

# Detailing of Reinforcement in Concrete Structures

## 8.1 Scope

Provisions of Sec. 8.1 and 8.2 of Chapter 8 shall apply for detailing of reinforcement in reinforced concrete members, in general. For reinforced concrete structures, subject to earthquake loadings in seismic design categories B, C and D, special provisions contained in Sec. 8.3 of this chapter shall apply. For notations used in Sec. 8.1 and 8.2 and not explained therein, see 6.1.1.1.

### 8.1.1 Standard Hooks

The term "standard hook" as used in this Code shall mean one of the following:

- (a) 180° bend plus an extension of at least 4 bar diameters, but not less than 65 mm at the free end of the bar.
- (b) 90° bend plus an extension of at least 12 bar diameters at the free end of the bar.
- (c) For stirrup and tie anchorage
  - i. For 16 mm  $\phi$  bar and smaller, a 90° bend plus an extension of at least 6 bar diameters at the free end of the bar,
  - ii. For 19 mm to 25 mm  $\phi$  bars, a 90° bend plus an extension of at least 12 bar diameters at the free end of the bar,
  - iii. For 25 mm  $\phi$  bar and smaller, a 135° bend plus an extension of at least 6 bar diameters at the free end of the bar,
  - iv. For closed ties and continuously wound ties, a 135° bend plus an extension of at least 6 bar diameters, but not less than 75 mm.
- (d) Seismic hooks as defined in 2.2.

### 8.1.2 Minimum Bend Diameters

8.1.2.1 The minimum diameter of bend measured on the inside of the bar, for standard hooks other than for stirrups and ties in sizes 10 mm  $\phi$  through 16 mm  $\phi$ , shall not be less than the values shown in Table 8.1.1.

**Table 8.1.1 Minimum Diameters of Bend**

Bar Size	Minimum Diameter of Bend
10 mm $\leq d_b \leq$ 25 mm	$6d_b$
25 mm $< d_b \leq$ 40 mm	$8d_b$
40 mm $< d_b \leq$ 57 mm	$10d_b$

8.1.2.2 For stirrups and tie hooks, inside diameter of bend shall not be less than 4 bar diameters for 16 mm  $\phi$  bar and smaller. For bars larger than 16 mm  $\phi$ , diameter of bend shall be in accordance with Table 8.1.1.

8.1.2.3 Inside diameter of bend in welded wire reinforcement for stirrups and ties shall not be less than 4 bar diameters for deformed wire larger than MD40 and 2 bar diameters for all other wires. Bends with inside diameter of less than 8 bar diameters shall not be less than 4 bar diameters from nearest welded intersection.

### 8.1.3 Bending

- 8.1.3.1 Unless otherwise permitted by the engineer, all reinforcement shall be bent cold.
- 8.1.3.2 Reinforcement partially embedded in concrete shall not be bent in place, except as permitted by the engineer or as shown in the design drawings.

### 8.1.4 Surface Conditions of Reinforcement

- 8.1.4.1 When concrete is placed, metal reinforcement shall be free from mud, oil, or other nonmetallic coatings that decrease bond. Epoxy-coating of steel reinforcement in accordance with standards referenced in this code shall be permitted.
- 8.1.4.2 Metal reinforcement with rust, mill scale, or a combination of both, shall be considered satisfactory, provided the minimum dimensions (including height of deformations) and weight of a hand-wire-brushed test specimen are not less than applicable ASTM specification requirements.

### 8.1.5 Placing of Reinforcement

- 8.1.5.1 Reinforcement shall be accurately placed and adequately supported before concrete is placed, and shall be secured against displacement within tolerances permitted in 8.1.5.2 below.
- 8.1.5.2 Reinforcement shall be placed within the following tolerances unless otherwise specified by the engineer:
- (a) Tolerances for depth  $d$ , and minimum concrete cover in flexural members, walls and compression members shall be as set forth in Table 8.1.2.

**Table 8.1.2 Tolerances for Placing Reinforcement**

	<b>Tolerance for <math>d</math></b>	<b>Tolerance for Minimum Concrete Cover</b>
$d \leq 200$ mm	$\pm 10$ mm	-10 mm
$d > 200$ mm	$\pm 13$ mm	-13 mm

- (b) Notwithstanding the provision of (a) above, tolerance for the clear distance to formed soffits shall be minus 6 mm and tolerance for cover shall not exceed minus 1/3 the minimum concrete cover specified in the design drawings or specifications.
- (c) Tolerance for longitudinal location of bends and ends of reinforcement shall be  $\pm 50$  mm, except at discontinuous ends of brackets and corbels, where tolerance shall be  $\pm 13$  mm and at discontinuous ends of other members, where tolerance shall be  $\pm 25$  mm. The tolerance for concrete cover of 8.1.5.2a shall also apply at discontinuous ends of members.
- 8.1.5.3 Welded wire reinforcement (with wire size not greater than MW30 or MD30) used in slabs not exceeding 3 m in span shall be permitted to be curved from a point near the top of slab over the support to a point near the bottom of slab at midspan, provided such reinforcement is either continuous over, or securely anchored at support.
- 8.1.5.4 Welding of crossing bars shall not be permitted for assembly of reinforcement unless authorized by the engineer.

### 8.1.6 Spacing of Reinforcement

- 8.1.6.1 The minimum clear spacing between parallel bars in a layer shall be equal to one bar diameter, but not less than 25 mm, or 4/3 the maximum nominal size of coarse aggregate, whichever is larger.

- 8.1.6.2 Where parallel reinforcement is placed in two or more layers, bars in the upper layers shall be placed directly above those in the bottom layer with clear distance between layers not less than 25 mm.
- 8.1.6.3 For compression members, the clear distance between longitudinal bars shall be not less than 1.5 bar diameters nor 40 mm nor  $4/3$  the maximum nominal size of coarse aggregate.
- 8.1.6.4 Clear distance limitation between bars shall apply also to the clear distance between a contact lap splice and adjacent splices or bars.
- 8.1.6.5 In walls and one-way slabs the maximum bar spacing shall not be more than three times the wall or slab thickness  $h$  nor 450 mm.
- 8.1.6.6 For two-way slabs, maximum spacing of bars shall not exceed two times the slab thickness  $h$  nor 450 mm.
- 8.1.6.7 For temperature steel only, maximum spacing shall not exceed five times the slab thickness  $h$  nor 450 mm.
- 8.1.6.8 **Bundled Bars**
- Groups of parallel reinforcing bars bundled in contact to act as a unit shall be limited to four in any one bundle.
  - Bundled bars shall be enclosed within stirrups or ties.
  - Bars larger than 32 mm  $\phi$  shall not be bundled in beams.
  - Individual bars within a bundle terminated within the span of flexural members shall terminate at different points with at least  $40d_b$  stagger.
  - Where spacing limitations and minimum concrete cover are based on bar diameter  $d_b$ , a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area.

### 8.1.7 Exposure Condition and Cover to Reinforcement

- 8.1.7.1 The nominal concrete cover to all reinforcement (including links), maximum free water-cement ratio and minimum cement content required for various minimum concrete strengths used in different exposure conditions shall be as specified in Table 8.1.3. However, for mild environment, the minimum concrete cover specified in 8.1.7.2 and 8.1.7.3 for various structural elements may be used.

#### 8.1.7.2 Cast-in-place Concrete :

- Minimum concrete cover for concrete cast against and permanently exposed to earth shall be 75 mm.
- Concrete exposed to earth or weather:

	Minimum cover, mm
19 mm $\phi$ through 57 mm $\phi$	50
16 mm $\phi$ bar and smaller	40

- The following minimum concrete cover may be provided for reinforcement for concrete surfaces not exposed to weather or in contact with ground:

	Minimum cover, mm
Slabs, Walls :	
40 mm $\phi$ to 57 mm $\phi$	40
36 mm $\phi$ bar and smaller	20

		Minimum cover, mm
Beams, Columns :		
Primary reinforcement, Ties, stirrups, spirals		40
Shells, folded plate members :		
19 mm $\phi$ bar and larger		20
16mm $\phi$ bar and smaller		16

**Table 8.1.3\* Concrete Cover and other Requirements for Various Exposure Conditions**

	Environment	Exposure Conditions	Minimum $f'_c$ N/mm <sup>2</sup>						
			20	25	30	35	40	45	50
Nominal cover (mm)	Mild	Concrete surfaces protected against weather or aggressive conditions	30	25	20	20	20**	20**	20**
	Moderate	Concrete surface away from severe rain Concrete subject to condensation Concrete surfaces continuously under water Concrete in contact with non-aggressive soil	40	35	30	25	20	20	20
	Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or severe condensation		45	40	30	25	25	20
	Very severe	Concrete surfaces exposed to sea water spray, corrosive fumes			50	40	30	30	25
	Extreme	Concrete surfaces exposed to abrasive action, e.g. sea water carrying solids or flowing water with pH $\leq$ 4.5 or machinery or vehicles				60	50	40	30
	Maximum water/cement ratio			0.65	0.65	0.60	0.55	0.50	0.45
Minimum cement content, (kg/m <sup>3</sup> )			315	325	350	375	400	410	420

\*This table relates to aggregate of 20 mm nominal maximum size.

\*\* May be reduced to 15 mm provided the nominal maximum aggregate size does not exceed 15mm.

**8.1.7.3 Precast Concrete (manufactured under plant control conditions) :**

(a) Concrete exposed to earth or weather:

		Minimum cover, mm
Wall Panels:		
40 mm $\phi$ to 57 mm $\phi$		40
36 mm $\phi$ bar and smaller		20
Other Members:		
40 mm $\phi$ to 57 mm $\phi$		50
19 mm $\phi$ through 36 mm $\phi$		40
16mm $\phi$ bar and smaller		30

(b) Concrete not exposed to weather or in contact with ground:

	Minimum cover, mm
Slabs, Walls:	
40 mm $\phi$ to 57 mm $\phi$	30
36 mm $\phi$ bar and smaller	16
Beams, columns :	
Primary reinforcement	$20 \leq d_b \leq 40$
Ties, stirrups, spiral	15
Shells, folded plate members :	
19 mm $\phi$ bar and larger	16
16 mm $\phi$ bar and smaller	10

8.1.7.4 For concrete cast against and permanently exposed to earth, minimum cover shall be 75 mm. If, concrete cover specified in 8.1.7.1 (Table 8.1.3) conflicts with those specified in 8.1.7.2 or 8.1.7.3, the larger value shall be taken.

8.1.7.5 **Bundled Bars** : Minimum concrete cover shall be equal to the equivalent diameter of the bundle, but need not be greater than 50 mm.

8.1.7.6 **Future Extension** : Exposed reinforcement, inserts, and plates intended for bonding with future extensions shall be protected from corrosion.

8.1.7.7 **Fire Protection** : If a thickness of cover for fire protection greater than the concrete covers specified in 8.1.7.1 through 8.1.7.6 is required, such greater thicknesses shall be specified.

8.1.7.8 **Corrosive Environments**: If a thickness of cover for corrosive environment or other severe exposure conditions greater than the concrete covers specified in 8.1.7.1 through 8.1.7.6 is required, such greater thicknesses shall be specified. For corrosion protection, a specified concrete cover for reinforcement not less than 50 mm for walls and slabs and not less than 65 mm for other members may be used. For precast concrete members a specified concrete cover not less than 40 mm for walls and slabs and not less than 50 mm for other members may be used.

## 8.1.8 Reinforcement Details for Columns

8.1.8.1 **Offset Bars** : Offset bent longitudinal bars shall conform to the following:

- (a) The maximum slope of inclined portion of an offset bar with axis of column shall not exceed 1 in 6.
- (b) Portions of bar above and below an offset shall be parallel to the axis of column.
- (c) Horizontal support at offset bends shall be provided by lateral ties, spirals, or parts of the floor construction. Horizontal support provided shall be designed to resist 1.5 times the horizontal component of the computed force in the inclined portion of the offset bars. Lateral ties or spirals, if used, shall be placed not more than 150 mm away from points of bend.
- (d) Offset bars shall be bent before placement in the forms (see 8.1.3).
- (e) Where the face of the column above is offset 75 mm or more from the face of the column below, longitudinal bars shall not be permitted to be offset bent. The longitudinal bars adjacent to the offset column faces shall be lap spliced using separate dowels. Lap splices shall conform to 8.2.14.

