Yield stress f_y = 60 ksi (413.69 MPa)
Modulus of Elasticity = 29000 ksi (199,949 MPa)
Minimum Rebar Cover = 0.75 in Top and Bottom (19 mm)

(ii) Loading

Dead load = self weight + 20 psf (superimposed)
Live load = 40 psf (1.92 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 (γM_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:

$$v_u = \frac{V_u}{A_c} + \frac{\gamma \times M_u \times c}{J_c} \qquad (1\text{-}1)$$

Where,

V_u = absolute value of the direct shear;
M_u = Unbalanced column moment;
A_c = area of concrete of assumed critical section;
γ_v = 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 γ_v for all types of columns are given below.

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

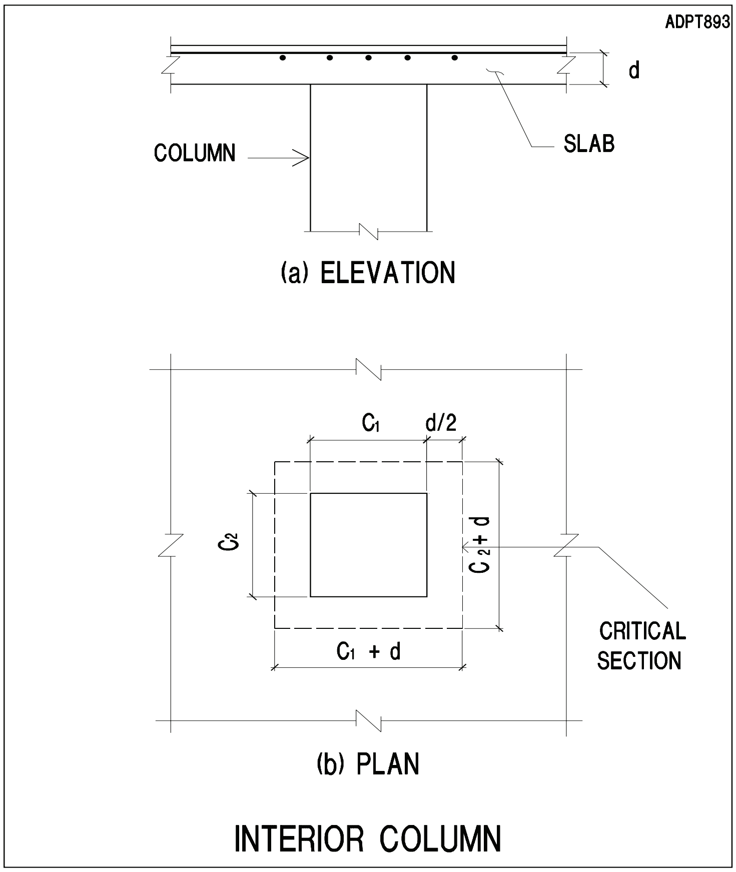

FIGURE 1.1-1

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

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.

(ii) End Column (Refer Fig. 1.1-2)

