s}{k} \]

Tensile unit stress in longitudinal reinforcement,

\[ f_s = \frac{M}{A_s j_d p_j b^2 d} \]

Steel ratio for balanced reinforcement,

\[ p = \frac{1}{3} \left( f_s - f_c \right) - \frac{1}{3} \left( f_s - f_c \right) \]

Note: For approximate computations, the following assumptions may be made:

\[ j = \frac{1}{3} \]

\[ k = \frac{1}{3} \]

\[ A_s = \frac{M}{\frac{j}{3}b^2 d} \]

\[ f_e = \frac{6M}{b d^2} \]

(2) Reinforced for both tension and compression:

Position of neutral axis,

\[ k = \sqrt{\frac{2n \left( p + p' \right)}{d} + n^2 \left( p + p' \right)^2 - n \left( p + p' \right)} \]

Position of resultant compression,

\[ z = \frac{k^2 + 2p' n \left( \frac{k - d'}{d} \right)}{d} \]

Arm of resisting couple,

\[ j_d = d - z \]

Compressive unit stress in extreme fiber of concrete,

\[ f_c = \frac{6M}{b d^2 \left[ 3k - k^2 + \frac{6p' n}{k} \left( \frac{k - d'}{d} \right) \left( 1 - \frac{d'}{d} \right) \right]} \]
Tensile stress in longitudinal reinforcement,
\[ f_s = \frac{M}{\eta jd} = nf_c \left( \frac{1-k}{k} \right) \]

Compressive stress in longitudinal reinforcement,
\[ f'_s = nf_c \left( \frac{k-d'}{d} \right) \]

(b) Flexure of Reinforced Concrete T-Beams;

Computations of flexure in reinforced concrete T-beams shall be based on the following formulas:

(a) Neutral axis in the flange:
Use the formulas for rectangular beams and slabs.

(b) Neutral axis below the flange:
The following formulas neglect the compression in the stem:

Position of neutral axis,
\[ kd = \frac{2ndA + b^2}{2nA + 2bt} \]

Position of resultant compression,
\[ z = \frac{3kd - 2t}{2kd - t} \]

Am of resisting couple,
\[ jd = d - z \]

Compressive unit stress in extreme fiber of concrete,
\[ f_c = \frac{Mkd}{bt(kd - \frac{1}{2}t)jd} \]

Tensile unit stress in longitudinal reinforcement,
\[ f_s = \frac{M}{Asjd} \]

(For approximate results, the formulas for rectangular beams may be used.)

The following formulas take into account the compression in the stem: they are recommended where the flange is small compared with the stem:

Position of neutral axis,
\[ kd = \sqrt{2ndA + (b - b')^2} \]

Position of resultant compression,
\[ z = \frac{(kd - t)^2 + \frac{1}{2}(kd - t)}{t(2kd - t) + (kd - t)^2} \]

Am of resisting couple,
\[ jd = d - z \]

Compressive unit stress in extreme fiber of concrete,
\[ f_c = \frac{2Nkd}{[(2kd - t)bt + (kd - t)^2b']jd} \]

Tensile unit stress in longitudinal reinforcement,
\[ f_s = \frac{M}{Asjd} \]

(c) Shear, Bond and Web Reinforcement

Diagonal tension and shear in reinforced concrete beams shall be calculated by the following formulas:

Shearing unit stress,
\[ V = \frac{V'}{bjd} \]

Stress in vertical web reinforcement.
\[ V' = \frac{f'v}{A_vjd} \]

When a series of web bars or bent-up longitudinal bars is used, the web reinforcement shall be designed in accordance with the formula:
\[ A_v = \frac{V'}{f_vjd (\sin a) + \cos a} \]

When the web reinforcement consists of bars bent up in a single plane so as to reinforce all sections of the beam which require it, the bent-up bars shall be designed in accordance with the formula:
\[ V' = \frac{A_vf_v}{\sin a} \]

The bond between concrete and reinforcement bars in reinforced concrete beams and slabs shall be computed by the formula:
\[ V = \frac{u}{jdZ_0} \]

(For approximate results "j, f_v, Z_0" in the above formulas, may be taken as \( \frac{1}{3} \).

The value of "Z_0" in bundled bars should reflect only the outside surface of the bundle.

