LOAD COMINATIONS

- A structure should be designed to resist the combined effects of the loadings. Basic load combinations for strength design are given in Section 1605.2.1 IBC code

$$1.4(D+F)$$

$$1.2(D+F+T)+1.6(L+H)+0.5(L_r \text{ or } S \text{ or } R)$$

$$1.2D+1.6(L_r \text{ or } S \text{ or } R)+(f_1 L \text{ or } 0.8W)$$

$$1.2D+1.6W+f_1 L+0.5(L_r \text{ or } S \text{ or } R)$$

$$1.2D+1.0E+f_1 L+f_2 S$$

$$0.9D+1.6W+1.6H$$

$$0.9D+1.0E+1.6H$$

f_1 = 1 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m²), and for parking garage live load, and

= 0.5 for other live loads.

f_2 = 0.7 for roof configurations (such as saw tooth) that do not shed snow off the structure, and

= 0.2 for other roof configurations.

LOAD COMINATIONS

Where

D = Dead Load

L = Live Load excluding roof live load

L_r = Roof live load (Loads produced during maintaince of workers, equipment and materials; or during the life of the structure by movable objects such as planters and by people)

S = Snow Load

R = Rain Load

LOAD COMINATIONS

Where

W = Wind Load

E = Seismic load effect

H= Load due to lateral earth pressures, ground water pressure or pressure of bulk materials.

Special Seismic load combination for strength design should be used in case of structures having certain plan or vertical irregularities in SDC B or higher. Special Load combinations for strength design are given in IBC, Section 1605.4.

LOAD COMINATIONS

1. $1.2\,D + f_1 L + E_m$
2. $0.9D + E_m$

Where

f_1 = 1.0 for floors in places of public assembly for live loads in excess of 100psf and for parking garage live load

f_1 = 0.5 for other live loads

E_m = the maximum effect of horizontal and vertical forces.

SEISMIC LOAD EFFECT (E)

The seismic load effect can be determined using ASCE 7-05 Section 12.4.

It is applicable to all members of the structure, including those not part of the seismic force resisting system.

Seismic load effects are axial, shear and vertical seismic forces.

SEISMIC LOAD EFFECT (E)

The seismic load effect, E, shall be determined using the following equations

1. $E = E_h + E_v$

When the effect of gravity and seismic ground motions are counteractive , the seismic load effect is calculated from

2. $E = E_h - E_v$

Where

Eh = Effect of horizontal seismic forces

Ev = Effect of vertical seismic forces.

HORIZONTAL SEISMIC LOAD EFFECT (E_h)

12.4.2.1 Horizontal Seismic Load Effect. The horizontal seismic load effect, E_h, shall be determined in accordance with Eq. 12.4-3 as follows:

$$E_h = \rho Q_E \qquad (12.4\text{-}3)$$

where

Q_E = effects of horizontal seismic forces from V or F_p. Where required in Sections 12.5.3 and 12.5.4, such effects shall result from application of horizontal forces simultaneously in two directions at right angles to each other.

ρ = redundancy factor, as defined in Section 12.3.4

VERTICAL SEISMIC LOAD EFFECT (E_h)

12.4.2.2 Vertical Seismic Load Effect. The vertical seismic load effect, E_v, shall be determined in accordance with Eq. 12.4-4 as follows:

$$E_v = 0.2 S_{DS} D \quad (12.4\text{-}4)$$

where

S_{DS} = design spectral response acceleration parameter at short periods obtained from Section 11.4.4

D = effect of dead load

REDUNDANCY COEFFICIENT

Redundancy coefficient can be determined using ASCE 7-05 Section 12.3.4.

1. For structures assigned to seismic design category A, B or C , the value of the redundancy coefficient is 1.
2. For structures assigned to SDC , D, E or F, the redundancy coefficient, shall be taken equal to 1.3.

