PUNCHING SHEAR CALCULATIONS
ACI – 318; ADAPT-PT

1. OVERVIEW

Punching shear calculation applies to column-supported slabs, classified as two-way structural systems.

This writing (i) defines the different conditions for punching shear calculation, (ii) presents the relationships used for code check of each condition using ACI-318, (iii) presents a numerical example for each condition, and (iv) demonstrates that the program ADAPT-PT correctly recognizes each case, and accordingly. This writing also serves as a guideline for verification of punching shear calculations reported by ADAPT-PT.

Depending on the location of a column with respect to the slab edges, four conditions are identified. These are:

- **Interior** column, where the distance from each face of a column to the slab edge is at least four times the slab thickness (columns 4 and 5 in Fig. 1-1);
- **Edge** column, where one face of a column in direction of design strip is closer to the slab edge in the same direction by four times the slab thickness (column 2 in Fig. 1-1);
- **Corner** column, where two adjacent faces of a column are closer to their associated slab edges by less than four times the slab thickness (column 1 in Fig. 1-1);
- **End** column, where a column face is closer to a slab edge normal to the design strip by less than four times the slab thickness (Column 6 in Fig. 1-1)

Columns at re-entrant corners, such as column 3 in Fig. 1-1 are conservatively treated as edge columns. Punching shear relationships of the code do not apply to columns that are connected to one or more beams, nor do they apply to walls/supports. Adequacy of shear transfer in such cases has to be established differently.

The calculations are presented by way of a numerical example. The geometry, material, loading and other particulars of the structure selected for the numerical example are given below and in Fig. 1-1.

Thickness of slab = 9 in (229 mm)
(i) Material Properties

- **Concrete:**
  - Compressive strength, $f'_c$ = 4000 psi (27.58 MPa)
  - Weight = 150 pcf (2403 kg/m³)
  - Modulus of Elasticity = 3605 ksi (24856 MPa)

- **Prestressing:**
  - Low Relaxation, Unbonded System
  - Strand Diameter = ½ in (13 mm)
  - Strand Area = 0.153 in² (98 mm²)
  - Modulus of Elasticity = 28000 ksi (193054 MPa)
  - Ultimate strength of strand, $f_{pu}$ = 270 ksi (1862 MPa)

  - Minimum strand cover
    - From top fiber = 1 in all spans (25 mm)
    - From bottom fiber
      - Interior spans = 1 in (25 mm)
      - Exterior spans = 1 in (25 mm)

- **Nonprestressed Reinforcement:**
Yield stress \( f_y \) = 60 ksi \((413.69 \text{ MPa})\)  
Modulus of Elasticity = 29000 ksi \((199,949 \text{ MPa})\)  
Minimum Rebar Cover = 0.75 in Top and Bottom \((19 \text{ mm})\)

(ii) Loading  
Dead load = self weight + 20 psf (superimposed)  
Live load = 40 psf \((1.92 \text{ kN/m}^2)\)

1.1. Relationships

The calculations are intended to determine whether or not a given slab-column connection meets the minimum safety requirements of the code against failure. It is not the intent of the calculations to find the “actual” condition of stress distribution at the column-slab location. The relationships used are empirical. Using test results, the relationships are calibrated to deliver safe designs.

The calculation steps are:

- Determine the factored column moment (design moment \( M_u \)) and the factored shear (design shear \( V_u \)). In many instances, column reaction is conservatively used as design value for punching shear.
- Consider a fraction of the unbalanced moment \( J M_u \) to contribute to the punching shear demand. The unbalanced moment is conservatively taken as the sum of upper and column moments at a joint.
- Using the code relationships, select an assumed (critical) failure surface and calculate a hypothetical maximum punching shear stress for the assumed surface.
- Using the geometry of the column-slab location and its material properties, calculate an “allowable” punching shear stress.
- If the maximum punching shear stress calculated does not exceed the allowable value, the section is considered safe.
- If the hypothetical maximum punching shear stress exceeds the allowable value by a moderate amount, punching shear reinforcement may be provided to bring the connection within the safety requirements of the code. The design of punching shear reinforcement is not covered in this writing.
- If the hypothetical maximum punching shear reinforcement exceeds the allowable values by a large margin, the section has to be enlarged.

The basic relationship is as follows:

\[
V_u = \frac{V_u}{A_c} + \frac{\gamma \times M_u \times c}{J_c} \tag{1-1}
\]

Where,
- \( V_u \) = absolute value of the direct shear;
- \( M_u \) = Unbalanced column moment;
- \( A_c \) = area of concrete of assumed critical section;
- \( J_c \) = fraction of the moment transferred by shear;
\( c = \) distance from centroidal axis of critical section to the perimeter of the critical section in the direction of analysis; and
\( J_c = \) a geometry property of critical section, analogues to polar moment of inertia of segments forming area \( A_c \).

The first critical shear failure plane is assumed at a distance \( d/2 \) from the face of support. Where “\( d \)” is the effective depth of the section.

Expressions for \( A_c, J_c, \) and \( \gamma_v \) for all types of columns are given below.

(i) Interior Column (Fig. 1.1-1)

![Diagram of Interior Column](image)

\[
A_c = 2d(c_1 + c_2 + 2d)
\]
\[
J_c = (c_1 + d) \cdot d^3/6 + (c_1 + d)^3 d/6 + d \cdot (c_2 + d) \cdot (c_1 + d)^2/2
\]
\[
\gamma_v = 1 - \{1/[1+(2/3)\cdot((c_1+d)/(c_2+d))^{1/3}]\}
\]

Where \( c_1 \) and \( c_2 \) are the column dimensions with \( c_1 \) perpendicular to the axis of moment, and \( d \) is the effective depth.

(ii) End Column (Refer Fig. 1.1-2)
FIGURE 1.1-2 FOR A DESIGN STRIP IN LEFT-RIGHT DIRECTION

\[ A_c = d \left( 2c_1 + c_2 + 2d \right) \]
\[ c_{AB} = \left( c_1 + d/2 \right) / \left( 2c_1 + c_2 + 2d \right) \]
\[ c_{CD} = \left( c_1 + d/2 \right) - c_{AB} \]
\[ J_c = (c_1 + d/2)^3/6 + 2d \times \left( c_{AB}^3 + c_{CD}^3 \right) / 3 + d \times (c_2 + d) c_{AB}^2 \]
\[ \gamma_V = 1 - \{1/(1+ (2/3) \times ((c_1 + d/2) / (c_2 + d)))^{1/2}\} \]

Where \(c_1\) and \(c_2\) are the column dimensions with \(c_1\) parallel to the axis of moment, and \(d\) is the effective depth.

(iii) Edge Column (Refer Fig. 1.1-3)

\[ A_c = d \left( 2c_2 + c_1 + 2d \right) \]
\[ J_c = (c_1 + d)^3 \times d / 12 + (c_1 + d) \times d^3/12 + d \times (c_2 + d/2) \times (c_1 + d)^2 / 2 \]
\[ \gamma_V = 1 - \{1/[1+ (2/3) \times ((c_1 + d) / (c_2 + d))]^{1/2}\} \]

Where \(c_1\) and \(c_2\) are the column dimensions with \(c_1\) perpendicular to the axis of moment and \(d\) is the effective depth.