8.1.8.2 **Steel Cores**: Load transfer in structural steel cores of composite compression members shall be provided by the following:

- (a) Ends of structural steel cores shall be accurately finished to bear at end bearing splices, with positive provision for alignment of one core above the other in concentric contact.

- (b) At end bearing splices, bearing shall be considered effective to transfer not more than 50 per cent of the total compressive stress in the steel core.
- (c) Transfer of stress between column base and footing shall be designed in accordance with 6.8.8.
- (d) Base of structural steel section shall be designed to transfer the total load from the entire composite member to the footing; or, the base shall be designed to transfer the load from the steel core only, provided ample concrete section is available for transfer of the portion of the total load carried by the reinforced concrete section to the footing by compression in the concrete and by reinforcement.

### 8.1.9 Lateral Reinforcement for Columns

8.1.9.1 Lateral reinforcement for compression members shall conform to the provisions of 8.1.9.3 and 8.1.9.4 below and where shear or torsion reinforcement is required, shall also conform to provisions of Sec 6.4.

8.1.9.2 Lateral reinforcement requirements for composite columns shall conform to 6.3.13.7 and 6.3.13.8.

8.1.9.3 **Spirals** : Spiral reinforcement for columns shall conform to 6.3.9.3 and to the following:

- (a) Spirals shall consist of evenly spaced continuous bar or wire of such size and so assembled as to permit handling and placing without distortion from designed dimensions.
- (b) Size of spirals shall not be less than 10 mm diameter for cast-in-place construction.
- (c) The minimum and maximum clear spacing between spirals shall be 25 mm and 75 mm respectively.
- (d) Anchorage of spiral reinforcement shall be provided by 1.5 extra turns of spiral bar or wire at each end of a spiral unit.
- (e) Splices in spiral reinforcement shall be lap splices of 48 spiral diameter for deformed uncoated bar or wire and 72 spiral diameter for other cases, but not less than 300 mm.
- (f) Spirals shall extend from the top of footing or slab in any storey to the level of the lowest horizontal reinforcement in members supported above.
- (g) Spirals shall extend above termination of spiral to bottom of slab or drop panel, where beams or brackets do not frame into all sides of a column.
- (h) Spirals shall extend to a level at which the diameter or width of capital is 2 times that of the column, in case of columns with capitals.
- (i) Spirals shall be held firmly in place and true to line.

8.1.9.4 **Ties** : Tie reinforcement for compression members shall conform to the following:

- (a) All bars shall be enclosed by lateral ties, at least 10 mm  $\phi$  in size for longitudinal bars 32 mm  $\phi$  or smaller, and at least 12 mm  $\phi$  in size for 36 mm  $\phi$  to 57 mm  $\phi$  and bundled longitudinal bars.
- (b) Vertical spacing of ties shall not exceed 16 longitudinal bar diameters or 48 tie diameters, or the least dimension of the compression members.
- (c) Ties shall be arranged such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a tie with an included angle not more than 135 deg. No vertical bar shall be farther than 150 mm clear on each side along the tie from such a laterally supported bar. Where longitudinal bars are located around the perimeter of a circle, a complete circular tie is allowed.
- (d) The lowest tie in any storey shall be placed within one-half the required tie spacing from the top most horizontal reinforcement in the slab or footing below. The uppermost tie in any storey shall be within one-half the required tie spacing from the lowest horizontal reinforcement in the slab or drop panel above.
- (e) Where beams or brackets provide concrete confinement at the top of the column on all (four) sides, the top tie shall be within 75 mm of the lowest horizontal reinforcement in the shallowest of such beams or brackets.
- (f) Where anchor bolts are placed in the top of columns or pedestals, the bolts shall be enclosed by lateral reinforcement that also surrounds at least four vertical bars of the column or pedestal. The lateral reinforcement shall be distributed within 125 mm of the top of the column or pedestal, and shall consist of at least two 12 mm  $\phi$  bars or three 10 mm  $\phi$  bars.
- (g) Where longitudinal bars are arranged in a circular pattern, individual circular ties per specified spacing may be used.

### 8.1.10 Lateral Reinforcement for Beams

- 8.1.10.1 Compression reinforcement in beams shall be enclosed by ties or stirrups satisfying the size and spacing limitations in 8.1.9.4 above. Such ties or stirrups shall be provided throughout the distance where compression reinforcement is required.
- 8.1.10.2 Lateral reinforcement for flexural framing members subject to stress reversals or to torsion at supports shall consist of closed ties, closed stirrups, or spirals extending around the flexural reinforcement.
- 8.1.10.3 Closed ties or stirrups shall be formed in one piece by overlapping standard stirrup or tie end hooks around a longitudinal bar, or formed in one or two pieces lap spliced with a Class B splice (lap of  $1.3\ell_d$ ) or anchored in accordance with 8.2.10.

### 8.1.11 Shrinkage and Temperature Reinforcement

- 8.1.11.1 Where the flexural reinforcement extends in one direction only, reinforcement for shrinkage and temperature stresses shall be provided perpendicular to flexural reinforcement in structural slabs. Shrinkage and temperature reinforcement shall be provided in accordance with 8.1.11.2 below.
- 8.1.11.2 Deformed reinforcement conforming to 5.3.2 shall be provided in accordance with the following:
- Area of shrinkage and temperature reinforcement shall provide at least the following ratios of reinforcement area to gross concrete area:
 

0.0020	for slabs where reinforcement with $f_y = 275 \text{ N/mm}^2$ or $350 \text{ N/mm}^2$ are used,
0.0018	for slabs where reinforcement with $f_y = 420 \text{ N/mm}^2$ are used,
$0.0018 \left( \frac{420}{f_y} \right)$	or slabs where reinforcement with $f_y$ exceeding $420 \text{ N/mm}^2$ are used.

In any case, the reinforcement ratio shall not be less than 0.0014.
  - Area of shrinkage and temperature reinforcement for brick aggregate concrete shall be at least 1.5 times that provided in (a) above.
  - Shrinkage and temperature reinforcement shall be spaced not farther apart than 5 times the slab thickness, nor 450 mm.
  - At all sections where required, reinforcement for shrinkage and temperature stresses shall develop the specified yield strength  $f_y$  in tension in accordance with Sec 8.2.

### 8.1.12 Requirements for Structural Integrity

- 8.1.12.1 In the detailing of reinforcement and connections, members of a structure shall be effectively tied together to improve integrity of the overall structure.
- 8.1.12.2 The minimum requirements for cast-in-place construction shall be:
- In one-way slab construction, at least one bottom bar shall be continuous or shall be spliced over the support with a Class A tension splice. At non-continuous supports, the bars may be terminated with a standard hook.
  - Beams at the perimeter of the structure shall have at least one-sixth of the tension reinforcement required for negative moment at the support, but not less than two bars and one-quarter of the positive moment reinforcement required at midspan, but not less than two bars made continuous over the span length passing through the region bounded by the longitudinal reinforcement of the column around the perimeter and tied with closed stirrups. Closed stirrups need not be extended through any joints. The required continuity may be provided with top reinforcement spliced at mid-span and bottom reinforcement spliced at or near the support with Class B tension splices.
  - When closed stirrups are not provided, in other than perimeter beams, at least one-quarter of the positive moment reinforcement required at mid-span, but not less than two bars shall pass through the region bounded by the longitudinal reinforcement of the column and shall be continuous or shall

be spliced over the support with a Class B tension splice. At non-continuous supports the bars shall be anchored to develop  $f_y$  at the face of the support using a standard hook.

8.1.12.3 To effectively tie elements together, tension ties shall be provided in the transverse, longitudinal, and vertical directions and around the perimeter of the structure for precast concrete construction.

### 8.1.13 Connections

8.1.13.1 Enclosure shall be provided for splices of continuing reinforcement and for anchorage of terminating reinforcement at connections of principal framing elements (such as beams and columns),

8.1.13.2 External concrete or internal closed ties, spirals, or stirrups shall be used as enclosures at connections.

## 8.2 DEVELOPMENT AND SPLICES OF REINFORCEMENT

### 8.2.1 Development of Reinforcement - General

Calculated tension or compression stress in reinforcement at each section of reinforced concrete members shall be developed on each side of that section by embedment length, hook or mechanical device, or a combination thereof. Hooks may be used in developing bars in tension only.

### 8.2.2 Scope and Limitation

The values of  $\sqrt{f'_c}$  used in Sec 8.2 shall not exceed 8.3 MPa. In addition to requirements stated here that affect detailing of reinforcement, structural integrity requirements of 8.1.12 shall be satisfied.

### 8.2.3 Development of Deformed Bars and Deformed Wires in Tension

8.2.3.1 Development length for deformed bars and deformed wire in tension,  $l_d$  shall be determined from either 8.2.3.2 or 8.2.3.3 and applicable modification factors of 8.2.3.4 and 8.2.3.5, but  $l_d$  shall not be less than 300 mm.

8.2.3.2 For deformed bars or deformed wire,  $l_d$  shall be as follows:

Spacing and cover	19 mm $\phi$ and smaller bars and deformed wires	20 mm $\phi$ and larger bars and deformed wires
Clear spacing of bars or wires being developed or spliced not less than $d_b$ , clear cover not less than $d_b$ , and stirrups or ties throughout $l_d$ not less than the Code minimum	$\left(\frac{f_y \psi_t \psi_e}{2.1 \lambda \sqrt{f'_c}}\right) d_b$	$\left(\frac{f_y \psi_t \psi_e}{1.7 \lambda \sqrt{f'_c}}\right) d_b$
or Clear spacing of bars or wires being developed or spliced not less than $2d_b$ and clear cover not less than $d_b$		
Other cases	$\left(\frac{f_y \psi_t \psi_e}{1.4 \lambda \sqrt{f'_c}}\right) d_b$	$\left(\frac{f_y \psi_t \psi_e}{1.1 \lambda \sqrt{f'_c}}\right) d_b$

8.2.3.3 For deformed bars or deformed wire,  $l_d$  shall be

$$l_d = \left( \frac{f_y}{1.1 \lambda \sqrt{f'_c}} \psi_t \psi_e \psi_s \left( \frac{c_b + K_{tr}}{d_b} \right) \right) d_b \quad 8.2.1$$

in which the confinement term  $(c_b + K_{tr})/d_b$  shall not be taken greater than 2.5, and

$$K_{tr} = \frac{40A_{tr}}{sn} \quad 8.2.2$$



where  $n$  is the number of bars or wires being spliced or developed along the plane of splitting. It shall be permitted to use  $K_{tr} = 0$  as a design simplification even if transverse reinforcement is present.

8.2.3.4 The factors used in the expressions for development of deformed bars and deformed wires in tension in 8.2.3 are as follows:

- Where horizontal reinforcement is placed such that more than 300 mm of fresh concrete is cast below the development length or splice,  $\psi_t = 1.3$ . For other situations,  $\psi_t = 1.0$ .
- For epoxy-coated bars or wires with cover less than  $3d_b$ , or clear spacing less than  $6d_b$ ,  $\psi_e = 1.5$ . For all other epoxy-coated bars or wires,  $\psi_e = 1.2$ . For uncoated and zinc-coated (galvanized) reinforcement,  $\psi_e = 1.0$ . However, the product  $\psi_t \psi_e$  need not be greater than 1.7.
- For 19 mm  $\phi$  and smaller bars and deformed wires,  $\psi_s = 0.8$ . For 20 mm  $\phi$  and larger bars,  $\psi_s = 1.0$ .
- Where lightweight concrete is used,  $\lambda$  shall not exceed 0.75 unless  $f_{ct}$  is specified (see 6.1.8.1). Where normalweight concrete is used,  $\lambda = 1.0$ .