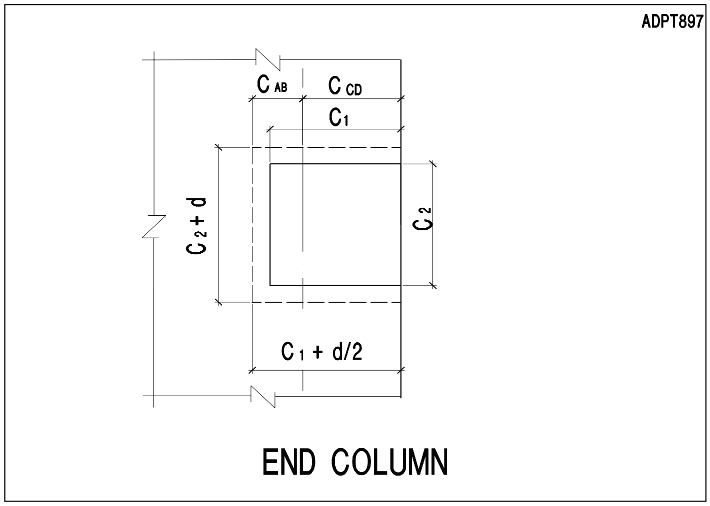

FIGURE 1.1-2 FOR A DESIGN STRIP IN LEFT-RIGHT DIRECTION

$$A_c = d\,(2c1 + c2 + 2d)$$
$$c_{AB} = (c_1 + d/2)^2 \,/\, (2c_1 + c_2 + 2d)$$
$$c_{CD} = (c_1 + d/2) - c_{AB}$$
$$J_c = (c_1 + d/2) * d^3/6 + 2d * (c_{AB}^3 + c_{CD}^3) / 3 + d * (c_2 + d)\, c_{AB}^2$$
$$\gamma_V = 1 - \{1/[1 + (2/3) * ((c_1 + d/2) / (c_2 + d))^{1/2}]\}$$

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)

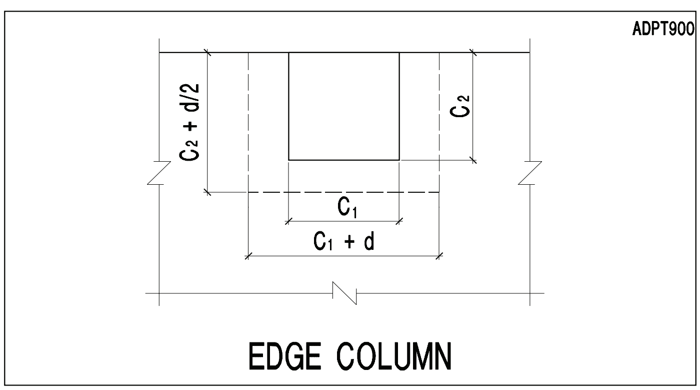

FIGURE 1.1-3 FOR A DESIGN STRIP IN LEFT-RIGHT DIRECTION

$$A_c = d\,(2c_2 + c_1 + 2d)$$
$$J_c = (c_1 + d)^3 * d/12 + (c_1 + d) * d^3/12 + d * (c_2 + d/2) * (c_1 + d)^2/2$$
$$\gamma_V = 1 - \{1/[1 + (2/3) * ((c_1 + d) / (c_2 + d/2))^{1/2}]\}$$

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.

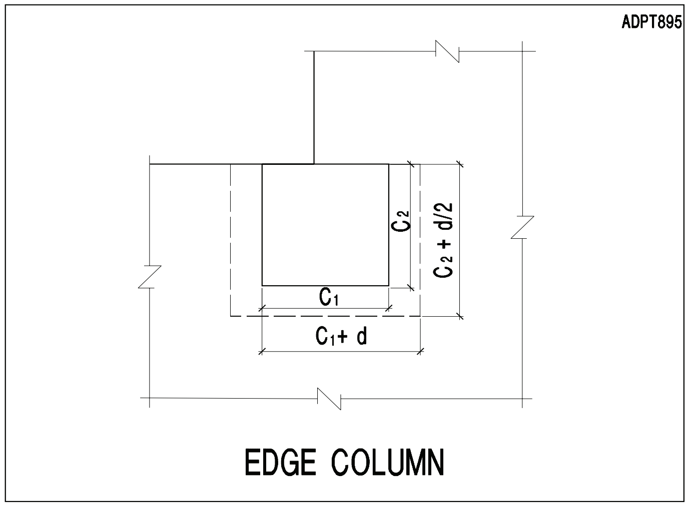

FIGURE 1.1-4

(iv) Corner Column (Refer Fig. 1.1-5)