Z_0 2-bar bundle = Z_0 2 bars
Z_0 3-bar bundle = Z_0 2 1/2 bars
Z_0 4-bar bundle = Z_0 3 bars

As regards shear and bond stress for tensile steel, the above formulas apply also to beams reinforced for compression.
CONCRETE DESIGN

RECTANGULAR BEAM
WITHOUT COMPRESSIVE REINFORCEMENT

RECTANGULAR BEAM
WITH COMPRESSIVE REINFORCEMENT

T-BEAM

BOX GIRDER
RESISTING MOMENTS OF BEAMS AND SLABS
FOR
BALANCED DESIGN AND COMPRESSION REINFORCEMENT

NOTATION

\( f_c = 1,300 \)
\( f_s = 24,000 \) psi
\( n = 10 \) for tensile reinforcement
\( n = 20 \) for compressive reinforcement
\( M_t = \) Total moment produced by external loads (ft-kips)
\( M = \) Resisting moment for balanced design (ft-kips)
\( M_s = \) Resisting moment of compressive reinforcement
\( \) (ft-kips)
\( A_s = \) Area of tensile reinforcement for balanced design (sq in)
\( A_s' = \) Area of compressive reinforcement for balanced design (sq in)
\( d = \) Effective depth of beam (inches)
\( d' = \) Embedment to center of gravity of compressive steel (inches)
\( b = \) width of girder (feet)

EXAMPLE I

Required: Tensile and compressive reinforcement

Given: A rectangular beam
\( d = 40'' \)
\( d' = 2'' \)
\( b = 1.5'' \)
\( M_t = 600 \) ft-kips

Solution:
\( M = 323 \) ft-kips (from Table 5-11)
\( M = 1.5 \times 323 = 484 \) ft-kips
\( \frac{116}{67} = 1.73 \) sq in
\( A_s = \frac{600}{323} \times 4.57 = 8.50 \) sq in

EXAMPLE II

Required: Width of beam and tensile reinforcement

Given: A rectangular beam
\( d = 40'' \)
\( d' = 2'' \)
\( M_t = 600 \) ft-kips
\( A_s = 3.12 \) sq in

Solution:
\( M_s = 67 \) ft-kips per sq in (from Table 5-12)
\( M' = 3.12 \times 67 = 209 \) ft-kips
\( M = 391 \) ft-kips
\( b = 1.21 \) ft. say 1 ft 3 in
\( A_s = \frac{600}{323} \times 4.57 = 8.50 \) sq in

T-BEAM VALUES

EXAMPLE III

Required: Tensile and Compressive Reinforcement

Given: A T-Beam
\( d = 40'' \)
\( b = 12'' \)
\( b' = 66'' \)
\( t = 5.5'' \)
\( M_t = 600 \) ft-kips

Solution:
\( M = 323 \) ft-kips (from Table 5-11)
\( M = 215 \) ft-kips (from Table 5-10.1)
\( M = 1183 \) ft-kips
\( M = 1506 \) ft-kips
\( 1506 > 600 \), no compressive reinforcement required.
\( A_s = \frac{600}{323} \times 4.57 = 8.50 \) sq in
Column Design Charts

Notations:

- \( d \) = least lateral dimension of column.
- \( f_c \) = compressive strength of concrete.
- \( f_y \) = yield stress of reinforcement.
- \( M \) = design bending moment at section due to ultimate loads.
- \( P \) = design axial load at section due to ultimate loads.

Design Specifications:

See Bridge Design Manual Volume 1, Article 6-11

Stresses:

- \( f'_c \) = 3,250 psi
- \( f'_y \) = 60,000 psi

Max Column Length = 10d

Use of Charts:

Enter the charts with an ultimate axial design load and an ultimate design bending moment and determine the reinforcement required.

**Example 1:** Given - 5'-6" octagonal column

\[ P = 7,500 \text{ kips} \]
\[ M = 5,500 \text{ ft-kips} \]

by interpolation Use 24-18

**Example 2:** Given - 6' round column

\[ P = 3,250 \text{ kips} \]
\[ M = 12,250 \text{ ft-kips} \]

by interpolation Use 24-18

Spiral Spacing:

The spacing of spirals depends on the strength of concrete and clearance to the spiral. For \( f'_c \) of 3250 psi and 2" clear to the spiral, #4 at 3-1/2" is sufficient for all column diameters. If a stronger concrete or a greater clearance is needed (as for corrosion protection in high chloride or marine environment), the spiral must be calculated from the following formula:

\[ P^1 = 0.45 \frac{A_g}{A_c} - 1 \frac{f'_c}{f'_{sp}} \]

\[ f'_{sp} = 60,000 \text{ psi} \]

\[ A_g = \text{gross column area} \]
\[ A_c = \text{area of column core} \]

\[ P^1 = \text{ratio of volume of spiral reinforcement to the volume of the concrete core (out to out of spirals).} \]