8.2.3.5 **Excess Reinforcement** : Development length may be reduced by the factor  $\left[ \frac{A_{s,required}}{A_{s,Provided}} \right]$  where reinforcement in a flexural member is in excess of that required by analysis except where anchorage or development for  $f_y$  is specifically required or the reinforcement is designed under the provisions of 8.3.3.1(c).

## 8.2.4 Development of Deformed Bars and Deformed Wires in Compression

8.2.4.1 Development length for deformed bars and deformed wire in compression,  $l_{dc}$ , shall be determined from 8.2.4.2 and applicable modification factors of 8.2.4.3, but  $l_{dc}$  shall not be less than 200 mm.

8.2.4.2 For deformed bars and deformed wire,  $l_{dc}$  shall be taken as the larger of  $(0.24f_y/\lambda\sqrt{f'_c})d_b$  and  $(0.043f_y)d_b$ , with  $\lambda$  as given in 8.2.3.4(d) and the constant 0.043 carries the unit of  $\text{mm}^2/\text{N}$ .

8.2.4.3 Length  $l_{dc}$  in 8.2.4.2 shall be permitted to be multiplied by the applicable factors for:

- Reinforcement in excess of that required by analysis .....  $\left[ \frac{A_{s,required}}{A_{s,Provided}} \right]$
- Reinforcement enclosed within spiral reinforcement not less than 6 mm diameter and not more than 100 mm pitch or within 12 mm  $\phi$  ties in conformance with 8.1.9.4 and spaced at not more than 100 mm on center .....0.75

## 8.2.5 Development of Bundled Bars

8.2.5.1 Development length of individual bars within a bundle, in tension or compression, shall be that for the individual bar, increased 20 per cent for 3 bar bundles and 33 per cent for 4 bar bundles.

8.2.5.2 For determining the appropriate spacing and cover values in 8.2.3.2, the confinement term in 8.2.3.3, and the  $\psi_e$  factor in 8.2.3.4(b), a unit of bundled bars shall be treated as a single bar of a diameter derived from the equivalent total area and having a centroid that coincides with that of the bundled bars.

## 8.2.6 Development of Standard Hooks in Tension

8.2.6.1 Development length  $l_{dh}$  for deformed bars in tension terminating in a standard hook shall be computed as the product of the basic development length for deformed bars,  $l_{dh}$  of 8.2.6.2 below and the applicable modification factor or factors of 8.2.6.3, but  $l_{dh}$  shall be not less than  $8d_b$ , nor less than 150 mm.

8.2.6.2 For deformed bars,  $l_{dh}$  shall be  $(0.24\psi_e f_y / \lambda \sqrt{f'_c}) d_b$  with  $\psi_e$  taken as 1.2 for epoxy-coated reinforcement, and  $\lambda$  taken as 0.75 for lightweight concrete. For other cases,  $\psi_e$  and  $\lambda$  shall be taken as 1.0.

8.2.6.3 Length  $l_{dh}$  in 8.2.6.2 shall be permitted to be multiplied by the following applicable factors:

(a) For 36 mm  $\phi$  bar and smaller hooks with side cover (normal to plane of hook) not less than 65 mm, and for 90-degree hook with cover on bar extension beyond hook not less than 50 mm 0.7

(b) For 90-degree hooks of 36 mm  $\phi$  bar and smaller bars that are either enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than  $3d_b$  along  $l_{dh}$ ; or enclosed within ties or stirrups parallel to the bar being developed, spaced not greater than  $3d_b$  along the length of the tail extension of the hook plus bend 0.8

(c) For 180-degree hooks of 36 mm  $\phi$  bar and smaller bars that are enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than  $3d_b$  along  $l_{dh}$  0.8

(d) Where anchorage or development for  $f_y$  is not specifically required, reinforcement in excess of that required by analysis  $\left[ \frac{A_{s,required}}{A_{s,provided}} \right]$

In 8.2.6.3(b) and 8.2.6.3(c),  $d_b$  is the diameter of the hooked bar, and the first tie or stirrup shall enclose the bent portion of the hook, within  $2d_b$  of the outside of the bend.

8.2.6.4 For bars being developed by a standard hook at discontinuous ends of members with both side cover and top (or bottom) cover over hook less than 65 mm, the hooked bar shall be enclosed within ties or stirrups perpendicular to the bar being developed, spaced not greater than  $3d_b$  along  $l_{dh}$ . The first tie or stirrup shall enclose the bent portion of the hook, within  $2d_b$  of the outside of the bend, where  $d_b$  is the diameter of the hooked bar. For this case, the factors of 8.2.6.3(b) and (c) shall not apply.

8.2.6.5 Hooks shall not be considered effective in developing bars in compression.

## 8.2.7 Development of Flexural Reinforcement - General

8.2.7.1 Tension reinforcement may be developed by bending across the web to be anchored or made continuous with reinforcement on the opposite face of member.

8.2.7.2 Critical sections for development of reinforcement in flexural members are at points of maximum stress and at points within the span where adjacent reinforcement terminates, or is bent. In addition, the provisions of 8.2.8.3 shall also be satisfied.

8.2.7.3 Reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance not less than  $d$  nor less than  $12d_b$ , except at supports of simple spans and at free end of cantilevers.

8.2.7.4 Continuing reinforcement shall have an embedment length not less than the development length  $l_d$  beyond the point where the bent or terminated tension reinforcement is no longer needed to resist bending.

8.2.7.5 No flexural bar shall be terminated in a tension zone unless one of the following conditions is satisfied:

(a)  $V_u$  at the location of termination is not over two-thirds of  $\phi V_n$ .

(b) Stirrup area in excess of that normally required for shear and torsion is provided over a distance along each terminated bar or wire equal to  $0.75d$  from the point of cutoff. Excess stirrup area  $A_v$  shall be not less than  $\frac{0.41b_w s}{f_{yt}}$ . Spacing  $s$  shall not exceed  $\frac{d}{8\beta_b}$ , where  $\beta_b$  is the ratio of area of reinforcement cut off to total area of tension reinforcement at the section.

- (c) For 36 mm  $\phi$  bar and smaller, the continuing bars provide twice the area required for flexure at the cutoff point and the shear  $V_u$  does not exceed three-quarter of  $\phi V_n$ .

8.2.7.6 Where the reinforcement stress is not directly proportional to moment, such as in sloped, stepped, or tapered footings, brackets, deep flexural members, or members in which tension reinforcement is not parallel to the compression face, adequate anchorage shall be provided for the tension reinforcement. See 8.2.8.4 and 8.2.9.4 for deep flexural members.

## 8.2.8 Development of Positive Moment Reinforcement

8.2.8.1 At least one-third of the positive moment reinforcement in simple members and one-fourth of the positive moment reinforcement in continuous members shall extend along the same face of member into the support. In beams, such reinforcement shall extend into the support at least 150 mm.

8.2.8.2 When the flexural member is a part of the primary lateral load resisting system, positive moment reinforcement extended into the support by 8.2.8.1 above shall be anchored to develop the specified yield strength  $f_y$  in tension at the face of support.

8.2.8.3 At simple supports and at points of inflection, positive moment tension reinforcement shall be limited to a diameter such that  $l_d$  computed for  $f_y$  by 8.2.3 satisfies Eq (8.2.3), except that Eq (8.2.3) need not be satisfied for reinforcement terminating beyond the centreline of simple supports by a standard hook or a mechanical anchorage at least equivalent to a standard hook.

$$l_d \leq \frac{M_n}{V_u} + l_a \quad (8.2.3)$$

where

$M_n$  = nominal moment strength assuming all reinforcement at section to be stressed to  $f_y$ .

$V_u$  = factored shear force at section

$l_a$  = at a support, embedded length of bar beyond centre of support; at point of zero moment, shall be limited to  $d$  or  $12 d_b$ , whichever is greater.

The value of  $M_n/V_u$  may be increased 30 per cent when the ends of reinforcement are confined by a compressive reaction.

8.2.8.4 At simple supports of deep beams, positive moment tension reinforcement shall be anchored to develop  $f_y$  in tension at the face of the support except that if design is carried out using Appendix A, the positive moment tension reinforcement shall be anchored in accordance with A.4.3. At interior supports of deep beams, positive moment tension reinforcement shall be continuous or be spliced with that of the adjacent spans.

## 8.2.9 Development of Negative Moment Reinforcement

8.2.9.1 Negative moment reinforcement in a continuous, restrained, or cantilever member, or in any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks or mechanical anchorage.

8.2.9.2 Negative moment reinforcement shall have an embedment length into the span as required by 8.2.1, 8.2.2. and 8.2.7.3.

8.2.9.3 At least one-third of the total tension reinforcement provided for negative moment at the support shall be extended beyond the point of inflection a distance not less than  $d$ ,  $l_n/16$ , or  $12d_b$ , whichever is greater.

8.2.9.4 At interior supports of deep flexural members, negative moment tension reinforcement shall be continuous with that of the adjacent spans.

### 8.2.10 Development of Shear Reinforcement

8.2.10.1 Shear reinforcement shall be carried as close to compression and tension surfaces of member as cover requirements and proximity of other reinforcement permits.

8.2.10.2 The ends of single leg, simple U-, or multiple U-stirrups shall be anchored by one of the following means:

- (a) By a standard hook around longitudinal reinforcement for MD200 wires, and 16 mm  $\phi$  bars and smaller and for 19 mm  $\phi$  through 25 mm  $\phi$  bars with  $f_{yt} \leq 280 \text{ N/mm}^2$ .
- (b) For 19 mm  $\phi$  through 25 mm  $\phi$  stirrups with  $f_{yt}$  greater than  $280 \text{ N/mm}^2$ , a standard stirrup hook around a longitudinal bar plus an embedment between mid-height of the member and the outside end of the hook equal to or greater than  $\frac{0.17d_b f_{yt}}{\lambda \sqrt{f'_c}}$ .
- (c) For each leg of welded plain wire reinforcement forming simple U-stirrups, either: (i) Two longitudinal wires spaced at a 50 mm spacing along the member at the top of the U; or (ii) One longitudinal wire located not more than  $d/4$  from the compression face and a second wire closer to the compression face and spaced not less than 50 mm from the first wire. The second wire shall be permitted to be located on the stirrup leg beyond a bend, or on a bend with an inside diameter of bend not less than  $8d_b$ .
- (d) For each end of a single leg stirrup of welded wire reinforcement, two longitudinal wires at a minimum spacing of 50 mm and with the inner wire at least the greater of  $d/4$  or 50 mm from  $d/2$ . Outer longitudinal wire at tension face shall not be farther from the face than the portion of primary flexural reinforcement closest to the face.
- (e) In joist construction, for 13 mm  $\phi$  bar and MD130 wire and smaller, a standard hook.

8.2.10.3 Each bend in the continuous portion of a simple U-stirrup or multiple U-stirrup shall enclose a longitudinal bar between anchored ends.

8.2.10.4 If extended into the region of tension, longitudinal bars bent to act as shear reinforcement shall be continuous with longitudinal reinforcement and, if extended into a region of compression, shall be anchored beyond mid-depth  $d/2$  as specified for development length in 8.2.3 for that part of  $f_{yt}$  required to satisfy Eq (6.4.12).

8.2.10.5 Pairs of U-stirrups or ties so placed as to form a closed unit shall be considered properly spliced when length of laps are  $1.3l_d$ . In members at least 450 mm deep, such splices with  $A_b f_{yt}$  not more than 40 kN per leg shall be considered adequate if stirrup legs extend the full available depth of member.

### 8.2.11 Development of Plain Bars

For plain bars, the minimum development length shall be twice that of deformed bars specified in 8.2.1 through 8.2.10 above.

### 8.2.12 Splices of Reinforcement - General

8.2.12.1 Splices of reinforcement shall be made only as required or permitted on design drawings, or in specifications, or as authorized by the engineer.

#### 8.2.12.2 Lap Splices

- (a) Lap splices shall not be used for 36 mm  $\phi$  bars and larger, except as provided in 8.2.14.2 & 6.8.8.2.3.
- (b) Lap splices of bundled bars shall be based on the lap splice length required for individual bars within the bundle, increased in accordance with 8.2.5. Individual bar splices within a bundle shall not overlap. Entire bundles shall not be lap spliced.

