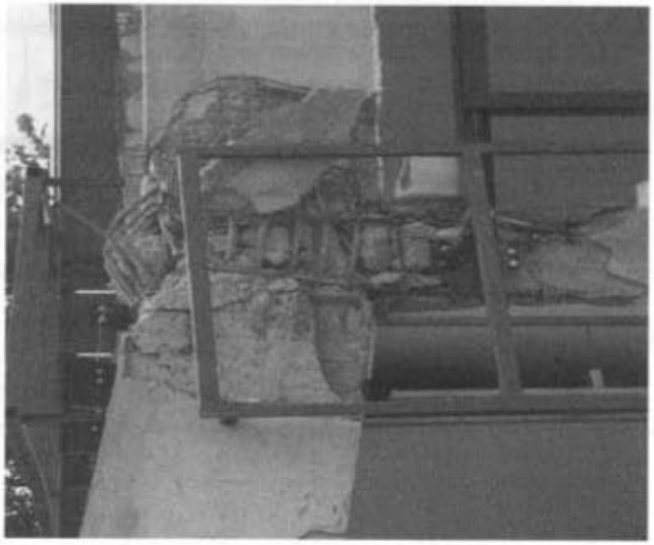
JOINTS

Shear strength requirements (Section 20.5.1.3.2) The nominal shear strength of the joint for normal-weight concrete should not exceed the following:

1. $20\sqrt{f'_c}A_j$ for joints confined on all four faces
2. $15\sqrt{f'_c}A_j$ for joints confined on three faces or on two opposite faces
3. $12\sqrt{f'_c}A_j$ for all other joints

where A_j is the effective area, as shown in Fig. 20.23.



Beam-column connection (joint).