Maximum clear spacing for spirals is 3".
# Column Bar Arrangement

## Single Ring of Main Reinforcement

<table>
<thead>
<tr>
<th>Minimum Bar Spacing</th>
<th>Maximum Number of Bars (c-c bars)</th>
<th>Column Diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>4'</td>
</tr>
<tr>
<td><strong>10 Bars</strong></td>
<td><img src="image1" alt="Diagram" /></td>
<td>27</td>
</tr>
<tr>
<td><strong>11 Bars</strong></td>
<td><img src="image2" alt="Diagram" /></td>
<td>25</td>
</tr>
<tr>
<td><strong>14 Bars</strong></td>
<td><img src="image3" alt="Diagram" /></td>
<td>23</td>
</tr>
<tr>
<td><strong>18 Bars</strong></td>
<td><img src="image4" alt="Diagram" /></td>
<td>21</td>
</tr>
</tbody>
</table>
COLUMN BAR ARRANGEMENT
TWO RINGS OF MAIN REINFORCEMENT

<table>
<thead>
<tr>
<th>Minimum Bar Spacing</th>
<th>Maximum Number of Bars (c-c bars)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#18 Bars</td>
<td>40</td>
</tr>
<tr>
<td>#8 Template</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td>62</td>
</tr>
<tr>
<td></td>
<td>84</td>
</tr>
</tbody>
</table>

The number of bars in the inner ring shall be a convenient fraction of the number of bars in the outer ring, so the reinforcement can be bundled and symmetrically placed.

Whenever the footing depth is sufficient to provide adequate bond length, straight bars shall be used for the inner ring of reinforcement. When footing depth is not sufficient to provide adequate bond length, hooked bars shall be used and detailed on the plans as shown below:
d = 4' COLUMN

Scale:
Horiz. 1" = 1000 k-ft
Vert. 1" = 1000 k
Inverted T-Caps

Inverted T-cap bents should be designed so that the falsework can be removed before the girders are placed. If, for unusual circumstances, it is necessary to leave the bent falsework in place until the superstructure is completed, suitable notes shall be placed on the plans requiring falsework to be designed to support the entire superstructure load and not to be removed until deck (or top of cap) concrete attains a specified strength.

In addition to the forces which are ordinarily used for design, end of girder and seat details are subjected to other forces caused by construction irregularities, skews, deflections, impact during construction, and changes in length caused by creep and shrinkage. These factors must be considered in the design process. Several instances of cracking in seats of inverted T-caps, used for supporting precast girders, have primarily been due to:

- Girder rotation
- Edge loading
- Plastic prestress shortening (creep) between bents
- Insufficient reinforcing steel
- Poor arrangement of reinforcing steel

*Memo to Designers 7-1* gives bearing pad recommendations which will help prevent spalling of the girder ends and seat edges. Sufficient prestressing steel must be placed in the girders, and sufficient reinforcement placed continuously across bent caps to satisfy tensile stresses caused by girder plastic prestress shortening between adjacent supports not having expansion joints.

Refer to the following pages for analysis and design instructions and examples.

Following are some illustrations which will help visualize the design procedure and complexity for inverted T-Caps.
Notes:

1. Horizontal plastic prestress shortening (creep) and thermal loads to be resisted by continuous reinforcement in deck.
2. Minimum tensile loads required by specifications.
Ledge Design Length

Dead Load Application
(Longitudinal View of Ledge)

Live Load Application
(Longitudinal View of Ledge)
A. Design Strategy

Dead load of girders and deck is transmitted directly to portion of ledge under girders through the pad, assuming diaphragm concrete is placed with deck concrete.

Live load and added dead load are transmitted through the deck and the girders to the end diaphragms into ledge.

1. Corbel Design
   a. Under girder: \( D_{\text{Deck + Girder}} + (L + I + D_{\text{Added}}) \) (Minor Portion)
   b. Between Girders: \( (L + I + D_{\text{Added}}) \) (Major Portion)

   ![Diagram of Corbel Design](image)

   **Typical Ledge Loading at Interior Girder**

   **Note:** Live load and added dead load distribution to ledge within width \( w + d \) should be assumed distributed uniformly across \( w + d \) for design purposes.

2. Bent Cap Design

Design should be similar to conventional bent caps (i.e., girders and wheel lines treated as concentrated loads). The inverted T-Section should be used for the shape of the design member, and all flexural and shear reinforcement should be fully contained within the section. One exception is that the top hooks of stirrups may extend into the deck slab.

It is recommended that other nominal or tensile reinforcement be extended from the horizontal and vertical ledge faces between fixed girder ends to enhance continuity.
Typical where spans are continuous across bent cap.

Section of Cap between Girders

Designers must address flexural problems in the cross-sectional direction if the inverted-T becomes relatively wide (see illustrations below). Normally the cap is slightly wider than the column with only the ledges extending noticeably beyond the column face. The designer must be sure that the support is stable under all temporary construction stages.