Column at the re-entrant corner as shown in Fig. 1.1-4 is treated as Edge-column.
(iv) Corner Column (Refer Fig. 1.1-5)

\[ Ac = d (c_1 + c_2 + d) \]
\[ c_{AB} = \left( c_1 + \frac{d}{2} \right)^2 / 2 * (c_1 + c_2 + d) \]
\[ c_{CD} = (c_1 + d/2) - c_{AB} \]
\[ J_c = (c_1 + d/2) \ast \frac{d^3}{12} + d \ast \left( c_{AB}^3 + c_{CD}^3 \right) / 3 + d \ast (c_2 + d/2) c_{AB}^2 \]
\[ \gamma_V = 1 - \frac{1}{1 + \left( \frac{c_2 + d/2}{c_1 + d/2} \right)^2} \]

Where \( c_1 \) and \( c_2 \) are the column dimensions with \( c_1 \) parallel to the axis of moment and \( d \) is the effective depth.

For corner columns (Fig. 1.1-6) the column reaction does not act at the centroid of the critical section. The governing moment for the analysis of the design section is:

\[ M_{ue} = M_u - V_u \ast e \]
(v) Support with Drop Cap (Refer Fig. 1.1-7)

For supports provided with drop caps, or drop panels, a minimum of two punching shear checks are necessary. The first check is at distance \(d_1/2\) from the face of the column, where \(d_1\) is the effective depth of the thickened section (drop cap or drop panel). The second check is at a distance \(d_2/2\) from the face of drop cap/panel, where \(d_2\) is the slab thickness.
1.2. Punching Shear Stress Calculations

In order to keep the focus on punching shear stress calculation, the work starts by assuming that the design values (Mu and Vu) for each column-slab condition are given. In the general case, these are calculated from the analysis of a design strip, using the Equivalent Frame Method, or Finite Elements. The values used in this writing are obtained from an ADAPT-PT computer run. The hand calculations of the stresses are compared with the computer output for verification. Excellent agreement is obtained.

A. Support #1 – corner column (Refer Fig. 1.1-5)

Actions at the joint are:
\[ Vu = 44.95 \text{ kips (199.95 kN)} \]  
\[ \text{(B 10.3, ADAPT PT)} \]

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\(^2\) The values in the parenthesis refer to the location in the program ADAPT-PT, where the information is given. For example B10.3 refers to data Block 10.3.
Mu= 111.67 kip-ft (151.40 kN-m)

i. Section Properties for Shear Stress Computations

Column width, \( c_1 = 24 \) in \((610 \text{ mm})\)
Column depth, \( c_2 = 24 \) in \((610 \text{ mm})\)
Slab depth, \( h = 9 \) in \((229 \text{ mm})\)
Rebar used #5, diameter \( = 0.625 \) in \((16 \text{ mm})\)
Top Cover to rebar \( = 0.75 \) in \((19 \text{ mm})\)
\( d = 9 - 0.75 - 0.625 = 7.625 \) in \((194 \text{ mm})\)

Since top bars in one direction are placed above the top bars in the other direction, the \( d \) value in this case is measured from the bottom of the slab to the bottom of the top layer of rebar.

For corner columns (Fig. 1.2-2) the column reaction does not act at the centroid of the critical section. The governing moment for the analysis of the design section is:

\[ M_{ue} = Mu – Vu* e \]

Where “e” is the eccentricity between the centroid of the column and that of the critical section being considered.

\[ c_1 + d/2 = 24 + (7.625/2) = 27.813 \text{ in} \quad \text{(706 mm)} \]
\[ c_2 + d/2 = 24 + (7.625/2) = 27.813 \text{ in} \quad \text{(706 mm)} \]
\[ A_c = d (c_1 + c_2 + d) = 7.625 * (24 + 24 + 7.625) \]
\[ = 424.14 \text{ in}^2 \quad \text{(2.736e+5 mm}^2) \]
\[ c_{AB} = (c_1 + d/2)^2 / 2 * (c_1 + c_2 + d) \]
\[ = 27.813^2 / (2 * (24 + 24 + 7.625)) \]
\[ = 6.953 \text{ in} \quad \text{(177 mm)} \]
\[ c_{CD} = (c_1 + d/2) - c_{AB} \]
\[ = 27.813 - 6.953 = 20.860 \text{ in} \quad \text{(530 mm)} \]
\[ J_c = (c_1 + d/2)^*d^3/12 + d^*(c_{AB}^3 + c_{CD}^3)/3 + d^* (c_2 + d/2) c_{AB}^2 \]
\[ = 27.813^* 7.625^3/12 + 7.625^* (6.953^3 + 20.860^3)/3 \]
\[ + 7.625^* 27.813^* 6.953^2 \]
\[ = 35,205 \text{ in}^4 \quad \text{(1.465e+10 mm}^4) \]
\[ \gamma_V = 1- \{1/[1+ (2/3) * ((c_2 +d/2) / (c_1 +d/2))^{1/2}]\} \]
\[ = 1- \{1/[1+ (2/3) * (27.813 / 27.813)^{1/2}]\} \]
\[ = 0.40 \]

ii. Stress Due To Direct Shear

\[ Vu / A_c = 44.95 * 1000 / 424 14 \]
\[ = 105.98 \text{ psi} \quad \text{(0.73 MPa)} \]

(ADAPT-PT 105.99 psi, B12, C5)
iii. Stress Due To Bending

For the first support, if the column moment is clockwise, the moment due to shear must be deducted from the column moment.

Eccentricity, \( e = (c_1 + d/2) - c_{AB} - c_1/2 = 27.813 - 6.953 - 12 \) = 8.860 in (225 mm)

\( M_{ue} = 111.67 - 44.95 * 8.860 /12 \) = 78.48 kip-ft (106.40 kN-m)

\( M_{stress} = (\gamma V \cdot M_{ue} \cdot c_{AB})/J_c \) = (0.40 * 78.48 * 12000 * 6.953)/ 35,205 = 74.40 psi (0.51 MPa)

(ADAPT-PT 74.41 psi, B12, C6)

iv. Total Stress

Total Stress = Stress due to shear + stress due to bending

= 105.98 + 74.40

= 180.38 psi (1.24 MPa)

(ADAPT-PT 180.40 psi, B12, C7)

v. Allowable Stress

Column cross section is closer to a discontinuous edge than 4 times the slab thickness. Therefore, according to ACI-318-02 section 11.12.2.2, allowable stress shall be computed according to section 11.12.2.1.

\[ \phi v_c = \phi * (2 + 4/\beta_c)^* \sqrt{f'_c} \]

\[ \phi = 0.75 \]

\[ \beta_c = \text{long side of column/ short side of column} = 24/24 = 1 \]

\[ \therefore \phi v_c = 0.75 * (2 + 4/1)^* \sqrt{4000} = 284.60 \text{ psi} \] (1.96 MPa)

\[ \phi v_c = \phi *((\alpha_s * d/ b_0) + 2)^* \sqrt{f'_c} \]

\[ \alpha_s = 20 \text{ for corner columns} \]

\[ d = 7.625 \text{ in (194 mm)} \]

\[ b_0 = \text{Perimeter of the critical section} = 2 * 27.813 = 55.626 \text{ in (1413 mm)} \]

\[ \phi v_c = 0.75 *((20 * 7.625/ 55.626 ) + 2)^* \sqrt{4000} = 224.91 \text{ psi} \] (1.55 MPa)

\[ \phi v_c = \phi *4^* \sqrt{f'_c} \]

\[ = 0.75 * 4^* \sqrt{4000} = 189.74 \text{ psi (1.31 MPa)} \]