- (c) Bars spliced by noncontact lap splices in flexural members shall not be spaced transversely farther apart than one-fifth the required lap splice length, nor 150 mm.

#### 8.2.12.3 Welded Splices and Mechanical Connections

- (a) Welded splices and other mechanical connections are allowed.
- (b) Except as provided in this Code, all welding shall conform to "Structural Welding Code - Reinforcing Steel" (AWS D1.4).
- (c) Welded splices shall be butted and welded to develop in tension at least 125 per cent of specified yield strength  $f_y$  of the bar.
- (d) A full mechanical connection shall develop in tension or compression, as required, at least 125 per cent of specified yield strength  $f_y$  of the bar.
- (e) Welded splices and mechanical connections not meeting the requirements of (c) or (d) above are allowed only for 16 mm  $\phi$  bars or smaller and in accordance with 8.2.13.4.

#### 8.2.13 Splices of Deformed Bars and Deformed Wire in Tension

- 8.2.13.1 The minimum length of lap for tension splices shall be as required for Class A or B splice, but not less than 300 mm, where the classification shall be as follows:

Class A splice  $1.0 l_d$

Class B splice  $1.3 l_d$

Where  $l_d$  is calculated in accordance with 8.2.3 to develop  $f_y$ , but without the 300 mm minimum of 8.2.3.1 and without the modification factor of 8.2.3.5.

- 8.2.13.2 Lap splices of deformed bars and deformed wire in tension shall be class B splices except that Class A splices are allowed when the area of reinforcement provided is at least twice that required by analysis over the entire length of the splice, and one-half or less of the total reinforcement is spliced within the required lap length.

- 8.2.13.3 Where area of reinforcement provided is less than twice that required by analysis, welded splices or mechanical connections used shall meet the requirements of 8.2.12.3(c) or 8.2.12.3(d) above.

- 8.2.13.4 Welded splices or mechanical connections not meeting the requirements of 8.2.12.3(c) or 8.2.12.3(d) shall be permitted for 16 mm  $\phi$  bars or smaller if the following requirements are met :

- (a) Splices shall be staggered at least 600 mm and in such manner as to develop at every section at least twice the calculated tensile force at the section but not less than 140 N/mm<sup>2</sup> for total area of reinforcement provided.
- (b) Spliced reinforcement stress shall be taken as the specified splice strength, in computing tensile force developed at each section, but not to exceed  $f_y$ . Unspliced reinforcement stress shall be taken as a fraction of  $f_y$  defined by the ratio of the shortest actual development length provided beyond the section to  $l_d$  but not to be taken greater than  $f_y$ .

- 8.2.13.5 When bars of different size are lap spliced in tension, splice length shall be the larger of  $l_d$  of larger bar and tension lap splice length of smaller bar.

- 8.2.13.6 Splices in tension tie members shall be made with a full welded splice or full mechanical connection in accordance with 8.2.12.3(c) or 8.2.12.3(d) and splices in adjacent bars shall be staggered at least 750 mm.

#### 8.2.14 Splices of Deformed Bars in Compression

- 8.2.14.1 The minimum length of lap for compression splice shall be  $0.071f_y d_b$  for  $f_y$  equal to 420 N/mm<sup>2</sup> or less or  $(0.13f_y - 24)d_b$  for  $f_y$  greater than 420 N/mm<sup>2</sup>, but not less than 300 mm. For  $f'_c$  less than 21 N/mm<sup>2</sup>, length of lap shall be increased by one-third.

8.2.14.2 When bars of different diameters are lap spliced in compression, the splice length shall be the larger of the development length,  $l_{dc}$  of the larger bar, and the compression splice length of the smaller bar. Lap splices of 40 mm  $\phi$ , 43 mm  $\phi$ , 50 mm  $\phi$  and 57 mm  $\phi$  bars to 36 mm  $\phi$  and smaller bars shall be permitted.

8.2.14.3 Welded splices or mechanical connections used in compression shall satisfy the requirements of 8.2.12.3(c) or 8.2.12.3(d).

#### 8.2.14.4 End Bearing Splices

- (a) Compression splices for bars required to transmit compressive stress only may consist of end bearing of square cut ends held in concentric contact by a suitable device.
- (b) Bar ends shall terminate in flat surfaces within 1.5 degrees of a right angle to the axis of the bars, and shall be fitted within 3 degrees of full bearing after assembly.
- (c) End bearing splices shall be used only in members containing closed ties, closed stirrups or spirals.

### 8.2.15 Special Splice Requirements for Columns

8.2.15.1 Lap splices, butt welded splices, mechanical connections, or end-bearing splices shall be used with the limitations of 8.2.15.2 through 8.2.15.4 below. A splice shall satisfy the requirements for all load combinations for the column.

#### 8.2.15.2 Lap Splices in Columns

- (a) Lap splices shall conform to 8.2.14.1, 8.2.14.2, and where applicable to 8.2.15.2(d) or 8.2.15.2(e) below, where the bar stress due to factored loads is compressive.
- (b) Where the bar stress due to factored loads is tensile and does not exceed  $0.5f_y$  in tension, lap splices shall be Class B tension lap splices if more than one-half of the bars are spliced at any section, or Class A tension lap splices if half or fewer of the bars are spliced at any section and alternate lap splices are staggered by  $l_d$ .
- (c) Where the bar stress due to factored loads is greater than  $0.5f_y$  in tension, lap splices shall be Class B tension lap splices.
- (d) In tied reinforced compression members, if throughout lap splice length ties have an effective area of at least  $0.0015hs$  in both directions, lap splice length is permitted to be multiplied by 0.83, but lap length shall not be less than 300 mm. Tie legs perpendicular to dimension  $h$  shall be used in determining effective area.
- (e) For spirally reinforced compression members, lap splice length of bars within a spiral is permitted to be multiplied by 0.75, but lap length shall not be less than 300 mm.

8.2.15.3 **Welded Splices or Mechanical Connectors in Columns:** Welded splices or mechanical connectors in columns shall meet the requirements of 8.2.12.3(c) or 8.2.12.3(d).

8.2.15.4 **End Bearing Splices in Columns:** End bearing splices complying with 8.2.14.4 may be used for column bars stressed in compression provided the splices are staggered or additional bars are provided at splice locations. The continuing bars in each face of the column shall have a tensile strength at least  $0.25f_y$  times the area of the vertical reinforcement in that face.

### 8.2.16 Splices of Plain Bars

For plain bars, the minimum length of lap shall be twice that of deformed bars specified in 8.2.12 through 8.2.15 above.

### 8.2.17 Development of headed and mechanically anchored deformed bars in tension

8.2.17.1 Development length for headed deformed bars in tension,  $l_{dt}$ , shall be determined from 8.2.17.2. Use of heads to develop deformed bars in tension shall be limited to conditions satisfying (a) through (f):

- (a) Bar  $f_y$  shall not exceed 420 MPa;

- (b) Bar size shall not exceed 36 mm  $\phi$ ;
- (c) Concrete shall be normalweight;
- (d) Net bearing area of head  $A_{brg}$  shall not be less than  $4A_b$ ;
- (e) Clear cover for bar shall not be less than  $2d_b$ ; and
- (f) Clear spacing between bars shall not be less than  $4d_b$ .

8.2.17.2 For headed deformed bars, development length in tension  $l_{dt}$  shall be  $(0.19\psi_e f_y / \sqrt{f'_c})d_b$ , where the value of  $f'_c$  used to calculate  $l_{dt}$  shall not exceed 40 MPa, and factor  $\psi_e$  shall be taken as 1.2 for epoxy-coated reinforcement and 1.0 for other cases. Where reinforcement provided is in excess of that required by analysis, except where development of  $f_y$  is specifically required, a factor of  $(A_{s,required}) / (A_{s,provided})$  may be applied to the expression for  $l_{dt}$ . Length  $l_{dt}$  shall not be less than the larger of  $8d_b$  and 150 mm.

8.2.17.3 Heads shall not be considered effective in developing bars in compression.

8.2.17.4 Any mechanical attachment or device capable of developing  $f_y$  of reinforcement is allowed, provided that test results showing the adequacy of such attachment or device are approved by the Engineer. Development of reinforcement shall be permitted to consist of a combination of mechanical anchorage plus additional embedment length of reinforcement between the critical section and the mechanical attachment or device.

### 8.2.18 Development of welded deformed wire reinforcement in tension

8.2.18.1 Development length for welded deformed wire reinforcement in tension,  $l_d$ , measured from the point of critical section to the end of wire shall be computed as the product of  $l_{dl}$  from 8.2.3.2 or 8.2.3.3, times welded deformed wire reinforcement factor,  $\psi_w$ , from 8.2.18.2 or 8.2.18.3. It shall be permitted to reduce  $l_d$  in accordance with 8.2.3.5 when applicable, but  $l_d$  shall not be less than 200 mm except in computation of lap splices by 8.2.20. When using  $\psi_w$  from 8.2.18.2, it shall be permitted to use an epoxy-coating factor  $\psi_e$  of 1.0 for epoxy-coated welded deformed wire reinforcement in 8.2.3.2 and 8.2.3.3.

8.2.18.2 For welded deformed wire reinforcement with at least one cross wire within  $l_d$  and not less than 50 mm from the point of the critical section,  $\psi_w$  shall be the greater of

$$\left( \frac{f_y - 240}{f_y} \right)$$

And

$$\left( \frac{5d_b}{s} \right)$$

but not greater than 1.0, where  $s$  is the spacing between the wires to be developed.

8.2.18.3 For welded deformed wire reinforcement with no cross wires within  $l_d$  or with a single cross wire less than 50 mm from the point of the critical section,  $\psi_w$  shall be taken as 1.0, and  $l_d$  shall be determined as for deformed wire.

8.2.18.4 Where any plain wires, or deformed wires larger than D-31, are present in the welded deformed wire reinforcement in the direction of the development length, the reinforcement shall be developed in accordance with 8.2.19.

### 8.2.19 Development of welded plain wire reinforcement in tension

Yield strength of welded plain wire reinforcement shall be considered developed by embedment of two cross wires with the closer cross wire not less than 50 mm from the point of the critical section. However,  $l_{dl}$  shall not be less than

$$l_d = 3.3 \frac{A_b f_y}{s \lambda \sqrt{f'_c}} \quad (8.2.4)$$

where  $l_d$  is measured from the point of the critical section to the outermost crosswire,  $s$  is the spacing between the wires to be developed, and  $\lambda$  as given in 8.2.3.4(d). Where reinforcement provided is in excess of that required,  $l_d$  may be reduced in accordance with 8.2.3.5. Length,  $l_d$ , shall not be less than 150 mm except in computation of lap splices by 8.2.21.

### 8.2.20 Splices of welded deformed wire reinforcement in tension

8.2.20.1 Minimum lap splice length of welded deformed wire reinforcement measured between the ends of each reinforcement sheet shall be not less than the larger of  $1.3l_d$  and 200 mm, and the overlap measured between outermost cross wires of each reinforcement sheet shall be not less than 50 mm, where  $l_d$  is calculated in accordance with 8.2.18 to develop  $f_y$ .

8.2.20.2 Lap splices of welded deformed wire reinforcement, with no cross wires within the lap splice length, shall be determined as for deformed wire.

8.2.20.3 Where any plain wires, or deformed wires larger than MD200, are present in the welded deformed wire reinforcement in the direction of the lap splice or where welded deformed wire reinforcement is lap spliced to welded plain wire reinforcement, the reinforcement shall be lap spliced in accordance with 8.2.21.

### 8.2.21 Splices of welded plain wire reinforcement in tension

Minimum length of lap for lap splices of welded plain wire reinforcement shall be in accordance with 8.2.21.1 and 8.2.21.2.

8.2.21.1 Where  $A_s$  provided is less than twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each reinforcement sheet shall be not less than the largest of one spacing of cross wires plus 50 mm,  $1.5 l_d$ , and 150 mm, where  $l_d$  is calculated in accordance with 8.2.19 to develop  $f_y$ .