Examples of Non-Typical Inverted T-Caps
B. Design Commentary

Lower cap projections which support girders must meet the criteria for corbels. Corbel design limits and criteria are presented in *Bridge Design Specifications*, Article 8.16.6.8. The following criteria are to be considered:

1. The corbel criteria is suitable without modifications at columns which provide a compression reaction below the resisting shear-friction plane. An additional calculation for diagonal shear reinforcement is required at locations between columns if the column is inset more than normal from the shear-friction plane, or if a non-structural column flare, which could be lost in a seismic event, exists. Article 8.16.6.2.3, “Shear in Tension Members”, should be used to satisfy diagonal shear.

![Nomenclature Sketch](image)
2. Use vertical and horizontal loads of:

\[ V_u = 1.30 \left[ DL + \frac{5}{3} (LL + I)_{HS} \right] \text{ or } \]

\[ 1.30 \left[ DL + (LL + I)_{P} \right] \text{ — Avoid widely spaced girders} \]

\( N_{uc} \) = shear force as per Memo to Designers 7-1 for expansion ends. In no case shall \( N_{uc} \) be less than \( 0.2 \) \( V_u \) (ACI 11.9.4) at both expansion and fixed ends.

3. Check for the effect of the appropriate loads acting with the girder on the area below the girder. Determine the width of seat “b”. For interior girders, “b” equals the width of the bearing pad plus the depth “d” of the corbel. For exterior girders, “b” equals the bearing pad plus one-half the depth “d” of the corbel plus edge distance to end of cap, not to exceed \( d/2 \). The seat reinforcement must be placed within the width of the seat, “b”.

4. Compute \( A_s \) for both exterior and interior girders, and for ledge between girders. On either fixed ends or expansion ends which require additional pads between the girders, a load distribution scheme must be determined by the designer consistent with the construction sequence. The ledge must be reinforced accordingly.

5. Secondary tension bars shall be uniformly distributed in the upper two-thirds of the effective depth “d”. They shall be placed parallel to the tension reinforcement “\( A_s \)” and have a cross-sectional area “\( A_h \)” not less than \( 0.5( A_s - A_h ) \). See Bridge Design Specifications, Article 8.16.6.8.

6. Longitudinal corbel distribution bars, \( A'_s \), shall be centered under all exterior bearing pads. Minimum area should be \( A_s/2 \). Uniformly space bars and extend them “d” beyond the seat width “b”.

7. Keep pad a minimum of 3 inches from the edge of corbel to prevent high edge loadings.

8. Reinforcing steel at the edges of bearing seats may need specially detailed hooks to accommodate intersecting bars because of tight clearances.

9. \( A_s \) bar size should be chosen to allow required extension and development in the confined area. Crossbars welded to the ends of straight tension reinforcement (\( A_s \)) is an alternative when the radius of the hook bend is too large relative to ledger size. Size of crossbars should be that of the tension reinforcement. The following table shows allowable lengths for minimum “\( X \)” (see illustrations) based on hooked top bars without enclosure and \( f'_c = 3,250 \) psi.
Note: It is not reasonable to use bars larger than #7 because the ledge extension would become excessively large. Closer girder spacing, deeper ledge section, or higher strength concrete are three methods to reduce the bar size.

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Minimum &quot;X&quot;</th>
</tr>
</thead>
<tbody>
<tr>
<td>#4</td>
<td>1'-3&quot;</td>
</tr>
<tr>
<td>#5</td>
<td>1'-5&quot;</td>
</tr>
<tr>
<td>#6</td>
<td>1'-9&quot;</td>
</tr>
<tr>
<td>#7</td>
<td>2'-6&quot;</td>
</tr>
</tbody>
</table>

*In accordance with Figure 11.9.6 in ACI 316-83 Commentary

Hook/Crossbar Illustrations
10. Check diagonal tension reinforcement requirements for loading on the beam ledge. Use *Bridge Design Specifications*, Articles 8.16.6.2.3 (Shear in Tension Members) and 8.16.6.3 (Shear Strength Provided by Shear Reinforcement).

Any combination of vertical and diagonal bars may be used to satisfy the condition. Diagonal bar areas must be corrected for the angle of the bar to an effective area.

These shear bars are not in addition to the cap shear stirrups from a bent analysis. The corbel loads used to satisfy the diagonal shear are the same loads used to analyze the bent. The analysis requiring the greatest area of reinforcement per unit length of bent cap should be used.

**C. Details**

Sufficient plan details must be provided to show all reinforcement patterns and for all stages of construction. The details must clearly identify corbel and bent cap reinforcement at columns and between columns, for loads at pads and for loads in between pads. Care must be taken to assure that the corbel steel can be placed amongst the column bars and spiral. Bridge skews complicate the layering and interweaving of bars. Special attention by the designer is required to avoid conflicts.