------------------ Controls
Allowable Stress = 189.74 psi (1.31 MPa)  
(ADAPT-PT 189.74 psi, B12, C8)

vi. Stress Ratio

\[
\text{Stress Ratio} = \frac{\text{Actual}}{\text{Allowable}}
\]

\[
= \frac{180.38}{189.74}
\]

\[
= 0.95 < 1 \quad \text{OK}
\]

(ADAPT-PT 0.95, B12, C9)

B. Support #2 – edge column (Refer Fig. 1.1-3)

Actions at the joint are:

\( V_u = 99.66 \text{ kips} \) (443.31 kN)  
\( M_u = 35.46 \text{ kip-ft} \) (48.08 kN-m)

i. Section Properties For Shear Stress Computations

Column width, \( c_1 = 24 \text{ in} \) (610 mm)
Column depth, \( c_2 = 24 \text{ in} \) (610 mm)
Slab depth, \( h = 9 \text{ in} \) (229 mm)
Rebar used #5, diameter = 0.625 in (16 mm)
Top Cover to rebar = 0.75 in (19 mm)
\( d = 9-0.75-0.625 = 7.625 \text{ in} \) (194 mm)

Since top bars in one direction are placed above the top bars in the other direction, the \( d \) value in this case is measured from the bottom of the slab to the bottom of the top layer of rebar.

\[
c_1 + d = 24 + 7.625 = 31.625 \text{ in} \quad (803 \text{ mm})
\]
\[
c_2 + d/2 = 24 + 7.625/2 = 27.813 \text{ in} \quad (706 \text{ mm})
\]
\[
A_c = d (2c_2 + c_1 + 2d) = 7.625 \times (2 \times 24 + 24 + 2 \times 7.625)
\]
\[
= 665.28 \text{ in}^2 \quad (4.292 \times 10^5 \text{ mm}^2)
\]
\[
J_c = (c_1+d)^3 \times d /12 + (c_1 + d) \times d^3/12 + d \times (c_2 + d/2) \times (c_1 + d)^2/2
\]
\[
= 31.625^3 \times 7.625 /12 + 31.625 \times 7.625^3/12 + 7.625 \times 27.813 \times 31.625^2/2
\]
\[
= 127,318 \text{ in}^4 \quad (5.299 \times 10^9 \text{ mm}^4)
\]
\[
\gamma_V = 1 - \{1/[1+ (2/3) * ((c_1 + d) / (c_2 + d/2))]\}
\]
\[
= 1 - \{1/[1+ (2/3) * (31.625 / 27.813)]\}
\]
\[
= 0.416
\]

ii. Stress Due To Direct Shear

\[
V_u / A_c = 99.66 \times 1000/ 665.28
\]
\[
= 149.80 \text{ psi} \quad (1.03 \text{ MPa})
\]

(ADAPT-PT 149.81 psi, B12, C5)
iii. Stress Due To Bending

\[ M_{\text{stress}} = (\gamma \cdot M_{\text{u}} \cdot (c_1 + d))/2 \cdot J_c \]
\[ = (0.416 \cdot 35.46 \cdot 12000 \cdot 31.625)/2 \cdot 127,318 \]
\[ = 21.98 \text{ psi } \quad (0.15 \text{ MPa}) \]
\[ \text{(ADAPT-PT 21.96 psi, B12, C6)} \]

iv. Total Stress

Total Stress = Stress due to shear + stress due to bending
\[ = 149.80 + 21.98 \]
\[ = 171.78 \text{ psi } \quad (1.18 \text{ MPa}) \]
\[ \text{(ADAPT-PT 171.77 psi, B12, C7)} \]

v. Allowable Stress

Column cross section is closer to a discontinuous edge than 4 times the slab thickness. Therefore, according to ACI-318-02 section 11.12.2.2, allowable stress shall be computed according to section 11.12.2.1.

\[ \therefore \text{Allowable stress is the least of } \]

- \[ \phi v_c = \phi * (2 + 4/\beta_c) * \sqrt{f'_c} \]
  \[ \phi = 0.75 \]
  \[ \beta_c = \text{long side of column/ short side of column} \]
  \[ = 24/24 = 1 \]
  \[ \therefore \phi v_c = 0.75 * (2 + 4/1) * \sqrt{4000} \]
  \[ = 284.60 \text{ psi (1.96 MPa)} \]

- \[ \phi v_c = \phi * (\alpha_s \cdot d/b_0 + 2) * \sqrt{f'_c} \]
  \[ \alpha_s = 30 \text{ for edge column} \]
  \[ d = 7.625 \text{ in (194 mm)} \]
  \[ b_0 = \text{Perimeter of the critical section} \]
  \[ = 2 \cdot 27.813 + 31.625 = 87.251 \text{ in (2216 mm)} \]
  \[ \phi v_c = 0.75 * ((30 \cdot 7.625/87.251) + 2) * \sqrt{4000} \]
  \[ = 219.23 \text{ psi (1.51 MPa)} \]

- \[ \phi v_c = \phi * 4 * \sqrt{f'_c} \]
  \[ = 0.75 * 4 * \sqrt{4000} \]
  \[ = 189.74 \text{ psi (1.31 MPa)} \quad \text{----------- Controls} \]

\[ \therefore \text{Allowable Stress } = \textbf{189.74 psi} \quad (1.31 \text{ MPa}) \]
\[ \text{(ADAPT-PT 189.74 psi, B12, C8)} \]

vi. Stress Ratio

Stress Ratio = Actual / Allowable
\[ = 171.78 / 189.74 \]
C. Support #3 – edge column (Refer Fig. 1.1-4)

Actions at the joint are:

\[ \begin{align*}
V_u &= 155.11 \text{ kips} \quad (689.96 \text{ kN}) \\
M_u &= 172.45 \text{ kip-ft} \quad (233.81 \text{ kN-m})
\end{align*} \]  

(B10.3, ADAPT PT)

i. Section Properties For Shear Stress Computations

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column width, ( c_1 )</td>
<td>28 in (711 mm)</td>
</tr>
<tr>
<td>Column depth, ( c_2 )</td>
<td>28 in (711 mm)</td>
</tr>
<tr>
<td>Slab depth, ( h )</td>
<td>9 in (229 mm)</td>
</tr>
<tr>
<td>Rebar used #5, diameter</td>
<td>0.625 in (16 mm)</td>
</tr>
<tr>
<td>Top Cover to rebar</td>
<td>0.75 in (19 mm)</td>
</tr>
</tbody>
</table>

Since top bars in one direction are placed above the top bars in the other direction, the \( d \) value in this case is measured from the bottom of the slab to the bottom of the top layer of rebar.