Seismic Force

When the effect of gravity and seismic force are additive, the seismic force effect, Em, should be calculated using the following equation.

$$E_m = \Omega_o Q_E + 0.2 S_{DS} D$$

Where Ω_o= the system over-strength factor given in Table12.2-1

When the effects of gravity and seismic forces counter act the seismic force effect, Em, should be calculated using the following equation.

$$E_m = \Omega_o Q_E - 0.2 S_{DS} D$$

SPECIAL REQUIREMENTS IN DESIGN OF STRUCTURES SUBJECTED TO EQ LOADS

1. The ACI Code 2008, Section 20.1.1.9.1 and 21.1, define five seismic design categories for earthquake resistant structures. These are A, B, C, D, E and F. The classification of these zones described in ACI Section R21.1.1 can be given in three different categories.
   a. SDC D, E, and F indicate high seismic risk zone with strong ground shaking.
   b. SDC C indicates moderate/ intermediate seismic risk zone with moderately strong ground shaking.

SPECIAL REQUIREMENTS IN DESIGN OF STRUCTURES SUBJECTED TO EQ LOADS

c. SDC A and B indicate low seismic risk zones with SDC A corresponding to the lowest seismic hazard zone.

2. For structures in high seismic risk (D,E&F) special requirements in flexural design and detailing are required. Special moment frames, Section 21.5, and special structural walls should be used as the structural system of a building.

SPECIAL REQUIREMENTS IN DESIGN OF STRUCTURES SUBJECTED TO EQ LOADS

3. For the structures in moderate seismic risk some special provision are required for satisfactory intermediate seismic performance. Structure and be designed as intermediate moment frame or intermediate structural wall systems. Structures from a higher category can also be utilized.
4. For the structures in low seismic risk, no special requirements in flexural design and detailing are required. Ordinary moment frames and ordinary structural walls shall be used.

Special Moment frames (Section 21.5, ACI-2008)

1. A special moment frame is a structural system that is designed and detailed to sustain strong earthquakes. Special provision for designing and detailing are given as follows.
   a. Flexural members of SMRF are subjected to only bending.
   b. SMRF members subjected to bending and axial load such as columns.
   c. Joints of special moment frames.

Special Moment frames (Section 21.5, ACI-2008) (Flexural members- General Requirements)

1. If factored axial compressive force Pu < Agfc'/10, then the member is considered to be subjected to bending. Ag represent the gross area of the concrete member. Flexural member should satisfy following the conditions
   a. Clear span Ln ≥ 4 x effective depth "d"
   b. The flexural member width to depth ratio, bw/d ≥ 0.3.
   c. Flexural member width bw ≥ 10 inch