8.2.21.2 Where  $A_s$  provided is at least twice that required by analysis at splice location, length of overlap measured between outermost cross wires of each reinforcement sheet shall not be less than the larger of  $1.5 l_d$ , and 50 mm, where  $l_d$  is calculated in accordance with 8.2.19 to develop  $f_y$ .

## 8.3 EARTHQUAKE-RESISTANT DESIGN PROVISIONS

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### 8.3.1 Notation

- $A_{ch}$  = cross-sectional area of a structural member measured out to out of transverse reinforcement, mm<sup>2</sup>
- $A_{cp}$  = area of concrete section resisting shear of an individual pier or horizontal wall segment, mm<sup>2</sup>
- $A_{cv}$  = net area of concrete section bounded by web thickness and length of section in the direction of shear force considered, mm<sup>2</sup>
- $A_g$  = gross area of section, mm<sup>2</sup>
- $A_j$  = effective cross-sectional area within a joint, see 8.3.7.3, in a plane parallel to plane of reinforcement generating shear in the joint. The joint depth shall be the overall depth of the column. Where a beam frames into a support of larger width, the effective width of the joint shall not exceed the smaller of :
- Beam width plus the joint depth
  - twice the smaller perpendicular distance from the longitudinal axis of the beam to the column side (See 8.3.7.3)



$A_{sh}$	= total cross-sectional area of transverse reinforcement (including cross ties) within spacing $s$ and perpendicular to dimension $h_c$
$b$	= effective compressive flange width of a structural member, mm
$b_w$	= web width or diameter of circular section, mm
$d_b$	= bar diameter, mm
$E$	= load effects of earthquake or related internal moments and forces
$f'_c$	= specified compressive strength of concrete
$f_y$	= specified yield strength of reinforcement
$f_{yt}$	= specified yield strength of transverse reinforcement
$h_c$	= cross-sectional dimension of column core measured centre to centre of confining reinforcement
$h_w$	= height of entire wall (diaphragm) or of the segment of wall (diaphragm) considered
$l_d$	= development length for a straight bar
$l_{dh}$	= development length for a bar with a standard hook
$l_o$	= minimum length, measured from joint face along axis of structural member, over which transverse reinforcement must be provided, mm
$l_w$	= length of entire wall (diaphragm) or of segment of wall (diaphragm) considered in the direction of shear force
$M_{pr}$	= probable flexural moment strength of members, with or without axial load, determined using the properties of the member at the joint faces assuming a tensile strength in the longitudinal bars of at least $1.25f_y$ , and a strength reduction factor $\phi$ of 1.0
$M_s$	= portion of slab moment balanced by support moment
$s$	= spacing of transverse reinforcement measured along the longitudinal axis of the structural member, mm
$s_o$	= maximum spacing of transverse reinforcement, mm
$V_c$	= nominal shear strength provided by concrete
$V_e$	= design shear force
$V_n$	= nominal shear strength
$V_u$	= factored shear force at section
$\alpha_c$	= coefficient defining the relative contribution of concrete strength to wall strength
$\rho$	= ratio of tension reinforcement = $A_s/bd$
$\rho_g$	= ratio of total reinforcement area to cross-sectional area of column
$\rho_n$	= ratio of distributed shear reinforcement on a plane perpendicular to plane of $A_{cv}$
$\rho_s$	= ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out to out)
$\rho_v$	= $A_{sv}/A_{cv}$ ; where $A_{sv}$ is the projection on $A_{cv}$ of area of distributed shear reinforcement crossing the plane of $A_{cv}$
$\phi$	= strength reduction factor.

### 8.3.2 Definitions

**BASE OF STRUCTURE :** The level at which earthquake motions are assumed to be imparted to a structure. This level does not necessarily coincide with the ground level.

**BOUNDARY MEMBERS** : Members along wall and diaphragm edges strengthened by longitudinal and transverse reinforcement. These members do not necessarily require an increase in the thickness of the wall or diaphragm. If required, edges of openings within walls and diaphragms shall be provided with boundary members.

**COLLECTOR ELEMENTS** : Elements that are used to transmit the inertial forces within the diaphragms to members of the lateral force resisting systems.

**CROSS TIE** : A continuous bar having a hook not less than 135 deg with at least a six diameter extension at one end but not less than 75 mm, and a hook not less than 90 deg with at least a six diameter extension at the other end. The hooks shall engage peripheral longitudinal bars. The 90 deg hooks of two successive cross ties engaging the same longitudinal bars shall be alternated end for end.

**DEVELOPMENT LENGTH FOR A BAR WITH A STANDARD HOOK** : The shortest distance between the critical section and a tangent to the outer edge of the 90 deg hook.

**HOOP** : A hoop is a closed tie or continuously round tie. A closed tie can be made up of several reinforcing elements with 135 deg hooks having a six diameter extension at each end (but not less than 75 mm). A continuously round tie shall have at each end a 135 deg hook with a six diameter extension that engages the longitudinal reinforcement but not less than 75 mm.

**LATERAL FORCE RESISTING SYSTEM** : That portion of the structure composed of members designed to resist forces related to earthquake effects.

**SHELL CONCRETE** : Concrete outside the transverse reinforcement confining the concrete

**STRUCTURAL DIAPHRAGMS** : Structural members, such as floor and roof slabs, which transmit inertial forces to lateral force resisting members.

**STRUCTURAL WALLS** : Walls designed to resist combinations of shears, moments, and axial forces induced by earthquake motions. A shear wall is a structural wall.

**STRUT** : An element of a structural diaphragm used to provide continuity around an opening in the diaphragm.

**TIE ELEMENTS** : Elements used to transmit inertial forces and prevent separation of building components.

### 8.3.3 General Requirements

#### 8.3.3.1 Scope

- (a) This section contains special requirements for design and construction of reinforced concrete members of a structure for which the design forces, related to earthquake motions, have been determined on the basis of energy dissipation in the nonlinear range of response.
- (b) The provisions of Chapter 6, shall apply except as modified by the provisions of this section.
- (c) Structures assigned to seismic design category SDC D (see Chapter 2), all reinforced concrete structures shall satisfy the requirements of special moment frames as given in 8.3.3 through 8.3.8 in addition to the requirements of Chapter 6.
- (d) Structures assigned to SDC C (see Chapter 2), all reinforced concrete structures shall be built to satisfy the requirements of intermediate moment frames as given in 8.3.10 in addition to the requirements of Chapter 6.
- (e) Structures assigned to SDC B (see Chapter 2), all reinforced concrete structures shall be built to satisfy the requirements of ordinary moment frames as given in 8.3.9 in addition to the requirements of Chapter 6.
- (f) Structures in lower SDCs are permitted to design with detailing provisions of higher SDCs to take advantage of lower design force levels.

#### 8.3.3.2 Analysis and Proportioning of Structural Members

- (a) The interaction of all structural and nonstructural members shall be considered in the analysis.
- (b) Rigid members which are not a part of the lateral force resisting system are allowed provided their effect on the response of the system is considered and accommodated in the structural design. Consequences of failure of structural and nonstructural members which are not a part of the lateral force resisting system shall also be considered.
- (c) Structural members below base of structure required to transmit forces resulting from earthquake effects to the foundation shall also comply with the requirements of this section.
- (d) All structural members which are not a part of the lateral force resisting system shall conform to 8.3.9.

### 8.3.3.3 Strength Reduction Factors

Strength reduction factors shall be in accordance with 6.2.3.2 to 6.2.3.4.

### 8.3.3.4 Concrete in Special Moment Frames and Special Structural Walls

Compressive strength  $f'_c$  of the concrete shall be not less than  $21 \text{ N/mm}^2$ . Specified compressive strength of lightweight concrete,  $f'_c$ , shall not exceed  $35 \text{ MPa}$  unless demonstrated by experimental evidence. Modification factor  $\lambda$  for lightweight concrete in Sec 8.3 shall be in accordance with 6.1.8 unless noted otherwise.

### 8.3.3.5 Reinforcement in Special Moment Frames and Special Structural Walls

Reinforcement resisting earthquake induced flexural and axial forces in frames and wall boundary members shall comply with ASTM A706M, ASTM A615M and BDS ISO 6935-2: 2007(E). Reinforcement with  $f_y = 275 \text{ N/mm}^2$  and  $f_y = 420 \text{ N/mm}^2$  are allowed in these members if (a) the actual yield strength based on mill tests does not exceed the specified yield strength by more than  $125 \text{ N/mm}^2$  (retests shall not exceed this value by more than an additional  $20 \text{ N/mm}^2$ ), and (b) the ratio of the actual ultimate tensile strength to the actual tensile yield strength is not less than 1.25. The value of  $f_{yt}$  used to compute the amount of confinement reinforcement shall not exceed  $700 \text{ MPa}$ . The value of  $f_y$  or  $f_{yt}$  used in the design of shear reinforcement shall conform to 6.4.3.2.

### 8.3.3.6 Welding

Reinforcement required by factored load combinations which include earthquake effect shall not be welded except as specified in 8.3.4.2(d) and 8.3.5.3(b). In addition, welding shall not be permitted on stirrups, ties, inserts, or other similar elements to longitudinal reinforcement required by design.

## 8.3.4 Flexural Members of Special Moment Frames

### 8.3.4.1 Scope

Requirements of this section shall apply to special moment frame members; (i) resisting earthquake induced forces, and (ii) proportioned primarily to resist flexure. These frame members shall also satisfy the following conditions:

- Factored axial compressive force on frame member shall not exceed  $0.1A_g f'_c$ .
- Clear span for the member,  $l_n$  shall not be less than four times its effective depth.
- The width to depth ratio shall be at least 0.3.
- The width shall not be (i) less than  $250 \text{ mm}$  and (ii) more than the width of the supporting member (measured on a plane perpendicular to the longitudinal axis of the flexural member) plus distances on each side of the supporting member not exceeding three-fourths of the depth of the flexural member.

These requirements are shown in Fig. 8.3.1.

### 8.3.4.2 Longitudinal Reinforcement

- At any section of a flexural member and for the top as well as for the bottom reinforcement, the amount of reinforcement shall be not less than  $1.4b_w d / f_y$  and the reinforcement ratio,  $\rho$  shall not exceed 0.025 (Fig. 8.3.2). At least two bars shall be provided continuously both top and bottom.
- The positive moment strength at the face of the joint shall be not less than one-half of the negative moment strength provided at that face as shown in Fig.8.3.2. Neither the negative nor the positive moment strength at any section along the member length shall be less than one-fourth the maximum moment strength provided at the face of either joint.
- Lap splices of flexural reinforcement shall be permitted only if hoop or spiral reinforcement is provided over the lap length. Maximum spacing of the transverse reinforcement enclosing the lapped bars shall not exceed  $d/4$  nor  $100 \text{ mm}$ . Lap splices shall not be used (i) within the joints, (ii) within a distance of twice the member depth from the face of the joint, and (iii) at locations where analysis indicates flexural yielding caused by inelastic lateral displacements of the frame. These requirements are shown in Fig.8.3.3.
- Welded splices and mechanical connections conforming to 8.2.12.3(a) through 8.2.12.3(d) are allowed for splicing provided not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the centre to centre distance between splices of adjacent bars is  $600 \text{ mm}$  or more measured along the longitudinal axis of the frame member. Welded splices

and mechanical connections (Type 1) shall not be used within a distance equal to twice the member depth from the column or beam faces for special moment frames or from sections where yielding of the reinforcement is likely to occur as a result of inelastic lateral displacement.