\[
\begin{align*}
\text{c}_1 + d &= 28 + 7.625 = 35.625 \text{ in} \quad (905 \text{ mm}) \\
\text{c}_2 + d/2 &= 28 + 7.625/2 = 31.813 \text{ in} \quad (808 \text{ mm}) \\
Ac &= d (2\text{c}_2 + \text{c}_1 + 2d) = 7.625 \times (2 \times 28 + 28 + 2 \times 7.625) \\
&= 756.78 \text{ in}^2 \quad (4.882 \times 10^5 \text{ mm}^2) \\
J_c &= \frac{(\text{c}_1 + d)^3 \times d}{12} + \frac{\text{c}_1 + d \times d^3}{12} \\
&+ \frac{\text{d}^3}{2} \\
&= 35.625^3 \times 7.625 / 12 + 35.625 \times 7.625^3 / 12 \\
&+ 7.625 \times 31.813^3 \\
&= 183,976 \text{ in}^4 \quad (7.658 \times 10^9 \text{ mm}^4) \\
\gamma_V &= 1 - \left\{ 1 + \left( \frac{2}{3} \right) \times ((\text{c}_1 + d) / (\text{c}_2 + d/2))^{\gamma_V} \right\} \\
&= 1 - \left\{ 1 + \left( \frac{2}{3} \right) \times \left( \frac{35.625}{31.813} \right)^{0.414} \right\} \\
&= 0.414
\end{align*}
\]

ii. Stress Due To Direct Shear

\[
\frac{V_u}{Ac} = \frac{155.11 \times 1000}{756.78} = 204.96 \text{ psi} \quad (1.41 \text{ MPa})
\]  

(ADAPT-PT 204.96 psi, B12, C5)

iii. Stress Due To Bending

\[
M_{\text{stress}} = \left( \gamma_V \cdot M_u \cdot (\text{c}_1 + d) \right) / 2 \cdot J_c
\]

\[
= (0.414 \times 172.45 \times 12000 \times 35.625) / 2 \times 183,976
\]

\[
= 82.95 \text{ psi} \quad (0.57 \text{ MPa})
\]  

(ADAPT-PT 82.88 psi, B12, C6)
iv. **Total Stress**

Total Stress  =  Stress due to shear + stress due to bending  
=  204.96 + 82.95  
=  **287.91 psi**  
\(\text{(1.99 MPa)}\)  
(ADAPT-PT 287.85 psi, B12, C7)

v. **Allowable Stress**

Column cross section is closer to a discontinuous edge than 4 times the slab thickness. Therefore, according to ACI-318-02 section 11.12.2.2, allowable stress shall be computed according to section 11.12.2.1.

\[\phi v_c = \phi (2 + 4/\beta_c) \sqrt{f'_c}\]
\[\phi = 0.75\]
\[\beta_c = \text{long side of column/ short side of column} = 28/28 = 1\]
\[\therefore \phi v_c = 0.75 (2 + 4/1) \sqrt{4000} = 284.60 \text{ psi} \quad \text{(1.96 MPa)}\]

\[\phi v_c = \phi ((\alpha_s * d/ b_0) + 2) \sqrt{f'_c}\]
\[\alpha_s = 30 \text{ for edge column}\]
\[d = 7.625 \text{ in (194 mm)}\]
\[b_0 = \text{Perimeter of the critical section} = 2 * 31.813 +35.625 = 99.251 \text{ in (2521 mm)}\]
\[\phi v_c = 0.75 ((30 * 7.625/ 99.251 )+ 2) \sqrt{4000} = 204.19 \text{ psi} \quad \text{(1.41 MPa)}\]

\[\phi v_c = \phi 4 \sqrt{f'_c}\]
\[= 0.75 * 4 \sqrt{4000}\]
\[= 189.74 \text{ psi} \quad \text{(1.31 MPa)}\]  
----------- Controls

\[\therefore \text{Allowable Stress} = 189.74 \text{ psi} \quad \text{(1.31 MPa)}\]

(ADAPT-PT 189.74 psi, B12, C8)

vi. **Stress Ratio**

\[
\text{Stress Ratio} = \frac{\text{Actual}}{\text{Allowable}} = \frac{287.91}{189.74} = 1.52 > 1 \quad \text{N.G}\]

(ADAPT-PT 1.52, B12, C9)

For \(4\sqrt{f'}c\) allowable stress, according to ACI-318-02 section 11.12.3.2, the maximum allowed is 1.5 times the permissible value. Therefore enlarge the section resisting the punching shear.
D. Support #4 – interior column (Refer Fig.1.1-1)

Actions at the joint are:

\[Vu = 198.21 \text{ kips (881.68 kN)}\] (B10.3, ADAPT PT)
\[Mu = 29.07 \text{ kip-ft (39.41 kN-m)}\]

i. Section Properties For Shear Stress Computations

- Column width, \(c_1 = 24 \text{ in} \) (610 mm)
- Column depth, \(c_2 = 24 \text{ in} \) (610 mm)
- Slab depth, \(h = 9 \text{ in} \) (229 mm)
- Rebar used #5, diameter = 0.625 in (16 mm)
- Top Cover to rebar = 0.75 in (19 mm)

\[d = 9 - 0.75 - 0.625 = 7.625 \text{ in} \] (194 mm)

Since top bars in one direction are placed above the top bars in the other direction, the \(d\) value in this case is measured from the bottom of the slab to the bottom of the top layer of rebar.

\[c_1 + d = 24 + 7.625 = 31.625 \text{ in} \] (803 mm)
\[c_2 + d = 24 + 7.625 = 31.625 \text{ in} \] (803 mm)
\[Ac = 2d(c_1 + c_2 + 2d) = 2 \times 7.625 \times (24 + 24 + 2 \times 7.625) = 964.56 \text{ in}^2 \] (6.223e+5 mm²)
\[Jc = (c_1 + d)^3/6 + (c_1 + d)^3/6 + d^2 + 31.625^3/6 + 31.625^3/6 + 7.625^2 \times 31.625^2 \] /2
\[= 163,120 \text{ in}^4 \] (6.790e+10 mm⁴)

\[\gamma_V = 1 - [1/(1+ 2/3) \times (c_1 + d)/(c_2 + d)]^{1/2}\]
\[= 0.40\]

ii. Stress Due To Direct Shear

\[Vu/ Ac = 198.21 \times 1000/ 964.56 = 205.49 \text{ psi} \] (1.42 MPa)
(ADAPT-PT 205.49 psi, B12, C5)

iii. Stress Due To Bending

\[M_{\text{stress}} = (\gamma_V \times Mu \times (c_1 + d))/ (2 \times Jc)\]
\[= (0.40 \times 29.07 \times 12000 \times 31.625)/ 2 \times 163,120 = 13.53 \text{ psi} \] (0.09 MPa)
(ADAPT-PT 13.53 psi, B12, C6)

iv. Total Stress

Total Stress = Stress due to shear + stress due to bending
\[= 205.49 + 13.53 \]
v. Allowable Stress

From ACI-318-02 equation 11.36

Allowable Stress,

\[ \phi v_c = \phi \left[ \beta_p \sqrt{f'c} + 0.3 f_{pc} + V_p \right] \]