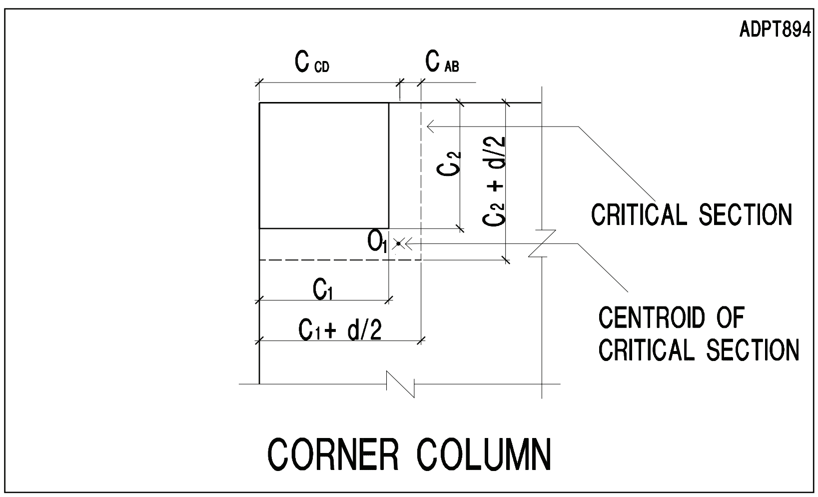

FIGURE 1.1- 5

$$Ac = d\,(c1 + c2 + d)$$
$$c_{AB} = (c_1 + d/2)^2 / 2 * (c_1 + c_2 + d)$$
$$c_{CD} = (c_1 + d/2) - c_{AB}$$
$$J_c = (c_1 + d/2) * d^3/12 + d * (c_{AB}^3 + c_{CD}^3) / 3 + d * (c_2 + d/2)\, c_{AB}^2$$
$$\gamma_V = 1- \{1/[1+ (2/3) * ((c_2 + d/2) / (c_1 + d/2))^{1/2}]\}$$

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.

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:

$$M_{ue} = Mu - Vu^* e$$

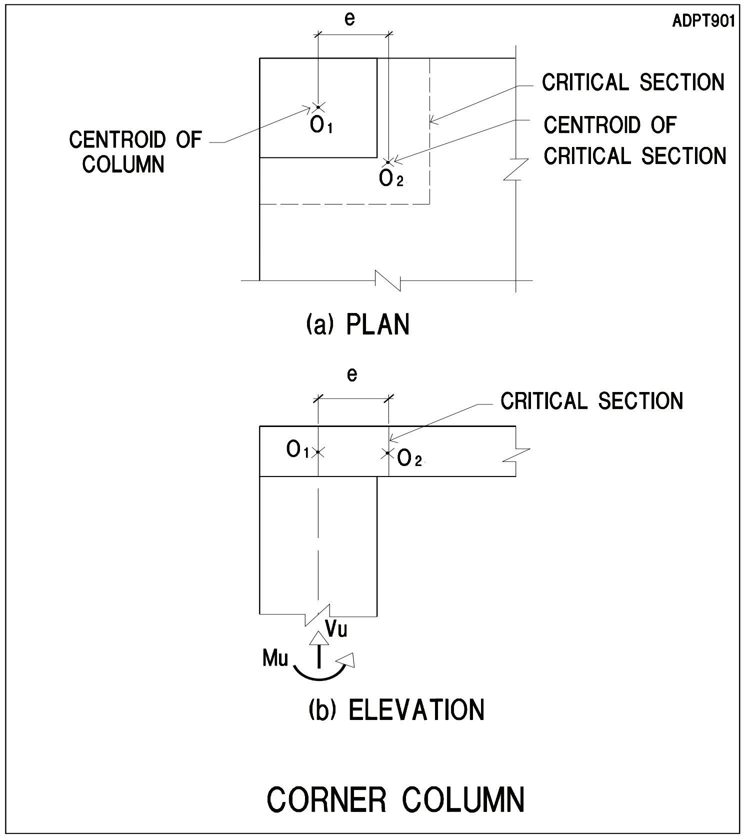

FIGURE 1.1- 6

(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.