JOINTS

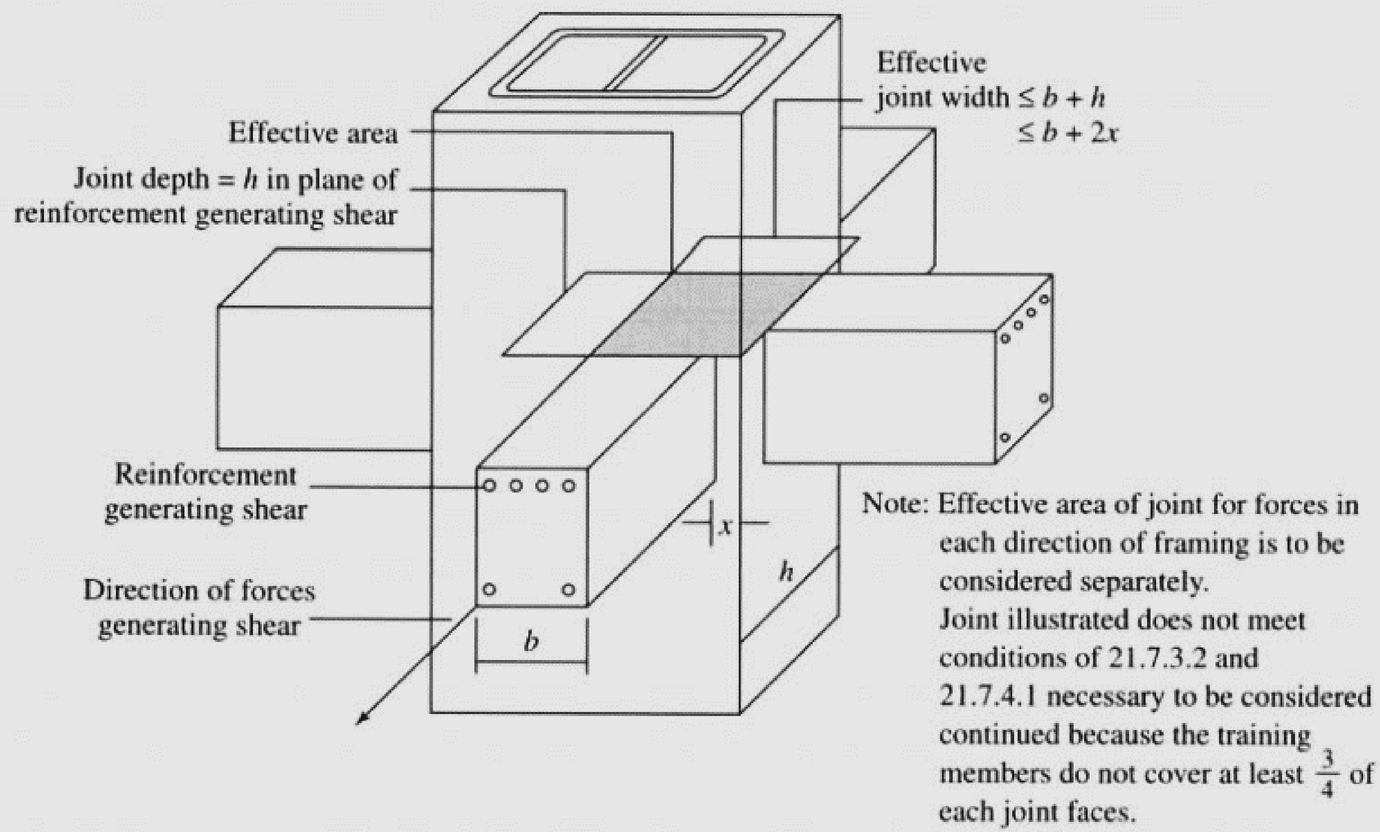
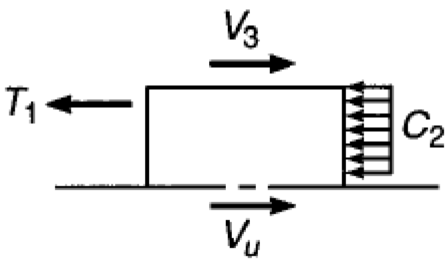
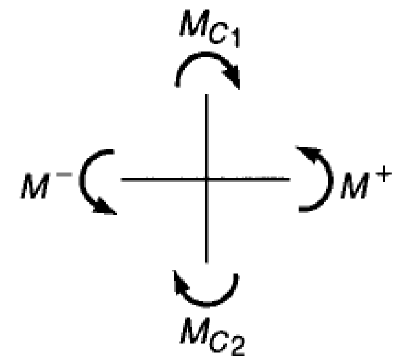


Figure 20.23 Effective *Aj* of joint. Courtesy of American Concrete Institute (ACI 2008).

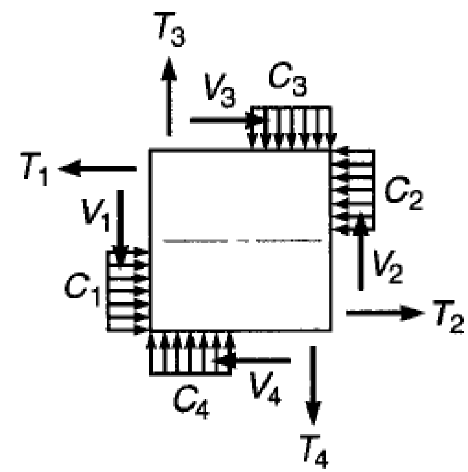
JOINTS



$$V_u = T_1 + C_2 - V_3$$



(b)



JOINTS

c. Joints and Development of Reinforcement

The design of beam-column joints is discussed in Section 11.2. The forces acting on a joint subjected to lateral loads are illustrated in Fig. 11.4. The factored shear acting on a joint is

$$\begin{aligned} V_u &= T_1 + C_2 - V_{col} \\ &= T_1 + T_2 - V_{col} \end{aligned} \tag{20.25}$$