Where,

\[ \phi = 0.75 \]

\[ \beta_p \text{ is the smaller of } 3.5 \text{ or } \left( \alpha_s \frac{d}{b_0} + 1.5 \right) \]

\[ \alpha_s = 40 \text{ for interior column} \]

\[ b_0 = \text{Perimeter of the critical section} \]

\[ = 4 \times 31.625 \]

\[ = 126.50 \text{ in} \] (3213 mm)

\[ d = 7.625 \text{ in} \] (194 mm)

\[ \beta_p = \left( \alpha_s \frac{d}{b_0} + 1.5 \right) = \left( 40 \times 7.625 / 126.50 \right) + 1.5 \]

\[ = 3.91 > 3.50, \quad \therefore \text{use } 3.50 \]

\[ f_{pc} = \frac{P}{A} = 125.03 \text{ psi} \] (0.86 MPa) \[ (\text{ADAPT-PT B 9.3}) \]

\[ \phi v_c = 0.75 \left( 3.5 \sqrt{4000} + 0.3 \times 125.03 \right) \]

\[ = 194.15 \text{ psi} \] (1.34 MPa)

\[ \therefore \text{Allowable Stress} = 194.15 \text{ psi} \] (1.34 MPa) \[ (\text{ADAPT-PT 194.15 psi, B12, C8}) \]

Note that in the evaluation of allowable stresses, the term corresponding to the vertical component of tendon force (Vp) is conservatively disregarded.

vi. Stress Ratio

Stress Ratio = Actual / Allowable

\[ = 219.02 / 194.15 \]

\[ = 1.13 > 1 \quad \text{N.G} \]

\[ (\text{ADAPT-PT 1.13, B12, C9}) \]

Punching Shear Stress exceeds the permissible value. Provide shear reinforcement.

E. Support #5 – interior column with drop cap (Refer Fig.1.1-7)

Actions at the joint are:

\[ V_u = 212.75 \text{ kips (946.35 kN)} \] \[ (B10.3, \text{ADAPT PT}) \]

\[ M_u = 35.95 \text{ kip-ft (48.74 kN-m)} \]
Section #1  (d/2 from the column face)

i. Section Properties For Shear Stress Computations

Column width, \( c_1 = 18 \) in (457 mm)
Column depth, \( c_2 = 18 \) in (457 mm)
Slab depth, \( h = 9 +9 = 18 \) in (457 mm)
Rebar used #5, diameter = 0.625 in (16 mm)
Top Cover to rebar = 0.75 in (19 mm)
\( d_1 = 18 - 0.75 - 0.625 = 16.625 \) in (422 mm)

Since top bars in one direction are placed above the top bars in the other direction, the \( d_1 \) value in this case is measured from the bottom of the drop panel to the bottom of the top layer of rebar.

\[
\begin{align*}
c_1 + d_1 &= 18 + 16.625 = 34.625 \text{ in} \quad (880 \text{ mm}) \\
c_2 + d_1 &= 18 + 16.625 = 34.625 \text{ in} \quad (880 \text{ mm}) \\
Ac &= 2d(c_1 + c_2 + 2d) = 2 \times 16.625 \times (18+18+2\times16.625) \\
&= 2302.56 \text{ in}^2 \quad (1.486e+6 \text{ mm}^2) \\
J_c &= (c_1+ d)^3/6 + (c_1 + d)^3d/6 +d^3 (c_1 + d)^2 /2 \\
&= 34.625^3 \times 16.625 \times (c_1 + d)^3/6 \times 34.625^3 \times 16.625^2 /6 + 16.625 \times 34.625^3 \times 34.625^2 \times 2 \\
&= 486,604 \text{ in}^4 \quad (2.025e+11 \text{ mm}^4) \\
\gamma_v &= 1 - \{1/[1+ (2/3) * ((c_1 + d) / (c_2 + d))^{3/2}]\} \\
&= 1 - \{1/[1+ (2/3) * (34.625 / 34.625)^{3/2}]\} \\
&= 0.40
\end{align*}
\]

ii. Stress Due To Direct Shear

\[
Vu / Ac = 212.75 \times 1000 / 2302.56 \\
= 92.40 \text{ psi} \quad (0.64 \text{ MPa})
\]

iii. Stress Due To Bending

\[
M_{\text{stress}} = (\gamma_v \cdot M_u \cdot (c_1 + d)) / (2 \times J_c) \\
= (0.40 \times 35.95 \times 12000 \times 34.625) / 2 \times 486,604 \\
= 6.14 \text{ psi} \quad (0.04 \text{ MPa})
\]

iv. Total Stress

Total Stress = Stress due to shear + stress due to bending
= 92.40 + 6.14
= 98.54 \text{ psi} \quad (0.68 \text{ MPa})

v. Allowable Stress
From ACI-318-02 (equation 11.36)

Allowable Stress,

\[ \phi v_c = \phi \times [\beta_p \sqrt{f'_c + 0.3 \times f_{pc}} + V_p] \]

Where,

\[ \phi = 0.75 \]
\[ \beta_p \text{ is the smaller of 3.5 or (} \alpha_s \times \frac{d}{b_0} + 1.5) \]
\[ \alpha_s = 40 \text{ for interior column} \]
\[ b_0 = \text{Perimeter of the critical section} \]
\[ = 4 \times 34.625 \]
\[ = 138.50 \text{ in (3518 mm)} \]
\[ d = 16.625 \text{ in (422 mm)} \]
\[ \beta_p = ((\alpha_s \times d/b_0) + 1.5) = ((40 \times 16.625 / 138.50) + 1.5) \]
\[ = 6.30 >3.50, \therefore \text{use 3.50} \]
\[ f_{pc} = \frac{P}{A} = 125.03 \text{ psi (0.86 MPa)} \quad \text{(ADAPT-PT B 9.3)} \]

\[ \phi v_c = 0.75 \times (3.5 \times \sqrt{4000} + 0.3 \times 125.03) \]
\[ = 194.15 \text{ psi (1.34 MPa)} \]

\[ \therefore \text{Allowable Stress} = 194.15 \text{ psi (1.34 MPa)} \]

Note that in the evaluation of allowable stresses, the term corresponding to the vertical component of tendon force \((V_p)\) is conservatively disregarded.

vi. Stress Ratio

\[
\text{Stress Ratio} = \frac{\text{Actual}}{\text{Allowable}} = \frac{98.56}{194.15} = 0.51
\]

Section #2 (d/2 from the drop cap face)

i. Section Properties For Shear Stress Computations

Cap width, \(c_1 = 45 \text{ in (1143 mm)}\)
Cap depth, \(c_2 = 45 \text{ in (1143 mm)}\)
Slab depth, \(h = 9 \text{ in (229 mm)}\)
Rebar used #5, diameter = 0.625 in (16 mm)
Top Cover to rebar = 0.75 in (19 mm)
\[ d_2 = 9 - 0.75 - 0.625 = 7.625 \text{ in (194 mm)} \]

Since top bars in one direction are placed above the top bars in the other direction, the \(d_2\) value in this case is measured from the bottom of the slab to the bottom of the top layer of rebar.

\[ c_{1\text{CAP}} + d_2 = 45 + 7.625 = 52.625 \text{ in (1337 mm)} \]
\[ c_2 \text{CAP} + d_2 = 45 + 7.625 = 52.625 \text{ in} \quad (1337 \text{ mm}) \]

\[ A_c = 2d(c_1 + c_2 + 2d) = 2 \times 7.625 \times (45 + 45 + 2 \times 7.625) \]
\[ = 1605.06 \text{ in}^2 \quad (1.036e+6 \text{ mm}^2) \]

\[ J_c = (c_1 + d)^3/6 + (c_2 + d)^3/6 + d(c_1 + d)(c_2 + d)/2 \]
\[ = (52.625 \times 7.625)^3/6 + (52.625 \times 7.625)^3/6 + (7.625 \times 52.625 \times 52.625)/2 \]
\[ = 744,729 \text{ in}^4 \quad (3.100e+11 \text{ mm}^4) \]

\[ \gamma_v = 1 - \left\{1/\left[1 + (2/3) \times \left( (c_1 + d) / (c_2 + d) \right)^{1/2} \right] \right\} \]
\[ = 0.40 \]

ii. Stress Due To Direct Shear

\[ V_u / A_c = 212.75 \times 1000 / 1605.06 \]
\[ = 132.55 \text{ psi} \quad (0.91 \text{ MPa}) \]