Sections need to be shown for constructing the inverted T, and also for a final condition with girders in place and diaphragm concrete cast around the girder ends.
D. Design Example

Following is a design example using the foregoing criteria. The example should be considered a guide, and not a standard solution for all inverted T-Caps. Major widenings should be designed with a T-Cap independent from the existing cap using the foregoing criteria. Strip widenings requiring an existing cap extension should use existing reinforcement details, but improved to meet the foregoing design considerations.

The latest OSD policies on bent cap joint shear are not considered in this example. The designer is responsible for performing a joint shear analysis, and provide supplemental reinforcement, as required, to satisfy load demands from the analysis.
Inverted "T" Bent Cap Design Example

I. Design Considerations

A. Flange/Ledge Design
   (1) flange punching shear at girder bearings
   (2) primary tension reinforcement
   (3) secondary tension reinforcement
   (4) corbel distribution reinforcement
   (5) diagonal tension

B. Overall Bent Cap Design
   The inverted "T" bent cap should be designed for the following conditions.
   - Max moment and associated shear and torsion
   - Max shear and associated moment and torsion
   - Max torsion and associated shear and moment
   These items will not be addressed in this example.

II. Design Procedure and Example Problem:
   Inverted 'T' Bent Cap – Ledge Design

   Design procedure for the ledge of the inverted "T" Bent Cap is as follows.
   The ledge will be designed at 3 locations along the bent cap:
   (a) interior girder
   (b) exterior girder
   (c) between girders
1/2 bearing pad
from face of conc. $a_v = 6'' + 6'' = 12''$
$d = 24'' - 2'' = 22''$

$\frac{1}{2}d$ = edge distance = 0

$\frac{1}{2}d$ = edge distance = 0

$x = \text{edge distance} = 0$

$\frac{1}{2}d$ = edge distance = 0

$b_{\text{int}} = 19'' + 22'' = 41''$

$\frac{1}{2}d$ = edge distance = 0

$b_{\text{ext}} = 19'' + 22''/2 + 0 = 30''$

$b_{i} = 72'' - 19'' - 22 = 31''$

$6'' + 6'' = 12''$ from face of conc.

$6'' + 6'' = 12''$ from face of conc.

$b_{i} = W + d$

$b_{i} = W + d$ between girders

$b_{i} = (6'' + 0'') - W - d$

$b_{i} = (6'' + 0'') - W - d$

interior girder

exterior girder

$b_{i} = \frac{W + d}{2} + x$

$b_{i} = \frac{W + d}{2} + x$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$

$6'' + 6'' = 12''$
A. Given

Girder spacing = 6' - 0" o.c.
plain bearing pads ½ × 12 × 19"
h = 24" say d = 22"

Loads per girder – computed by tributary area method
DL per girder 130 k (includes weight of top deck)
Added DL per girder 30 k
(LL + I)HS per girder 80 k
“P” loads not considered in this example.

B. Design Loads

1. Vertical Shear, \( V_u \)

\[ (W_o) \text{ Add DL + LL} = 1.3 \left[ 30 k + \frac{5}{3}(80 k) \right] \text{/6 feet} \]

\[ = 35.4 \frac{k}{\text{ft}} \]

**Interior Girder**

Design for DL + Add DL + LL

\[ V_u = 1.3 (130 k) + (35.4 \frac{k}{\text{ft}}) (\frac{\text{in}}{12 \text{in/ft}}) = 290 k \]

**Exterior Girder**

Design for DL + Add DL + LL

\[ V_u = 1.3 (130 k) = (35.4 \frac{k}{\text{ft}}) (\frac{\text{in}}{12 \text{in/ft}}) = 258 k \]

**Between Girders**

Design for Add DL + LL

\[ V_u = (35.4 \frac{k}{1}) (\frac{\text{in}}{12 \text{in/ft}}) = 92 k \]
2. Horizontal Shear, $N_{uc}$

$$N_{uc} \geq \begin{cases} 
\text{horizontal pad shear (Memos to Designers 7.1)} \\
0.2 \ V_u \text{ (BDS Art. 8.16.6.8.3)} 
\end{cases}$$

Pad shear $F_s = \frac{G(A)\delta_s}{t}$ \quad $\delta_s = 0.5$ \quad $F_s = \frac{(170\text{psi})(12\text{''})(19\text{''})(0.5\text{''})}{0.5\text{''}} = 39\text{ k}$

**Interior Girder**

$$N_{uc} \geq \begin{cases} 
\text{pad shear} = 39\text{ k} \\
0.2 \ V_u = 0.2 \ (290) = 58\text{ k} \ - \text{controls} 
\end{cases}$$

**Exterior Girder**

$$N_{uc} \geq \begin{cases} 
\text{pad shear} = 39\text{ k} \\
0.2 \ V_u = 0.2 \ (258) = 52 \text{ k} \ - \text{controls} 
\end{cases}$$