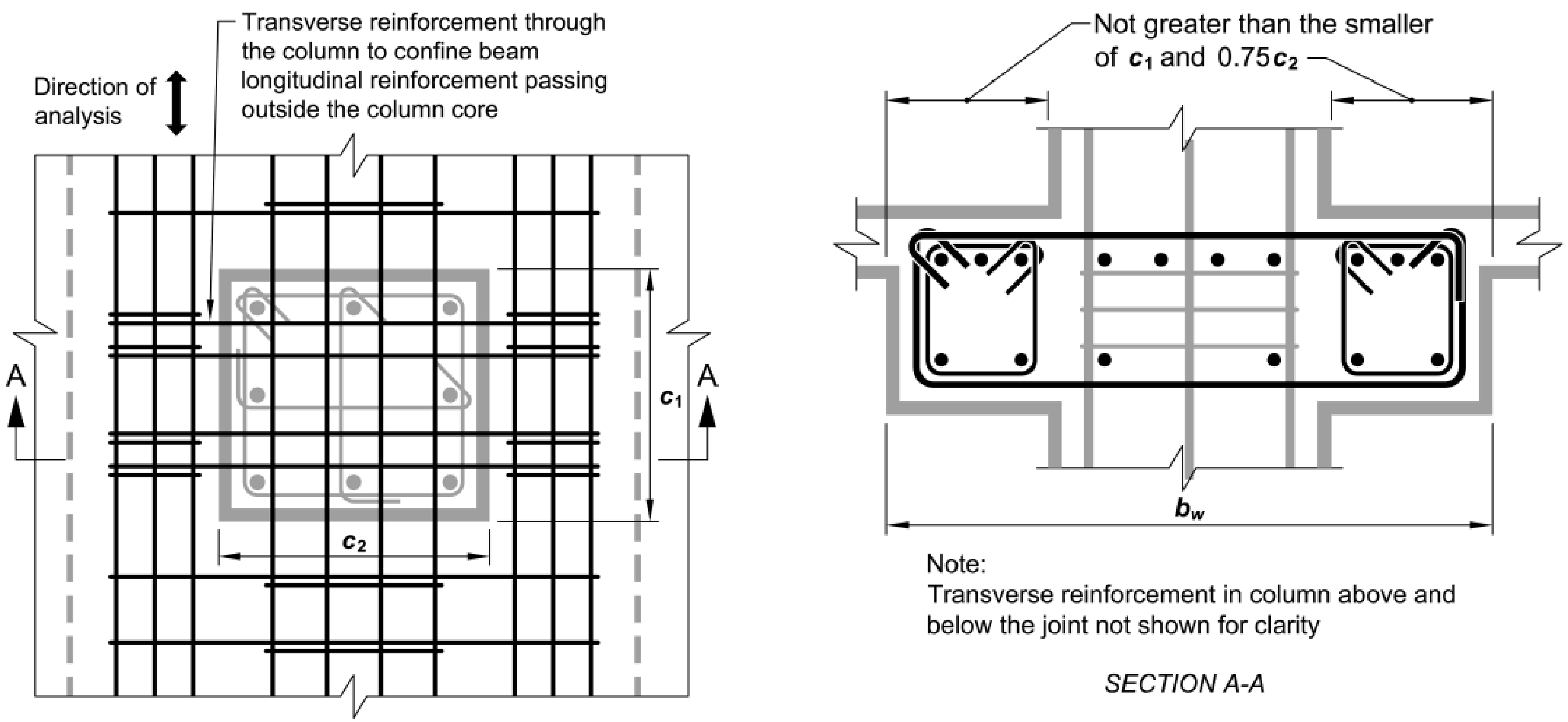
Special Moment frames (Section 21.5, ACI-2008) (Flexural members- General requirements)

d. Flexural member width b_w, shall not exceed width of supporting member, c2, plus a distance on each side of the supporting member equal to the small of (i) and (ii).

(i) Width of the supporting member, c2, and

(ii) 0.75 times the overall dimension of supporting member, c1.

Special Moment frames (Section 21.5, ACI-2008) (Flexural members-General requirements)



Special Moment frames (Section 21.5, ACI-2008) (Flexural members- Longitudnal reinf. requirements)

According to ACI-08 Section,21.5.2, the longitudinal reinf. Shall satisfy the following.

1. Longitudinal reinforcement for both top and bottom steel (A_s) should be in the range defined as follows:

$$\left.\begin{array}{c} \dfrac{3\sqrt{f'_c}bd}{f_y} \\ \dfrac{200bd}{f_y} \end{array}\right\} \leq (A_s) \leq 0.025bd$$

At least two bars should be provided continuously at both top and bottom. For the statically determined T-sections with flanges in tension the value of b in the expression $3\sqrt{f'_c}bd/f_y$ should be replaced by either $2b$ (width of web) or the width of the flange, whichever is smaller (ACI 2008, Section 10.5.2).

Special Moment frames (Section 21.5, ACI-2008) (Flexural members- Longitudnal reinf. requirements)

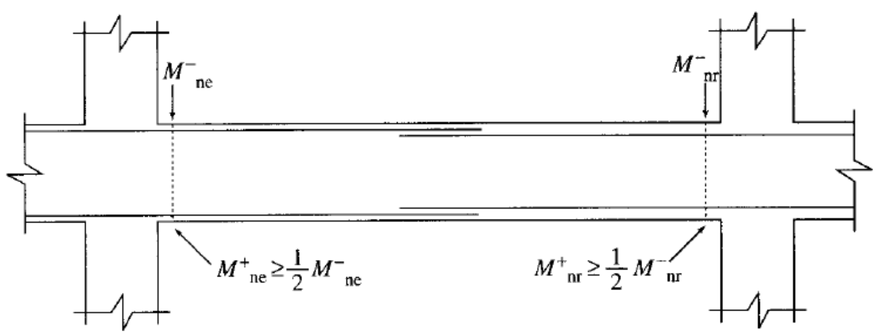
2. The positive moment strength at joint face should be greater or equal $\frac{1}{2}$ negative moment strength at that face of the joint (ACI Section 21.5.2.2):