(ADAPT-PT 132.55 psi, B12, C5)

iii. Stress Due To Bending

\[ M_{stress} = (\gamma_v \cdot M_u \cdot (c_1 + d)) / (2 \cdot J_c) \]
\[ = (0.40 \times 35.95 \times 12000 \times 52.625) / 2 \times 744,729 \]
\[ = 6.10 \text{ psi} \quad (0.04 \text{ MPa}) \]

(ADAPT-PT 6.10 psi, B12, C6)

iv. Total Stress

Total Stress = Stress due to shear + stress due to bending
\[ = 132.55 + 6.10 \]
\[ = 138.65 \text{ psi} \quad (0.96 \text{ MPa}) \]

(ADAPT-PT 138.65 psi, B12, C7)

v. Allowable Stress

From ACI-318-02 equation 11.36

Allowable Stress,
\[ \phi \nu_c = \phi \times [f'c + 0.3 \times f_{pc} + V_p] \]

Where,
\[ \phi = 0.75 \]
\[ \beta_p \text{ is the smaller of 3.5 or } (\alpha_s \times d / b_0) + 1.5 \]
\[ \alpha_s = 40 \text{ for interior column} \]
\[ b_0 = \text{Perimeter of the critical section} \]
\[ = 4 \times 52.625 \]
\[ = 210.50 \text{ in} \quad (5347 \text{ mm}) \]
\[ d = 7.625 \text{ in} \quad (194 \text{ mm}) \]
\[ \beta_p = (\alpha_s \times d / b_0) + 1.5 \]
\[ = (40 \times 7.625 / 210.50) + 1.5 \]
\[ f_{pc} = \frac{P}{A} = 125.03 \text{ psi (0.86 MPa)} \quad (\text{ADAPT B9.3}) \]

\[ \phi v_c = 0.75 \times (2.95 \times \sqrt{4000} + 0.3 \times 125.03) \]
\[ = 168.06 \text{ psi} \]
\[ (1.16 \text{ MPa}) \]

Allowable Stress = 168.06 psi \quad (1.16 \text{ MPa}) \quad (\text{ADAPT-PT 168.01 psi, B12, C8})

Note that in the evaluation of allowable stresses, the term corresponding to the vertical component of tendon force (Vp) is conservatively disregarded.

vi. Stress Ratio

Stress Ratio = Actual / Allowable
\[ = \frac{138.65}{168.06} \]
\[ = 0.83 < 1 \quad \text{OK} \quad (\text{ADAPT-PT 0.83, B12, C9}) \]

Since the stress ratio in section #2 is larger than the stress ratio in section #1, the section #2 governs and reported in the program.

F. Support #6 – end column (Refer Fig. 1.1-2)

Actions at the joint are:
\[ V_u = 100.97 \text{ kips (449.13 kN)} \quad (B10.3, \text{ADAPT PT}) \]
\[ M_u = 338.23 \text{kip-ft (458.57 kN-m)} \]

i. Section Properties For Shear Stress Computations

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column width, ( c_1 )</td>
<td>28 in (711 mm)</td>
</tr>
<tr>
<td>Column depth, ( c_2 )</td>
<td>28 in (711 mm)</td>
</tr>
<tr>
<td>Slab depth, ( h )</td>
<td>9 in (229 mm)</td>
</tr>
<tr>
<td>Rebar used #5, diameter</td>
<td>0.625 in (16 mm)</td>
</tr>
<tr>
<td>Top Cover to rebar</td>
<td>0.75 in (19 mm)</td>
</tr>
<tr>
<td>( d )</td>
<td>9-0.75-0.625 = 7.625 in (194 mm)</td>
</tr>
</tbody>
</table>

Since top bars in one direction are placed above the top bars in the other direction, the \( d \) value in this case is measured from the bottom of the slab to the bottom of the top layer of rebar.

\[ c_1 + \frac{d}{2} = 28 + 7.625/2 = 31.813 \text{ in (808 mm)} \]
\[ c_2 + d = 28 + 7.625 = 35.625 \text{ in (905 mm)} \]

\[ A_c = d (2c_1 + c_2 + 2d) = 7.625 \times (2 \times 28 + 28 + 2 \times 7.625) \]
\[ = 756.78 \text{ in}^2 \quad (4.882e+5 \text{ mm}^2) \]

\[ c_{AB} = \frac{(c_1 + d/2)^2}{(2c_1 + c_2 + 2d)} \]
\[ = 31.813^2 / (2 \times 28 + 28 + 2 \times 7.625) \]
Technical Note

\[ c_{CD} = 31.813 - 10.200 = 21.613 \text{ in} \]  
\[ J_c = 31.813 \times 7.625 \times 3 / 6 + 2 \times 7.625 \times (10.200^3 + 21.613^3) / 3 + 7.625 \times 35.625 \times 10.200^2 \]  
\[ = 87,327 \text{ in}^4 \] (3.635e+10 mm^4)

\[ \gamma V = 1 - \{1/[1+ (2/3) \times ((c_1 + d/2) / (c_2 + d))^{1/2}]\} \]  
\[ = 1 - \{1/[1+ (2/3) \times (31.813 / 35.625)^{1/2}]\} \]  
\[ = 0.386 \]

ii. Stress Due To Direct Shear

\[ V_u / A_c = 100.97 \times 1000 / 756.78 = 133.42 \text{ psi} \]  
(ADAPT-PT 133.42 psi, B12, C5)

iii. Stress Due To Bending

\[ M_{ue} = M_u - V_u \times e \]

For the last support, if the column moment is anticlockwise, the moment due to shear must be deducted.

\[ \text{Eccentricity, } e = (c_1 + d/2) - c_{AB} - c_1/2 = 31.813 - 10.200 - 14 \]  
\[ = 7.613 \text{ in} \] (193 mm)
\[ M_{ue} = 338.23- 100.97 \times 7.613 /12 \]  
\[ = 274.17 \text{ kip-ft} \] (371.72 kN-m)
\[ M_{stress} = (\gamma V \times M_{ue} \times c_{AB}) / J_c \]  
\[ = (0.386 \times 274.18 \times 12000 \times 10.200) / 87,327 \]  
\[ = 148.33 \text{ psi} \]  
(ADAPT-PT 148.48 psi, B12, C6)

iv. Total Stress

Total Stress = Stress due to shear + stress due to bending  
\[ = 133.42 + 148.33 \]  
\[ = 281.75 \text{ psi} \]  
(ADAPT-PT 281.89 psi, B12, C7)

v. Allowable Stress

Column cross section is closer to a discontinuous edge than 4 times the slab thickness. Therefore, according to ACI-318-02 section 11.12.2.2, allowable stress shall be computed according to section 11.12.2.1.

\[ \therefore \text{Allowable stress is the least of} \]