**Between Girder**

$$N_{uc} \geq \begin{cases} 
\text{pad shear} = 0 \\
0.2 \ V_u = 0.2 \ (92) = 18\text{ k} 
\end{cases}$$

3. Summary

- $(V_u)_{\text{int. girder}} = 290\text{ k}$ \quad $(N_{uc})_{\text{int}} = 58\text{ k}$
- $(V_u)_{\text{ext. girder}} = 258\text{ k}$ \quad $(N_{uc})_{\text{ext}} = 52\text{ k}$
- $(V_u)_{\text{bwn. girder}} = 92\text{ k}$ \quad $(N_{uc})_{\text{bwn}} = 18\text{ k}$
C. Flange Dimension Check

1. Check Punching Shear

\[ V_u < 0.85 \times \sqrt{f'_c} \times b_0 \times d \]

**Exterior Girder**

\[ (V_u)_{ext} < (0.85) \times 4\sqrt{3250} \times (59)(22) = 251 \text{ k} \]

\[ V_u = 258 \text{ k} > 251 \text{ k} \rightarrow \text{NG} \]

Seat inadequate for punching shear. Try increasing depth of flange.

Try \( h = 30'' \), \( d = 28'' \)

\[ (V_u)_{ext} \leq 0.85 \times (4) \times \sqrt{3250} \times (59)(28) = 220 \text{ k} \]

\( (V_u)_{ext} = 258 \text{ k} \rightarrow \text{okay} \)

\( (V_u)_{int} \leq 0.85 \times (4) \times \sqrt{3250} \times (99)(28) = 537 \text{ k} \)

\( (V_u)_{int} = 290 \text{ k} \rightarrow \text{okay} \)

\[ \therefore \text{ Use } h = 30 \text{ inches} \]

2. \( a/\eta_1 = 12/\eta_2 = 0.43 < 1.0 \)

\( \therefore \text{ Corbel design okay} \)
D. Compute $A_s$ – Primary Tension Reinforcement

$A_s$ to resist simultaneously
- shear $V_u$
- moment $V_u a_y + N_{uc}$ (h-d)
- tensile force $N_{uc}$

1. $A_{s,fr}$ – Shear Friction Reinforcement

**Interior Girder**

\[
V_n = \frac{V_u}{\phi} = \frac{290}{0.85} = 341 \, k
\]

\[
V_n \leq 0.2 f'_c A_{cv} = 0.2 (3.25)(41 \, \text{inches})(28) = 746 \, k \rightarrow \text{okay}
\]

\[
\leq 800 A_{cv} = 0.800 (41) (28) = 918 \, k \rightarrow \text{okay}
\]

\[
A_{s,fr} = \frac{V_n}{f_y} = \frac{341}{60(1.4)} = 4.06 \, \text{sq. in.}
\]

$\mu = 1.4$ for concrete placed monolithically

**Exterior Girder**

\[
V_n = \frac{258}{0.85} = 304 \, k
\]

\[
V_n \leq 0.2 f'_c A_{cv} = 0.2 (3.25)(30)(28) = 546 \, k \rightarrow \text{okay}
\]

\[
V_n \leq 800 A_{cv} = 0.8 (30)(28) = 672 \, k \rightarrow \text{okay}
\]

\[
A_{s,fr} = \frac{304}{60(1.4)} = 3.62 \, \text{sq. in.}
\]

**Between Girders**

\[
V_n = \frac{92}{0.85} = 108 \, k
\]

\[
A_{s,fr} = \frac{108}{60(1.4)} = 1.29 \, \text{sq. in.}
\]
2. $A_f$ – Flexural Reinforcement

**Interior Girder**

\[
M_u = \left[ V_u a_v + N_{uc} (h-d) \right] = (290)(12) + 58(2 \text{ inches}) = 3596 \text{ k-in.}
\]

\[
M_u = \phi A_f f_y \left[ d - A_f f_y/(1.7 f'_b) \right]
\]

\[
3596 = 0.85 A_f (60) \left[ 28 - \frac{A_f (60)}{1.7(3.25)(31)} \right]
\]

Solving for $A_f$ gives $A_f = 2.59 \text{ sq. in.}$

**Exterior Girder**

\[
M_u = (258)(12) + 52(2) = 3200 \text{ k-in}
\]

\[
3200 = 0.85 A_f (60) \left[ 28 - \frac{A_f (60)}{1.7(3.25)(30)} \right] \rightarrow A_f = 2.31 \text{ k-in}
\]

**Between Girders**

\[
M_u = (92)(12) = 1104 \text{ k-in}
\]

\[
1104 = 0.85 A_f (60) \left[ 28 - \frac{A_f (60)}{1.7(3.25)(31)} \right] \rightarrow A_f = 0.79 \text{ in}^2
\]