### 8.3.4.3 Transverse Reinforcement

- (a) Hoops shall be provided in the following regions of frame members:
  - i. At both ends of the flexural member, over a length equal to twice the member depth measured from the face of the supporting member toward midspan (Fig. 8.3.4).
  - ii. Over lengths equal to twice the member depth (Fig. 8.3.4), on both sides of a section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame.
- (b) The first hoop shall be located not more than 50 mm from the face of the supporting member (Fig. 8.3.4). Maximum spacing of the hoops shall not exceed (i)  $d/4$ , (ii) eight times the diameter of the smallest longitudinal bars, (iii) 24 times the diameter of the hoop bars, and (iv) 300 mm.
- (c) Where hoops are required, longitudinal bars on the perimeter shall have lateral support conforming to 8.1.9.4(c), and where hoops are not required, stirrups with seismic hooks shall be spaced not more than  $d/2$  throughout the length of the member (Fig. 8.3.4).
- (d) Hoops in flexural members are allowed to be made up of two pieces of reinforcement consisting of a U-stirrup having hooks not less than 135 deg with 6 diameter but not less than 75 mm extension anchored in the confined core and a cross tie to make a closed hoop (Fig. 8.3.5). Consecutive cross ties engaging the same longitudinal bar shall have their 90 deg hooks at opposite sides of the flexural member. If the longitudinal reinforcing bars secured by the cross ties are confined by a slab only on one side of the flexural frame member, the 90 deg hooks of the cross ties shall all be placed on that side.

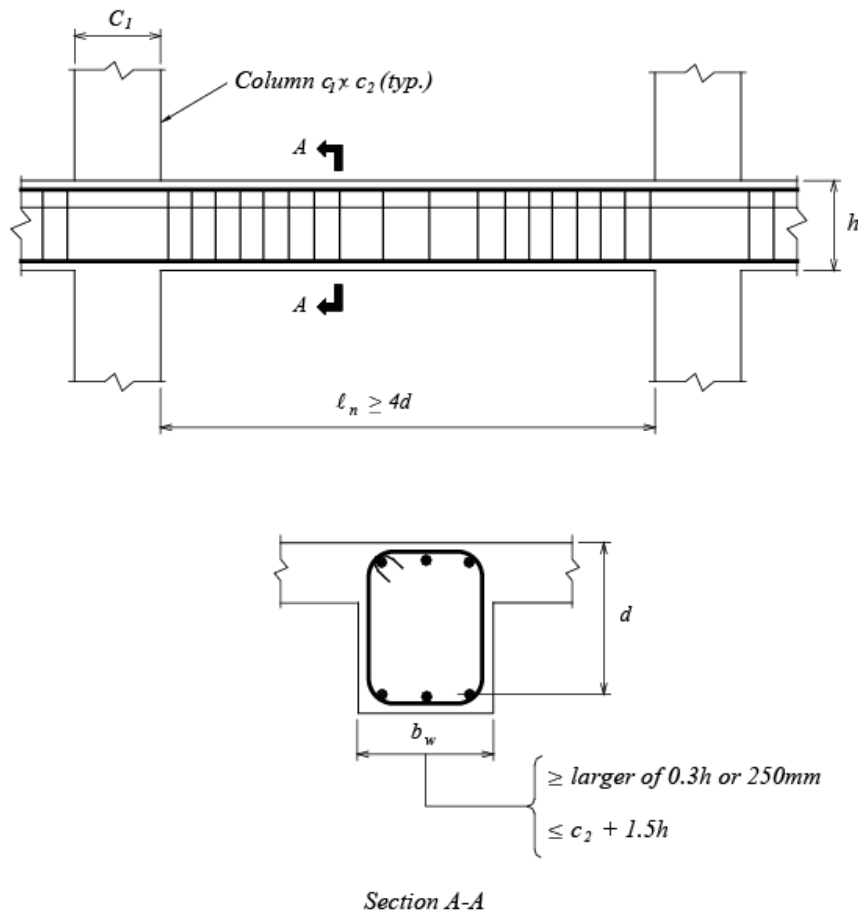


Fig. 8.3.1. General requirement for Flexural Members of Special Moment Frames

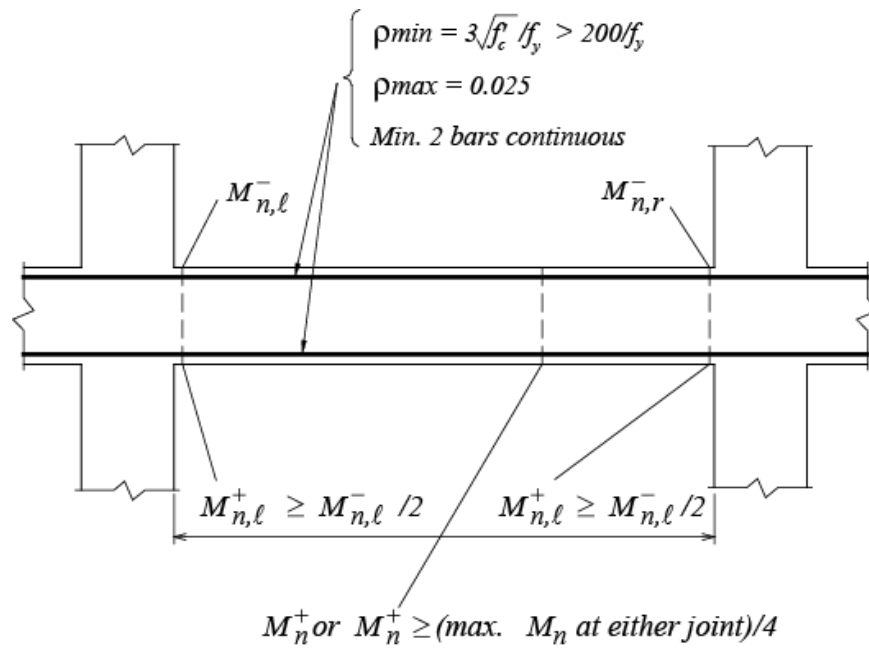
### 8.3.5 Special Moment Frame Members Subjected to Bending and Axial Load

#### 8.3.5.1 Scope

The requirements of this section shall apply to columns and other frame members serving to resist earthquake forces and having a factored axial force exceeding  $0.1A_g f'_c$ . These frame members shall also satisfy the following conditions:

- (a) The shortest cross-sectional dimension shall not be less than 300 mm.
- (b) The ratio of the shortest cross-sectional dimension to the perpendicular dimension shall not be less than 0.4.

These requirements are shown in Fig. 8.3.6.



Note: transverse reinforcement not shown for clarity

Fig. 8.3.2 Flexural Requirements for Flexural Members of Special Moment Frames

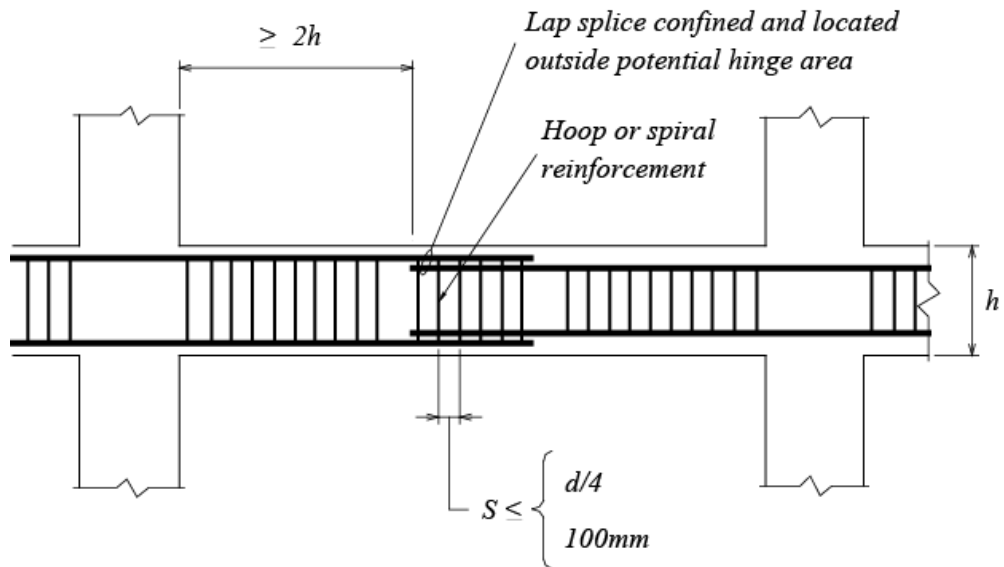


Fig. 8.3.3 Lap Splice Requirements for Flexural Members of Special Moment Frames

#### 8.3.5.2 Minimum Flexural Strength of Columns

(a) Flexural strength of any column designed to resist a factored axial compressive force exceeding  $0.1A_g f'_c$  shall satisfy (b) or (c) below. Lateral strength and stiffness of columns not satisfying (b) below shall be ignored in calculating the strength and stiffness of the structure but shall conform to 8.3.9.

(b) The flexural strength of the columns shall satisfy the following relation:

$$\sum M_c \geq 1.2 \sum M_g \quad (8.3.1)$$

where

$\sum M_c$  = sum of moments, at the centre of the joint, corresponding to the design flexural strength of the columns framing into that joint. The lowest flexural strength of the columns, calculated for the factored axial force, consistent with the direction of the lateral forces considered, shall be used.

$\sum M_g$  = sum of moments, at the centre of the joint, corresponding to the design flexural strengths of the girders framing into that joint.

Flexural strengths shall be summed such that the column moments oppose the beam moments. Eq (8.3.1) shall be satisfied for beam moments acting in both directions in the vertical plane of the frame considered.

(c) If the requirements of (b) above is not satisfied at a joint, columns supporting reactions from that joint shall be provided with transverse reinforcement as specified in 8.3.5.4 over their entire height.

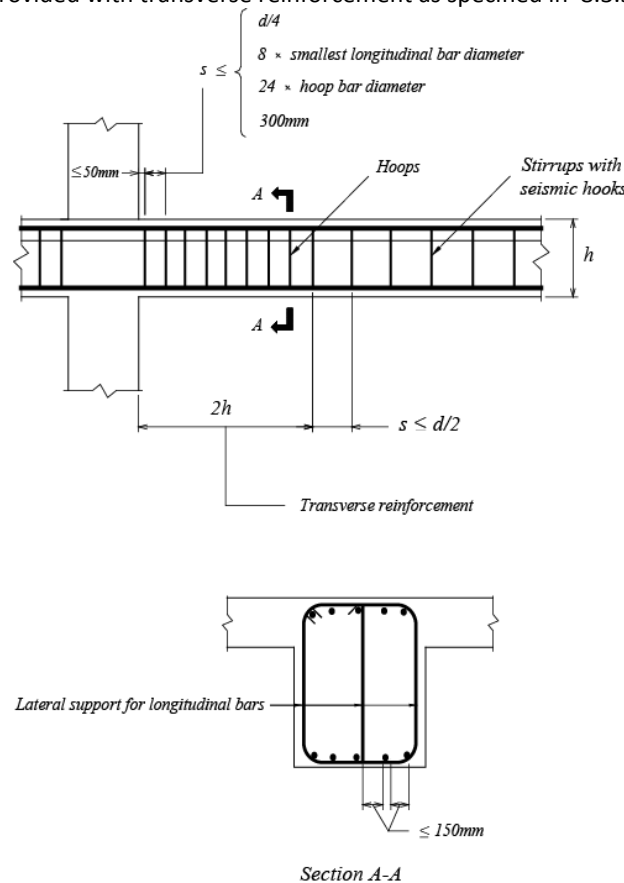


Fig. 8.3.4 Transverse Reinforcement Requirements for Flexural Members of Special Moment Frames

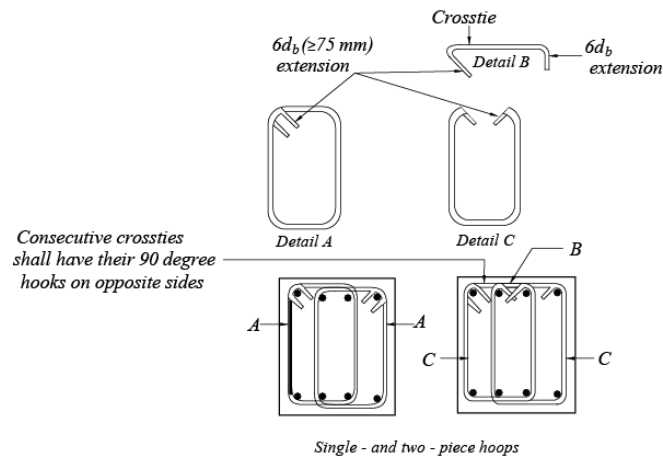


Fig. 8.3.5 Hoop Reinforcement Requirements for Flexural Members of Special Moment Frames