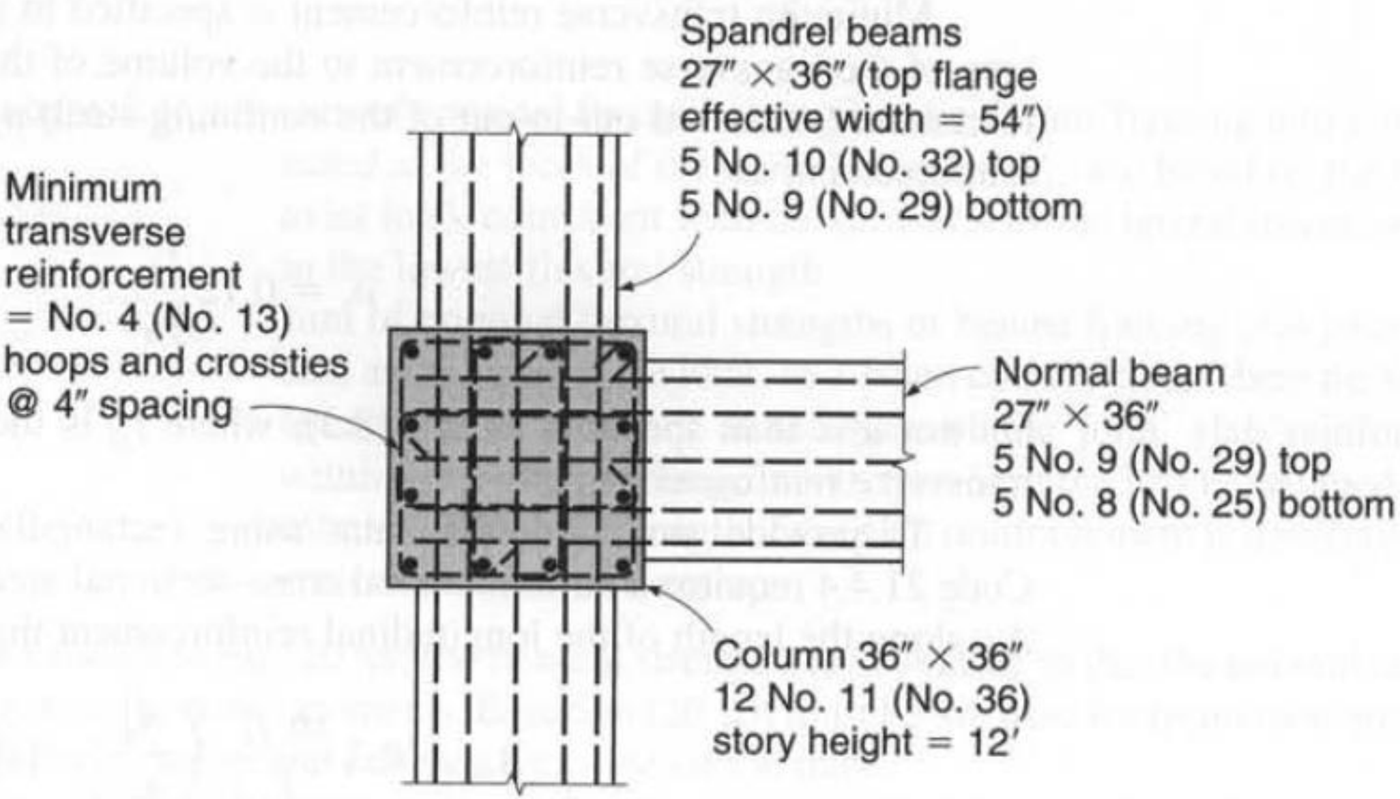
where T_1 = tensile force in negative moment beam steel on one side of a joint
T_2 = tensile force in positive moment beam steel on one side of a joint
C_2 = compressive force counteracting T_2
V_{col} = shear in the column at top and bottom faces of the joint corresponding to the net moment in the joint and points of inflection at midheight of columns (see Fig. 11.5)

JOINTS

Problem:-

Design the external joint for the given figure. Assume the frame story height is I2 ft. Material strengths are fc' = 4000 psi and fy= 60,000 psi.