3. $A_n$ – Direct Tension Reinforcement

\[
N_{uc} \leq \phi A_n f_y \rightarrow A_n = \frac{N_{uc}}{0.85 f_y}
\]

**Interior Girder**

\[
A_n = \frac{58}{(0.85)(60)} = 1.14 \text{ in}^2
\]

**Exterior Girder**

\[
A_n = \frac{52}{(0.85)(60)} = 1.02 \text{ in}^2
\]

**Between Girders**

\[
A_n = 0
\]
4. Compute $A_s$

\[
A_s \geq \begin{cases} 
\left( \frac{2A_{vl} + A_n}{3} \right) \\
(A_t + A_n) \\
0.04\left( \frac{f'_s}{f_y} \right) \cdot bc = 0.0607b
\end{cases}
\]

(BDS Art. 8.16.6.8.5)

**Interior Girders**

\[
\left[ \frac{2 \times (4.06)}{3} + 1.14 \right] = 3.85 \text{ in}^2
\]

\[
A_s \geq [2.59 + 1.14] = 3.73
\]

\[
0.0607(41) = 2.49
\]

$A_s = 3.85 \text{ sq. in.}$  Use #6 tot. 9

**Exterior Girders**

\[
\left( \frac{2}{3}(3.62) + 1.02 \right) = 3.43 \text{ in}^2
\]

\[
A_s \geq 2.31 + 1.02 = 3.33 \text{ in}^2
\]

\[
0.0607(30) = 1.82 \text{ in}^2
\]

$A_s = 3.43 \text{ in}^2$  Use #6 tot. 8

**Between Girders**

\[
[\frac{(2/3)(1.29)}{3}] = 0.86 \text{ in}^2
\]

\[
A_s \geq 0.79 \text{ in}^2
\]

\[
0.0607(31) = 1.88 \text{ in}^2
\]

$A_s = 1.88 \text{ in}^2$  Use #6 tot. 5
E. Compute \( A_h \) – Shear Reinforcement (Secondary Tension Reinforcement)

\[
A_h \geq 0.5 (A_s - A_n) \quad \text{(BDS Art. 8.16.6.8.4)}
\]

**Interior Girder**

\[
A_h = 0.5 (3.85 - 1.14) = 1.36 \text{ in}^2 \quad \text{Use } \#5 \text{ tot 3}
\]

**Exterior Girders**

\[
A_h = 0.5 (3.43 - 1.02) = 1.21 \text{ in}^2 \quad \text{Use } \#5 \text{ tot 3}
\]

**Between Girders**

\[
A_h = 0.5 (1.88) = 0.94 \text{ in}^2 \quad \text{Use } \#5 \text{ tot 4}
\]

F. Compute \( A'_s \) – Longitudinal Corbel Distribution Reinforcement

**Exterior Girder**

\[
(A'_s)_{\text{min}} = 0.5 \ A_s
\]

\[
A'_s = 0.5 (3.43) \text{ sq. in.} = 1.72 \text{ in}^2 \quad \text{Use } 4 \#6 \text{ each end.}
\]

**Other Locations**

Provide minimum distribution reinforcement 2 \#5 bars.
G. Compute $A_v$ – Diagonal Tension Reinforcement

Diagonal Tension reinforcement is required between columns to cross the diagonal crack. At column supports the shear can be carried through column steel.

\[ V_U = \phi(V_S + V_C) \]

$V_C$ is reduced for concrete in tension

* $d_{eff}$ is used in calculations of $V_C$.

1. At Girders – Assume interior girder controls

\[ V_c = 2 \left[ 1 + \frac{N_u}{(500 A_g)} \right] \sqrt{f'_c} b_w d_{eff} \]

(BDS Art. 8.16.6.2.3.)

- $N_u = -V_u = -290 \text{ k} \text{ (tension)}$
- $d_{eff} = 18 \text{ inches}$
- $A_g = (18)(41 \text{ inches}) = 738 \text{ in}^2$

\[ V_c = 2 \left[ 1 - \frac{290}{0.5(738)} \right] \sqrt{3250} (41)(18) = 18 \text{ k} \]
(V_s/req) = \frac{V_u}{\phi} - V_c = \frac{290}{0.85} - 18 = 323 \text{ k} \quad \text{This force is resisted by reinforcement crossing the tension crack.}

Try #6 \(\Box\) at 18 inches max. bent cap stirrups and 4 #7 \(\sqrt{\phantom{a}}\) at each girder

\#6 \(\Box\) \(A = (6 \text{ legs})(0.44 \text{ in}^2) = 2.64 \text{ in}^2 \quad \text{(6 legs effective within } b_{\text{int}} = 41 \text{ inches})\)