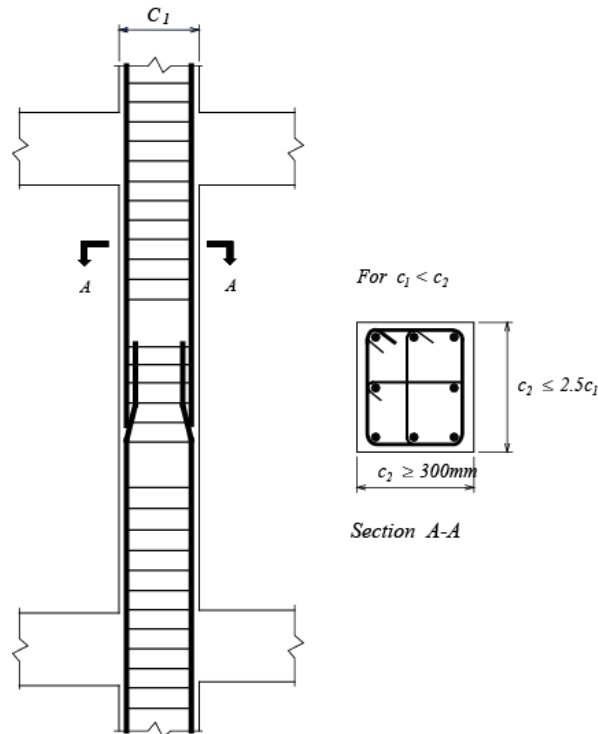


Fig. 8.3.6 General Requirements for frames subjected to bending and axial load

### 8.3.5.3 Longitudinal Reinforcement

- (a) The reinforcement ratio,  $\rho_g$ , shall not be less than 0.01 and shall not exceed 0.06.
- (b) Lap splices are permitted only within the centre half of the member length and shall be designed as tension splices. Welded splices and mechanical connections conforming to 8.2.12.3(a) through 8.2.12.3(d) are allowed for splicing the reinforcement at any section provided not more than alternate longitudinal bars are spliced at a section and the distance between splices is 600 mm or more along the longitudinal axis of the reinforcement.

These provisions are shown in Fig. 8.3.7.

### 8.3.5.4 Transverse Reinforcement

(a) Transverse reinforcement shall be provided as specified below and shown in Fig. 8.3.8 and Fig. 8.3.9 unless a larger amount is required by 8.3.8.

- i) The volumetric ratio of spiral or circular hoop reinforcement,  $\rho_s$  shall not be less than that indicated by the following equation:

$$\rho_s = \frac{0.12f'_c}{f_{yt}} \quad (8.3.2)$$

and shall not be less than that required by Eq (6.3.6).

- ii) The total cross-sectional area of rectangular hoop reinforcement shall not be less than that given by the following equations:

$$A_{sh} = 0.3(sh_c f'_c / f_{yt}) [(A_g / A_{ch}) - 1] \quad (8.3.3)$$

$$A_{sh} = \frac{0.09sh_c f'_c}{f_{yt}} \quad (8.3.4)$$

- iii) Transverse reinforcement shall be provided by either single or overlapping hoops or cross ties of the same bar size and spacing. Each end of the cross ties shall engage a peripheral longitudinal reinforcing bar. Consecutive cross ties shall be alternated end for end along the longitudinal reinforcement.
- iv) If the design strength of member core satisfies the requirements of the specified loading combinations including earthquake effect, Eq (8.3.3) and (6.3.6) need not be satisfied.

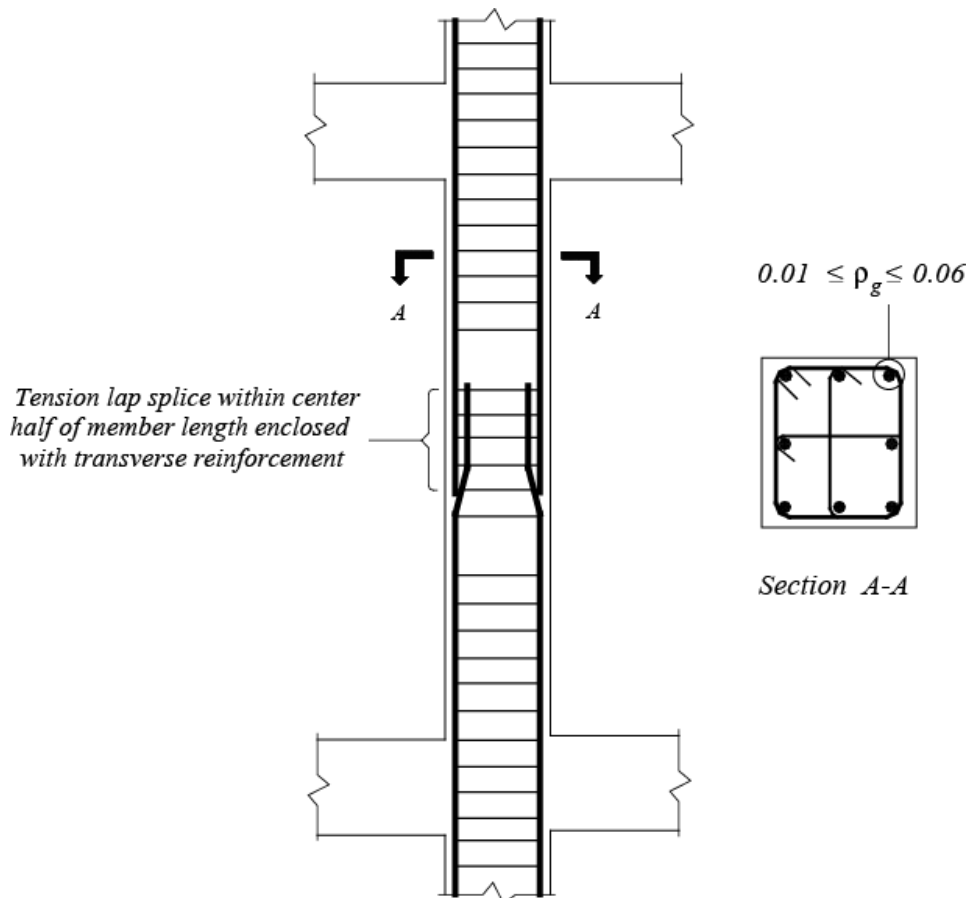


Fig. 8.3.7 Longitudinal Reinforcement Requirements

- (b) Transverse reinforcement shall not be spaced more than one-quarter of the minimum member dimension nor 100 mm.
- (c) Spacing of cross ties or legs of overlapping hoops shall not be more than 350 mm on centre in the direction perpendicular to the longitudinal axis of the member.



- (d) The volume of transverse reinforcement in amount specified in (a) through (c) above shall be provided over a length  $l_o$  from each joint face and on both sides of any section where flexural yielding is likely to occur in connection with inelastic lateral displacements of the frame. The length  $l_o$  shall not be less than (i) the depth of the member at the joint face or at the section where flexural yielding is likely to occur, (ii) one-sixth of the clear span of the member, and (iii) 450 mm.
- (e) If the factored axial force in columns supporting reactions from discontinued stiff members, such as walls, exceeds  $0.1A_g f'_c$  they shall be provided with transverse reinforcement as specified in (a) through (c) above over their full height beneath the level at which the discontinuity occurs. Transverse reinforcement shall extend into the discontinued member for at least the development length of the largest longitudinal reinforcement in the column in accordance with 8.3.7.4. If the lower end of the column terminates on a wall, transverse reinforcement as specified above shall extend into the wall for at least the development length of the largest longitudinal reinforcement in the column at the point of termination. If the column terminates on a footing or mat, transverse reinforcement as specified in above shall extend at least 300 mm into the footing or mat.
- (f) Where transverse reinforcement as specified in (a) through (c) above, is not provided throughout the full length of the column, the remainder of the column length shall contain spiral or hoop reinforcement with centre to centre spacing not exceeding the smaller of 6 times the diameter of the longitudinal column bars or 150 mm.

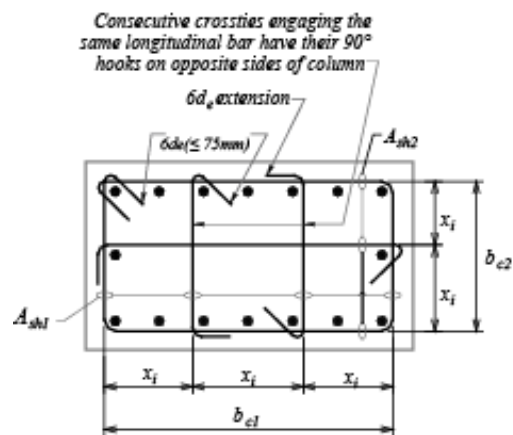
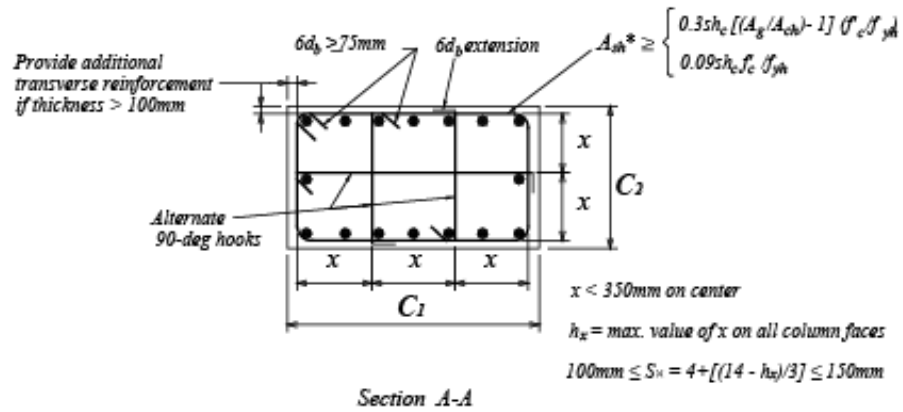
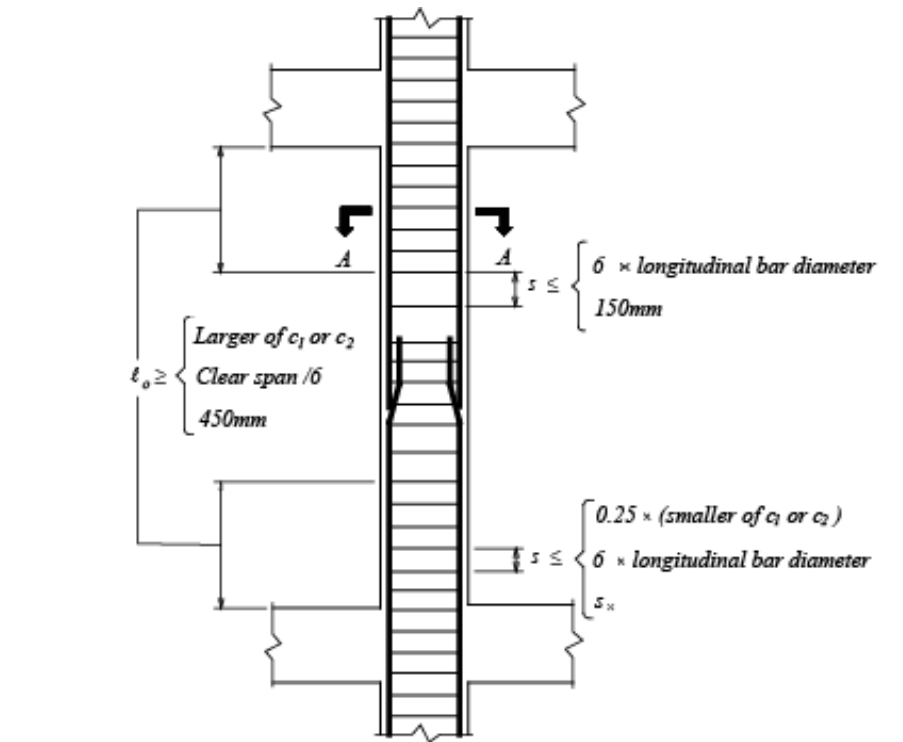


Fig. 8.3.8 Transverse Reinforcement Requirements- Rectangular Hoop

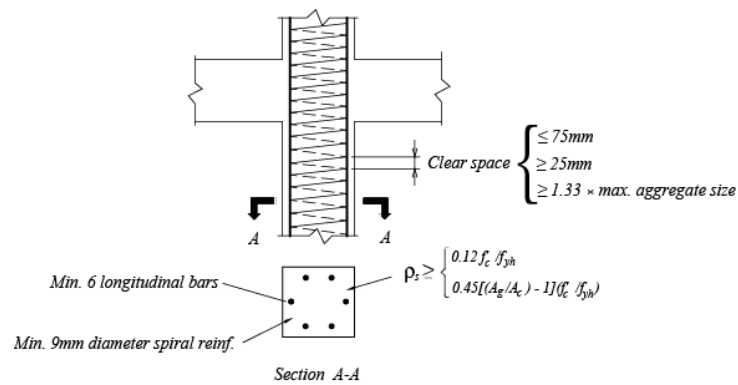


Fig. 8.3.9 Transverse Reinforcement Requirements- Spiral Hoop

## 8.3.6 Special Structural Walls and Diaphragms

### 8.3.6.1 Scope

The requirements of this section apply to structural walls serving as parts of the earthquake force resisting systems as well as to diaphragms, struts, ties, chords and collector members which transmit forces induced by earthquake.