JOINTS



Spandrel beams
27″ × 36″ (top flange
effective width = 54″)
5 No. 10 (No. 32) top
5 No. 9 (No. 29) bottom
Minimum
transverse
reinforcement
= No. 4 (No. 13)
hoops and crossties
@ 4″ spacing
Normal beam
27″ × 36″
5 No. 9 (No. 29) top
5 No. 8 (No. 25) bottom
Column 36″ × 36″
12 No. 11 (No. 36)
story height = 12′

(a)

JOINTS

Solution. As discussed in Chapter 11, a joint must be detailed so that the beam and column bars do not interfere with each other and so that placement and consolidation of the concrete are practical. Bar placement is shown in Fig. 20.14.

Development of the spandrel beam flexural steel within the joint is checked based on the requirement that the column dimension be at least 20 times the bar diameter of the largest bars. This requirement is met for the No. 10 (No. 32) bars used as top reinforcement.

$$20 \times 1.27 = 25.4 \text{ in.} < 36 \text{ in.}$$

To check the shear strength of the joint, the shear forces acting on the joint must be calculated based on a stress of $1.25f_y$ in the flexural reinforcement. By inspection, shear in the plane of the spandrel beam will control.

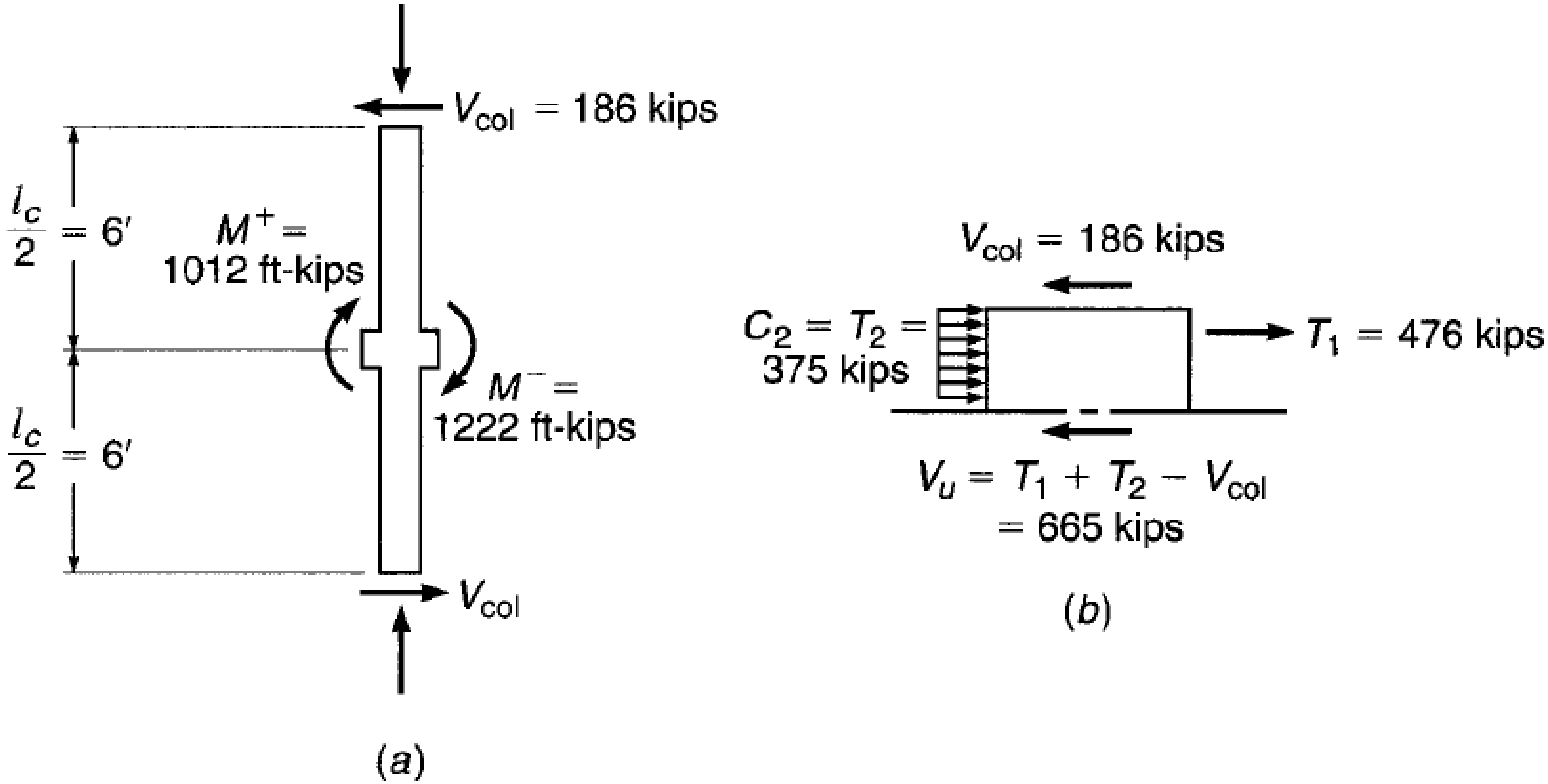
The tensile force in the negative steel is