$$\phi M_{n_l}^{+} \geq \frac{1}{2} \phi M_{n_l}^{-} \qquad \text{(left joint)}$$

$$\phi M_{n_r}^{+} \geq \frac{1}{2} \phi M_{n_r}^{-} \qquad \text{(right joint)}$$

where

M_{n_l} = moment strength at left joint of flexural member
M_{n_r} = moment strength at right joint of flexural member



Longitudinal reinforcement requirements.

Special Moment frames (Section 21.5, ACI-2008) (Flexural members- Longitudnal reinf. requirements)

3. Neither the negative nor positive moment strength at any section along the member should be less than $\frac{1}{4}$ the maximum moment strength provided at the face of either joint.

$$(\phi M_n^+ \text{ or } \phi M_n^-) \geq \frac{1}{4}(\max \phi M_n \text{ at either joint})$$

4. Anchorage of flexural reinforcement in support can be calculated using the following equation:

$$l_{\text{dh}} \geq \begin{cases} \dfrac{f_y d_b}{65\sqrt{f_c'}} \\ 8d_b \\ 6 \text{ in.} \end{cases}$$

where d_b is the diameter of longitudinal reinforcement.

Special Moment frames (Section 21.5, ACI-2008) (Flexural members- Longitudnal reinf. requirements)

5. Lap splices of flexural reinforcement are permitted only if hoop or spiral reinforcement is provided over the lap length. Hoop or spiral reinforcement spacing should not exceed $d/4$ or 4 in., whichever is smaller. Lap splices should not be used within a joint, within a distance of twice the member depth from the face of the joint, or at locations of plastic hinges.

Special Moment frames (Section 21.5, ACI-2008) (Flexural members- Transvers reinf. requirements)

For the special moment resisting frames, plastic hinges will form at the ends of flexural members. Those locations should be specially detailed to ensure sufficient ductility of the frame members. Transverse reinforcement gives lateral support for the longitudinal reinforcement and assists concrete to resist shear . It should satisfy the following.

Special Moment frames (Section 21.5, ACI-2008) (Flexural members- Transvers reinf. requirements)

1. Hoops are required over a length equal to twice the member depth from the face of the support at both ends of flexural member. Also, hoops are required over lengths equal to twice the member depth on both sides of section where flexural yielding may occur, as shown in Fig. 20.12.

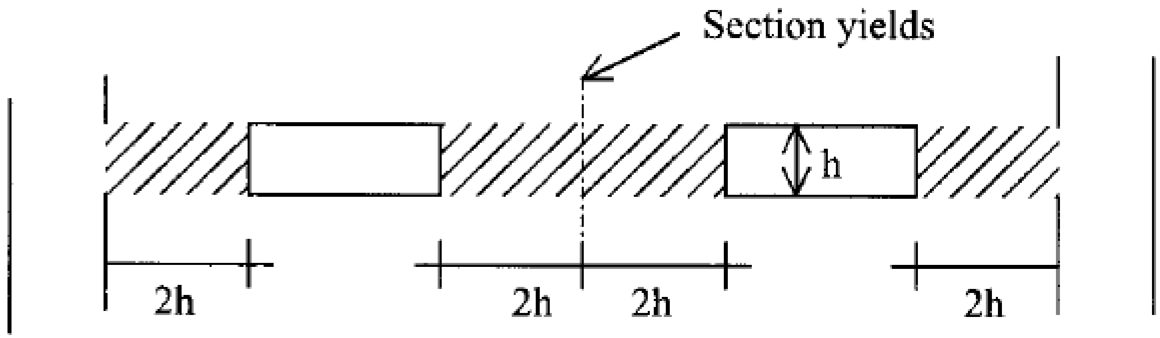


Figure 20.12 Areas of the flexural member where hoops are required. (Note: These areas do not necessarily occur at midspan.)