- \[ \phi \nu_c = \phi \times (2 + 4/\beta_c)^{1/2} f'_c \]  
\[ \phi = 0.75 \]
\[ \beta_c = \frac{\text{long side of column}}{\text{short side of column}} = \frac{28}{28} = 1 \]

\[ \therefore \phi v_c = 0.75 \times (2 + \frac{4}{1}) \times \sqrt{4000} = 284.60 \text{ psi} \quad (1.96 \text{ MPa}) \]

- \[ \phi v_c = \phi \times ((\alpha_s \times d / b_0) + 2) \times \sqrt{f'_c} \]
  \[ \alpha_s = 30 \text{ for end column} \]
  \[ d = 7.625 \text{ in} \quad (194 \text{ mm}) \]
  \[ b_0 = \text{Perimeter of the critical section} \]
  \[ = 2 \times 31.813 + 35.625 \]
  \[ = 99.251 \text{ in} \quad (2521 \text{ mm}) \]

\[ \phi v_c = 0.75 \times ((30 \times 7.625 / 99.251) + 2) \times \sqrt{4000} = 204.19 \text{ psi} \quad (1.41 \text{ MPa}) \]

- \[ \phi v_c = \phi \times 4 \times \sqrt{f'_c} \]
  \[ = 0.75 \times 4 \times \sqrt{4000} \]
  \[ = 189.74 \text{ psi} \quad (1.31 \text{ MPa}) \]

\[ \therefore \text{Allowable Stress} = 189.74 \text{ psi} \quad (1.31 \text{ MPa}) \]

(ADAPT-PT 189.74 psi, B12, C8)

vi. **Stress Ratio**

\[ \text{Stress Ratio} = \frac{\text{Actual}}{\text{Allowable}} \]

\[ = 281.75 / 189.74 \]

\[ = 1.48 > 1 \quad \text{N.G} \]

(ADAPT-PT 1.49, B12, C9)

Punching Shear Stress exceeds the permissible value. Provide shear reinforcement.
G. ADAPT OUTPUT

---

ADAPT-PT FOR POST-TENSIONED BEAM/SLAB DESIGN

Version 7.00  AMERICAN (ACI 318-02/IBC-03)

ADAPT CORPORATION - Structural Concrete Software System

1733 Woodside Road, Suite 220, Redwood City, California 94061

Phone: (650)306-2400, Fax: (650)364-4678

Email: info@AdaptSoft.com, Web site: http://www.AdaptSoft.com

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DATE AND TIME OF PROGRAM EXECUTION: At Time: 9:43
PROJECT FILE: Punch_US

PROJECT TITLE:
Two-Way Post Tensioned Floor System

1 - USER SPECIFIED  GENERAL DESIGN PARAMETERS

CONCRETE:
STRENGTH at 28 days, for BEAMS/SLABS .......... 4000.00 psi
for COLUMNS .............. 4000.00 psi

MODULUS OF ELASTICITY for BEAMS/SLABS .......... 3605.00 ksi
for COLUMNS ............. 3605.00 ksi

CREEP factor for deflections for BEAMS/SLABS ..... 2.00

CONCRETE WEIGHT .................................. NORMAL

SELF WEIGHT ........................................ 150.00 pcf

TENSION STRESS limits (multiple of (f'c)1/2)
At Top ............................................. 6.000
At Bottom .......................................... 6.000

COMPRESSION STRESS limits (multiple of (f'c))
At all locations ................................... .450

REINFORCEMENT:
YIELD Strength ...................................... 60.00 ksi
Minimum Cover at TOP ................................ .75 in
Minimum Cover at BOTTOM .......................... .75 in

POST-TENSIONING:
SYSTEM .............................................. UNBONDED
Ultimate strength of strand ......................... 270.00 ksi
Average effective stress in strand (final) ...... 175.00 ksi
Strand area........................................... .153 in^2
Min CGS of tendon from TOP........................ 1.00 in
Min CGS of tendon from BOTTOM for INTERIOR spans.. 1.00 in
Min CGS of tendon from BOTTOM for EXTERIOR spans.. 1.00 in
Min average precompression ......................... 125.00 psi
Max spacing between strands (factor of slab depth) 8.00
Tendon profile type and support widths............ (see section 9)

ANALYSIS OPTIONS USED:
Structural system ................................... TWO-WAY
Moment of Inertia over support is .................. NOT INCREASED
Moments REDUCED to face of support .............. YES
Limited plastification allowed (moments redistributed) NO

2 - INPUT GEOMETRY
______________________________________________________________

2.1.1 PRINCIPAL SPAN DATA OF UNIFORM SPANS

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LEGEND:
1 - SPAN
C = Cantilever

2.1.5 - DROP CAP AND DROP PANEL DATA

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<td>.00</td>
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</tbody>
</table>

LEGEND:
DROP CAP DIMENSIONS:
CAPT = Total depth of cap
CAPB = Transverse Width
CAPDL = Extension left of joint
CAPDR = Extension right of joint

DROP PANEL DIMENSIONS:
DROPTL = Total depth left of joint
DROPTR = Total depth right of joint
DROPB = Transverse Width
DROPL = Extension left of joint
DROPR = Extension right of joint

2.2 - SUPPORT WIDTH AND COLUMN DATA

SUPPORT <-------- LOWER COLUMN --------> <-------- UPPER COLUMN -------->
WIDTH    LENGTH   B(DIA)    D    CBC*    LENGTH   B(DIA)    D   CBC*
JOINT    in        ft      in      in              ft      in      in
--1-------2---------3-------4-------5-----6---------7-------8-------9----10---
1     24.00     10.00   24.00   24.00  (1)      10.00   24.00   24.00  (1)
2     24.00     10.00   24.00   24.00  (1)      10.00   24.00   24.00  (1)
3     28.00     10.00   28.00   28.00  (1)      10.00   28.00   28.00  (1)
4     24.00     10.00   24.00   24.00  (1)      10.00   24.00   24.00  (1)
5     18.00     10.00   18.00   18.00  (1)      10.00   18.00   18.00  (1)
6     28.00     10.00   28.00   28.00  (1)      10.00   28.00   28.00  (1)

*THE COLUMN BOUNDARY CONDITION CODES (CBC)
Fixed at both ends ... (STANDARD) .................................. = 1
Hinged at near end, fixed at far end ............................. = 2
Fixed at near end, hinged at far end ............................ = 3
Fixed at near end, roller with rotational fixity at far end .. = 4

3.1 - LOADING AS APPEARS IN USER`S INPUT SCREEN PRIOR TO PROCESSING
==============================================================================

UNIFORM (k/ft^2),    ( CON. or PART. )    ( M O M E N T )
SPAN  CLASS  TYPE   LINE(k/ft)   ( k@ft or ft-ft )    ( k-ft  @  ft )
-1-----2------3---------4------------5-------6-----------7-------8------------
CANT   L      U         .040
CANT   D      U         .020
1     L      U         .040
1     D      U         .020
2     L      U         .040
2     D      U         .020
3     L      U         .040
3     D      U         .020
4     L      U         .040
4     D      U         .020
5     L      U         .040
5     D      U         .020
CANT   L      U         .040
CANT   D      U         .020

NOTE: SELFWEIGHT INCLUSION REQUIRED

9 - SELECTED POST-TENSIONING FORCES AND TENDON PROFILES
==============================================================================