4 #7 \(\sqrt{\phantom{a}}\) \(A = 4(0.60 \text{ in}^2)(\sin x + \cos x) = 3.38 \text{ in}^2 \quad \text{for } x = 45^\circ \quad \text{(BDS Art. 8.16.6.3.3)}\)

\(A_{\text{tot}} = 2.64 + 3.38 = 6.02 \text{ in}^2\)

\(V_{s/\text{prov}} = A_s \times f_y = (6.02 \text{ in}^2)(60 \text{ ksi}) = 361 \text{ k}\)

\((V_s)_{\text{prov}} > (V_s)_{\text{req}} \rightarrow \text{okay}\)

\(\therefore \text{ Use } 4 \#7 \sqrt{\phantom{a}} \text{ at each girder}\)

\#6 \(\Box\) \(\sqrt{\phantom{a}}\) at 18 inches max. bent cap stirrups

2. Between Girders

\(N_u = -V_u = -92 \text{ k}\)

\(V_c = 2 \left(1 - \frac{92}{0.5(738)}\right)\sqrt{3250} (41)(18) = 63 \text{ k}\)

\((V_s)_{\text{req}} = \frac{V_u}{\phi} - V_c = \frac{92}{0.85} - 63 = 45 \text{ k}\)

\(\text{using } \#6 \Box \sqrt{\phantom{a}} \text{ at 18 along cap}\)

\(\text{for } b = 30 \text{ inches, 4 legs effective}\)

\(V_s = 4(0.44 \text{ in}^2)(60 \text{ ksi}) = 105 \text{ k} > (V_s)_{\text{req}} \rightarrow \text{okay}\)

\(\therefore \text{ No diagonal bars required}\)
III. Design Flow Chart

Start

Punching shear adequate?  
BDS Art. 8.16.6.6.2

No  
Increase d

Yes  

Given:

Design for Beam Shear.  
BDS Art. 8.16.6.2  
BDS Art. 8.16.6.3

Check Corbel Criteria  
$\alpha_v / d < 1$  
BDS Art. 8.16.6.8.1

No

Yes  

Shear friction  
$V_n = \frac{V_u}{0.85}$  
BDS Art. 8.16.6.8.3 (a)

$V_n \leq 0.20 f_c bd$  
BDS Art. 8.16.6.4.5

No

Yes

$V_n \leq 800 bd$  
BDS Art. 8.16.6.4.5

$A_w = \frac{V_n}{f_y \mu}$  
BDS Art. 8.16.6.4.4

$\mu_u = V_u (a_u) + N_{uc} (h-d)$  
BDS Art. 8.16.6.8.3 (c)

solve for $A_f$  
$\phi_{Af y} (d - \frac{A_f y}{2(0.85)f_y f_c b}) = \mu_u$  
BDS Art. 8.16.3.2

Yes

No  

Minimum Horizontal Shear  
$N_{uc} < 0.2 V_u$  
BDS Art. 8.16.6.8.3 (d)

$N_{uc} = 0.2 V_u$

BDS Art. 8.16.6.8.8.3 (c)

1
Direct Tension

\[ N_{uc} = 0.85 f_y \]

BDS Art. 8.16.6.8.3 (d)

Solve for \( A_s \)

\[ A_s \geq \begin{cases} (A_t + A_n) \\ (2A_v/f_c/3 + A_n) \\ (0.04 f_c/f_y b_d) \end{cases} \]

BDS Art. 8.16.6.8.3 (e)

\[ A_n = 0.5 (A_s - A_n) \]

BDS Art. 8.16.6.8.4

Corbel design finished.

Diagonal Tension Design

\[ V_c = 2 \left[ 1 + \frac{N_u}{500 A_y} \right] \sqrt{f_c (b_w d)} \]

BDS Art. 8.16.6.2.3

\[ V_u > \phi V_c \]

End \( \rightarrow \) No

\[ A_v = \frac{(V_u/\phi - V_c)}{f_y} \]

End

Compute Longitudinal Corbel Distribution Reinforcement (exterior girders)

\[ A_s' = 0.5 A_s \]

*Bridge Design Aids 5-47*
### Areas and Perimeters for Various Bar Sizes and Number of Bars

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**Note:** The table above provides the areas and perimeters for various bar sizes and number of bars. The top numbers in the table represent areas, while the bottom numbers represent perimeters. The units are typically in square meters or square feet, depending on the context and application.
### Areas and Perimeters for Various Bar Sizes and Spacing

**Top Numbers Are Areas**

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**Bottom Numbers Are Perimeters**

The table provides the areas and perimeters for various bar sizes and spacings. The values are given in inches and inches of spacing.