Special Moment frames (Section 21.5, ACI-2008) (Flexural members- Transvers reinf. requirements)

2. The spacing of the hoops, s, should not exceed the smallest of the following values:

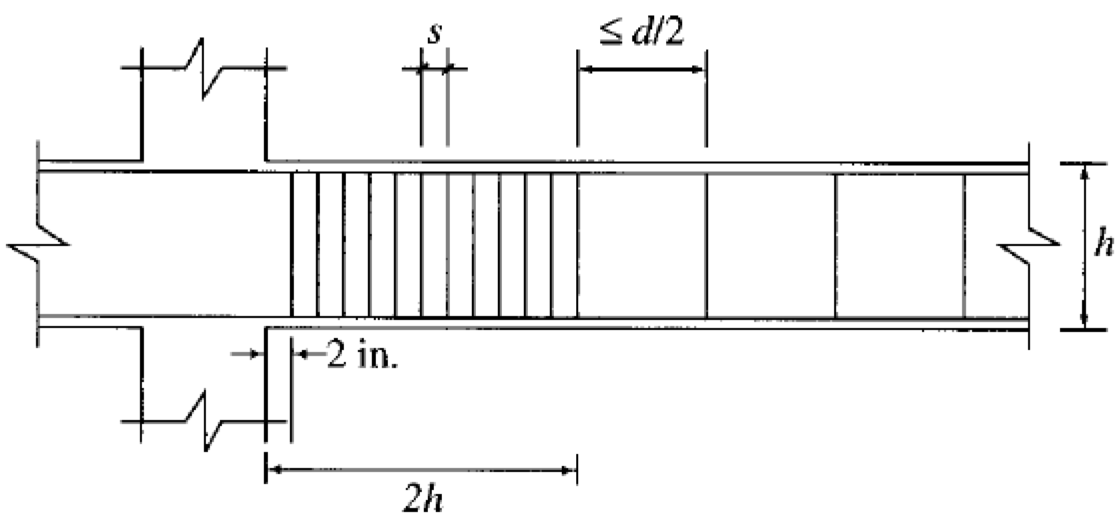
   a. $d/4$

   b. Eight times the diameter of the smallest longitudinal bar

   c. 24 times the diameter of the hoop bars

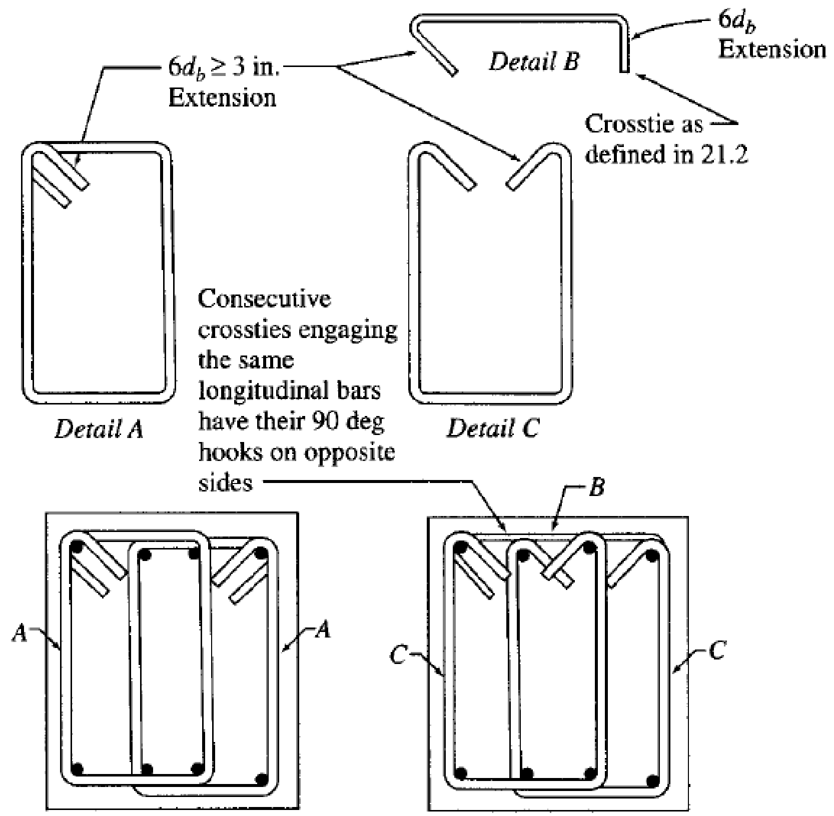
   d. 12 in.