9.1 PROFILE TYPES AND PARAMETERS

LEGEND:
For Span:
  1 = reversed parabola
  2 = simple parabola with straight portion over support
  3 = harped tendon

For Cantilever:
  1 = simple parabola
  2 = partial parabola
  3 = harped tendon

9.2 T E N D O N        P R O F I L E
TYPE     X1/L       X2/L       X3/L       A/L
----------1--------2---------3---------4--------5------
### 9.3 - SELECTED POST-TENSIONING FORCES AND TENDON DRAPE

Tendon editing mode selected: FORCE SELECTION

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<th>Right</th>
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<th>Wbal</th>
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Approximate weight of strand ......................... 1056.4 LB

### 10.3 FACTORED REACTIONS

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<th>min</th>
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### 12 - PUNCHING SHEAR CHECK

LEGEND:

- **CONDITION**
  - 1 = INTERIOR COLUMN
  - 2 = END COLUMN
  - 3 = CORNER COLUMN
  - 4 = EDGE COLUMN (PARALLEL TO SPAN)
  - 5 = EDGE BEAM, WALL, OR OTHER NON-CONFORMING GEOMETRY
    - PERFORM SHEAR CHECK MANUALLY
  - 6 = STRIP TOO NARROW TO DEVELOP PUNCHING SHEAR

- **CASE**
  - 1 = STRESS WITHIN SECTION #1 GOVERNS (COL.CAP OR SLAB)
  - 2 = STRESS WITHIN SECTION #2 GOVERNS (DROP PANEL OR SLAB)

FACTORED ACTIONS <- PUNCHING SHEAR STRESSES IN psi->

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PUNCHING SHEAR STRESS IN ONE OR MORE LOCATIONS EXCEEDS THE PERMISSIBLE VALUE. PROVIDE SHEAR REINFORCEMENT, OR ENLARGE THE SECTION RESISTING THE PUNCHING SHEAR.

Comments:
Where stress ratios exceed 1.00, punching shear reinforcement must be provided. If a stress ratio exceeds 1.50, the section has to be enlarged, or re-designed such as to bring the ratio to 1.50 or less. In this case, column 3 has been conservatively modeled as an “edge column.” Its punching shear capacity is larger than assumed in the program. For this reason, it is acceptable if reinforced.
H. ADAPT-SI OUTPUT

| ADAPT-PT FOR POST-TENSIONED BEAM/SLAB DESIGN | Version 7.00 AMERICAN (ACI 318-02/IBC-03) |
| ADAPT CORPORATION - Structural Concrete Software System |
| 1733 Woodside Road, Suite 220, Redwood City, California 94061 |
| Phone: (650)306-2400, Fax: (650)364-4678 |
| Email: Support@AdaptSoft.com, Web site: http://www.AdaptSoft.com |

DATE AND TIME OF PROGRAM EXECUTION: At Time: 15:7
PROJECT FILE: Punch_SI

PROJECT TITLE:
TWO-WAY POST-TENSIONED FLOOR SYSTEM
Punch_SI

1 - USER SPECIFIED GENERAL DESIGN PARAMETERS

CONCRETE:
STRENGTH at 28 days, for BEAMS/SLABS ............. 28.00 N/mm^2
for COLUMNS ............................... 28.00 N/mm^2

MODULUS OF ELASTICITY for BEAMS/SLABS ............ 24870.00 N/mm^2
for COLUMNS ............................... 24870.00 N/mm^2

CREEP factor for deflections for BEAMS/SLABS ...... 2.00

CONCRETE WEIGHT .................................. NORMAL

SELF WEIGHT ........................................ 2400.00 Kg/m^3

TENSION STRESS limits (multiple of (f'_c)1/2)
At Top .......................................... .498
At Bottom ....................................... .498

COMPRESSION STRESS limits (multiple of (f'_c))
At all locations ..................................... .450

REINFORCEMENT:
YIELD Strength ................................... 413.69 N/mm^2
Minimum Cover at TOP ......................... 19.05 mm
Minimum Cover at BOTTOM ...................... 19.05 mm

POST-TENSIONING:
SYSTEM ........................................... UNBONDED
Ultimate strength of strand ....................... 1863.00 N/mm^2
Average effective stress in strand (final) ....... 1206.60 N/mm^2
Strand area....................................... 99.000 mm^2
Min CGS of tendon from TOP ..................... 25.00 mm
Min CGS of tendon from BOTTOM for INTERIOR spans.. 25.00 mm
Min CGS of tendon from BOTTOM for EXTERIOR spans.. 25.00 mm
Min average precompression ....................... .86 N/mm^2
Max spacing between strands (factor of slab depth) 8.00
Tendon profile type and support widths............. (see section 9)

ANALYSIS OPTIONS USED:
Structural system ................................ TWO-WAY
Moment of Inertia over support is ................. NOT INCREASED
Moments REDUCED to face of support .............. YES
Limited plastification allowed (moments redistributed) NO

2 - INPUT GEOMETRY

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<th>F</th>
<th>TOP</th>
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LEGEND:
1 = Rectangular section
2 = T or Inverted L section
3 = I section
4 = Extended T or L section
7 = Joist
8 = Waffle
11 = Top surface to reference line

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LEGEND:
CAPT = Total depth of cap
CAPB = Transverse Width
CAPDL = Extension left of joint
CAPDR = Extension right of joint
DROPTL = Total depth left of joint
DROPTR = Total depth right of joint
DROPB = Transverse Width
DROPL = Extension left of joint
DROPR = Extension right of joint
### 2.2 - Support Width and Column Data

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<td>D CBC*</td>
<td>LENGTH B(DIA)</td>
<td>D CBC*</td>
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*THE COLUMN BOUNDARY CONDITION CODES (CBC)*

- Fixed at both ends (STANDARD) = 1
- Hinged at near end, fixed at far end = 2
- Fixed at near end, hinged at far end = 3
- Fixed at near end, roller with rotational fixity at far end = 4

### 3 - Input Applied Loading

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NOTE: SELFWEIGHT INCLUSION REQUIRED
9 - SELECTED POST-TENSIONING FORCES AND TENDON PROFILES
==============================================================================

9.1 PROFILE TYPES AND PARAMETERS

LEGEND:
For Span:
1 = reversed parabola
2 = simple parabola with straight portion over support
3 = harped tendon

For Cantilever:
1 = simple parabola
2 = partial parabola
3 = harped tendon

<table>
<thead>
<tr>
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9.2 TENDON PROFILE

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9.3 - SELECTED POST-TENSIONING FORCES AND TENDON DRAPE
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Approximate weight of strand ......................... 481.9 Kg

10.3 FACTORED REACTIONS 10.4 FACTORED COLUMN MOMENTS (kNm)

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### 12 - PUNCHING SHEAR CHECK

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PUNCHING SHEAR STRESS IN ONE OR MORE LOCATIONS EXCEEDS THE PERMISSIBLE VALUE. PROVIDE SHEAR REINFORCEMENT, OR ENLARGE THE SECTION RESISTING THE PUNCHING SHEAR

Comments:
Where stress ratios exceed 1.00, punching shear reinforcement must be provided. If a stress ratio exceeds 1.50, the section has to be enlarged, or re-designed such as to bring the ratio to 1.50 or less. In this case, column 3 has been conservatively modeled as an “edge column.” Its punching shear capacity is larger than assumed in the program. For this reason, it is acceptable if reinforced.