$$T_1 = 1.25 \times 6.35 \times 60 = 476 \text{ kips}$$

For an effective depth of 33.4 in. (Example 20.1) and a depth of stress block $a = 476/(0.85 \times 4 \times 27) = 5.19$ in., the moment due to negative bending is

$$M^- = \frac{476}{12}\left(33.4 - \frac{5.19}{2}\right) = 1222 \text{ ft-kips}$$

JOINTS



$V_{col} = 186$ kips
$\frac{l_c}{2} = 6'$
$M^+ =$ 1012 ft-kips
$M^- =$ 1222 ft-kips
$\frac{l_c}{2} = 6'$
V_{col}
(a)
$V_{col} = 186$ kips
$C_2 = T_2 =$ 375 kips
$T_1 = 476$ kips
$V_u = T_1 + T_2 - V_{col}$
$= 665$ kips
(b)

JOINTS

For positive bending on the other side of the column,

$$T_2 = 1.25 \times 5.00 \times 60 = 375 \text{ kips}$$

$$a = \frac{375}{0.85 \times 4 \times 54} = 2.04 \text{ in.}$$

$$M^+ = \frac{375}{12}\left(33.4 - \frac{2.04}{2}\right) = 1012 \text{ ft-kips}$$

The column shear corresponding to the sum of the moments M^+ and M^- and based on the free body of the column between assumed midheight inflection points, as shown in Fig. 20.16*a*, is $V_{col} = (1222 + 1012)/12 = 186$ kips. The shear forces acting on the joint are shown in Fig. 20.16*b*, and the factored joint shear is

$$V_u = T_1 + T_2 - V_{col} = 476 + 375 - 186 = 665 \text{ kips}$$

JOINTS

The column shear corresponding to the sum of the moments M^+ and M^- and based on the free body of the column between assumed midheight inflection points, as shown in Fig. 20.16*a*, is $V_{col} = (1222 + 1012)/12 = 186$ kips. The shear forces acting on the joint are shown in Fig. 20.16*b*, and the factored joint shear is

$$V_u = T_1 + T_2 - V_{col} = 476 + 375 - 186 = 665 \text{ kips}$$

For a joint confined on three faces with an effective cross-sectional area $A_j = 36 \times 36 = 1296 \text{ in}^2$, the nominal and design capacities of the joint are

$$V_n = 15\sqrt{f'_c}A_j = \frac{15\sqrt{4000} \times 1296}{1000} = 1229 \text{ kips}$$

$$\phi V_n = 0.85 \times 1229 = 1045 \text{ kips}$$

Since $\phi V_n > V_u$, the joint is satisfactory for shear.

Because the joint is not confined on all four sides, the transverse reinforcement in the column must be continued, unchanged, through the joint.