   The first hoop should be located not more than 2 in. from the face of the support.



Special Moment frames (Section 21.5, ACI-2008) (Flexural members- Transvers reinf. requirements)

3. Where hoops are not required, stirrups with seismic hooks at both ends should be used. Spacing between stirrups should be less than or equal to $d/2$.



Special Moment frames (Section 21.5, ACI-2008) (Flexural members- Transvers reinf. requirements)

4. Transverse reinforcement should be designed to resist the design shear force (Figs. 20.13 and 20.14). Design shear force for flexural members of special moment frames can be determined using the following equation (Fig. 20.15):

$$V_l = \frac{M_{pr}^{-} + M_{pr}^{+}}{l_n} + \frac{w_u l_n}{2} \qquad (20.38a)$$

$$V_r = \frac{M_{pr}^{+} + M_{pr}^{-}}{l_n} - \frac{w_u l_n}{2} \qquad (20.38b)$$

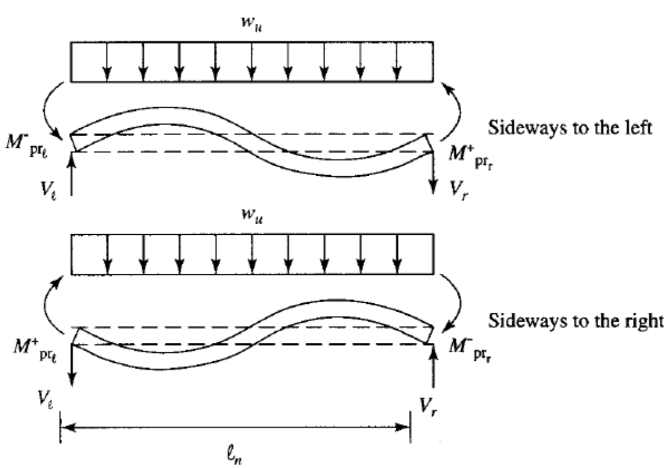


Figure 20.15 Design shear force.

Special Moment frames (Section 21.5, ACI-2008) (Flexural members- Transvers reinf. requirements)

where

V_l = design shear force at left joint of flexural member

V_r = design shear force at right joint of flexural member

M_{pr} = probable moment strength at the end of the beam determined as strength of the beam with the stress in the reinforcing steel equal to 1.25 f_y and a strength reduction factor of $\phi = 1.0$.

l_n = clear span of flexural member

w_u = factored distributed load determined by Eq. 20.47

$$w_u = 1.2D + 1.0L + 0.2S \qquad (20.39)$$

where

D = dead load

L = live load

S = snow load

Special Moment frames (Section 21.5, ACI-2008) (Flexural members- Transvers reinf. requirements)

Probable moment strength at the end of the beam, M_{pr}, can be calculated from the following equation:

$$M_{pr} = A_s(1.25 f_y)\left(d - \frac{a}{2}\right) \quad (20.40)$$

where

$$a = \frac{A_s(1.25 f_y)}{0.85 f'_c b} \quad (20.41)$$

The shear strength of concrete can be taken to be 0 when the earthquake-induced shear force is greater than or equal to 50% of the total shear force and the factored axial compressive force is less than $A_g f'_c/20$, where A_g is the gross area of the beam.

Special Moment frames (Example)

Design a beam AB on the second floor of a building. The building is constructed in the region of high seismic risk on soil class B.

The material properties are

Concrete fc'= 4000psi Fy = 60 ksi LL= 40psf

DL-SI=35psf

Beam size = 20x24inch

Column size = 24x 24 inch

Slab thickness = 7 inch