### 8.3.6.2 Reinforcement

- The reinforcement ratio,  $\rho_v$  for structural walls shall not be less than 0.0025 along the longitudinal and transverse directions. Reinforcement spacing each way shall not exceed 450 mm. Reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane. If the design shear force does not exceed  $0.083A_{cv}\lambda\sqrt{f'_c}$ , the shear reinforcement may conform to 6.6.3.
- At least two layers of reinforcement shall be used in a wall if the in-plane factored shear force assigned to the wall exceeds  $0.17A_{cv}\lambda\sqrt{f'_c}$ .
- Structural truss members, struts, ties, and collector members with compressive stresses exceeding  $0.2 f'_c$  shall have special transverse reinforcement, as specified in 8.3.5.4 over the total length of the member. The special transverse reinforcement is allowed to be discontinued at a section where the calculated compressive stress is less than  $0.15 f'_c$ . Stresses shall be calculated for the factored forces using a linear elastic model and gross section properties of the members considered.
  - All continuous reinforcement in structural walls, diaphragms, trusses, struts, ties, chords, and collector members shall be anchored or spliced in accordance with the provisions for reinforcement in tension as specified in 8.3.7.4.

### 8.3.6.3 Boundary Members for Structural Walls and Diaphragms

- Boundary members shall be provided at boundaries and edges around openings of structural walls and diaphragms for which the maximum extreme fibre stress exceeds  $0.2 f'_c$  unless the entire wall or diaphragm member is reinforced to satisfy 8.3.5.4(a) through 8.3.5.4(c). The boundary members may be discontinued where the calculated compressive stress is less than  $0.15 f'_c$ . Stresses shall be calculated for the factored forces using a linearly elastic model and gross section properties.
- Where required, boundary members shall have transverse reinforcement as specified in 8.3.5.4(a) through 8.3.5.4(c).
- Boundary members of structural walls shall be designed to carry all factored gravity loads on the wall, including tributary loads and self weight, as well as the vertical force required to resist overturning moment calculated from factored forces related to earthquake effect.
- Boundary members of structural diaphragms shall be proportioned to resist the sum of the factored axial force acting in the plane of the diaphragm and the force obtained from dividing the factored moment at the section by the distance between the edges of the diaphragm at that section.
- Transverse reinforcement in walls with boundary members shall be anchored within the confined core of the boundary member to develop the tensile yield stress.
- Transverse reinforcement terminating at the edges of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be

enclosed in the U-stirrups having the same size and spacing as, and spliced to, the transverse reinforcement, except when  $V_u/V_u$  in the plane of the wall is less than  $0.083A_{cv}\lambda\sqrt{f'_c}$ .

### 8.3.6.4 Construction Joints

All construction joints in walls and diaphragms shall conform to 5.16.4 and contact surfaces shall be roughened as specified in 6.4.5.9.

### 8.3.6.5 Discontinuous Walls

Columns supporting discontinuous walls shall be reinforced in accordance with 8.3.5.4(e).

## 8.3.7 Joints of Special Moment Frames

### 8.3.7.1 General Requirements

- Forces in longitudinal beam reinforcement at the faces of joints of reinforced concrete frames shall be determined for a stress of  $1.25 f_y$  in the reinforcement.
- Joint strength shall be calculated by the appropriate strength reduction factors specified in 6.2.3.1.
- Beam longitudinal reinforcement terminated in a column shall be extended to the far face of the confined column core and anchored in tension according to 8.3.7.4 below and in compression according to Sec 8.2.

### 8.3.7.2 Transverse Reinforcement

- As specified in 8.3.5.4, transverse hoop reinforcement shall be provided within the joint, unless the joint is confined by structural members as specified in (b) below.
- Within the depth of the shallowest framing member, transverse reinforcement equal to at least one-half the amount required by 8.3.5.4(a) shall be provided where members frame into all four sides of the joint and where each member width is at least three-fourths the column width. At these locations, the spacing specified in 8.3.5.4(b) may be increased to 150 mm.
- As required by 8.3.5.4, transverse reinforcement shall be provided through the joint to provide confinement for longitudinal beam reinforcement outside the column core if such confinement is not provided by a beam framing into the joint.

These provisions are shown in Fig.8.3.10 and Fig.8.3.11.

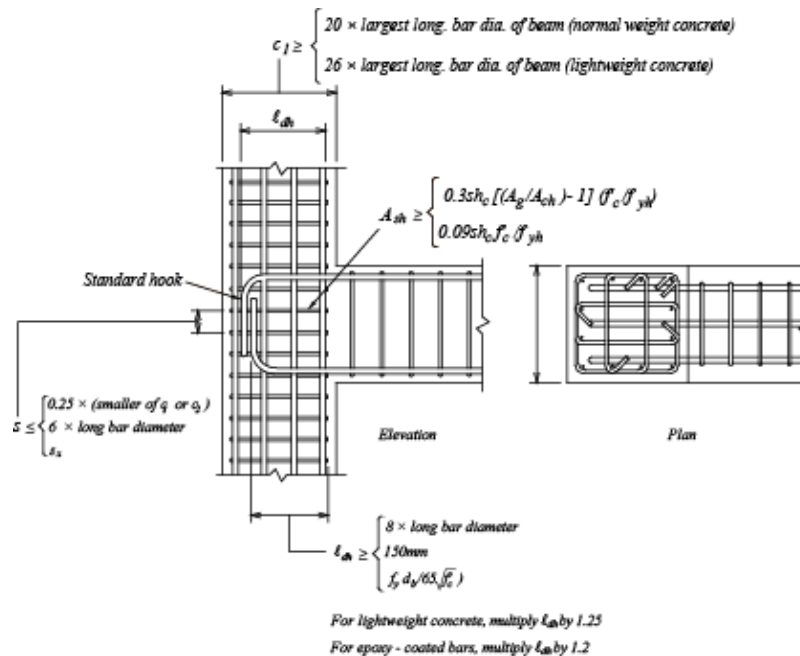


Fig. 8.3.10 General Requirements and Transverse Reinforcement Requirements for Joints not confined by Structural member

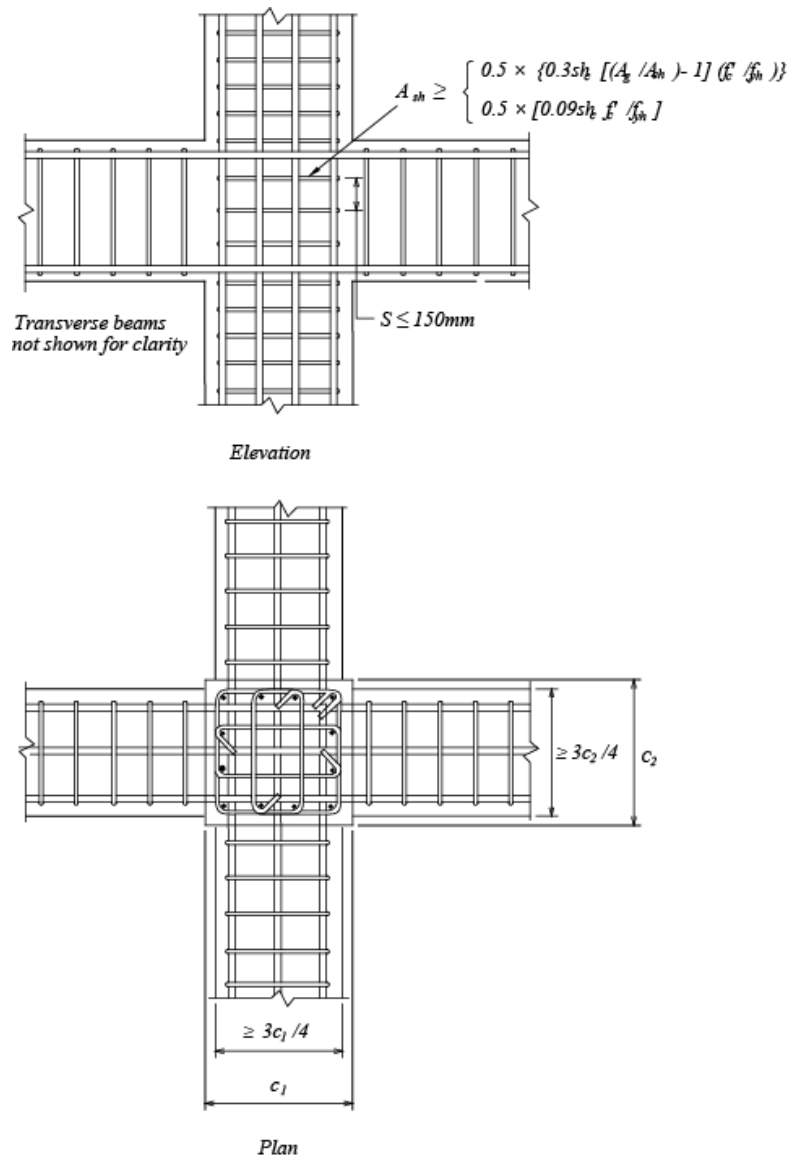


Fig. 8.3.11 Transverse Reinforcement Requirements for Joints Confined by Structural member

8.3.7.3 Shear Strength

The nominal shear strength for the joint shall be taken not greater than the forces specified below :

1.7√*f*<sub>c</sub>' *A*<sub>j</sub> for joints confined on all four faces

1.2√*f*<sub>c</sub>' *A*<sub>j</sub> for joints confined on three faces or on two opposite faces

1.0√*f*<sub>c</sub>' *A*<sub>j</sub> for others

A member that frames into a face is considered to provide confinement to the joint if at least three-quarters of the face of the joint is covered by the framing member. A joint is considered to be confined if such confining members frame into all faces of the joint.

8.3.7.4 Development Length of Bars in Tension

- (a) The development length, *l*<sub>dh</sub>, for a bar with a standard 90° hook shall be not less than (i) 8*d*<sub>b</sub>, (ii) 150 mm, and (iii) the length required by Eq (8.3.5).

$$l_{dh} = \frac{f_y d_b}{5.4 \sqrt{f_c'}} \tag{8.3.5}$$

for bar sizes 10 mm φ through 36 mm φ.

- (b) For bar sizes 10 mm  $\phi$  through 36 mm  $\phi$ , the development length,  $l_d$ , for a straight bar shall be not less than (i) 2.5 times the length required by (a) above, if the depth of the concrete cast in one lift beneath the bar does not exceed 300 mm, and (ii) 3.5 times the length required by (a) above, if the depth of the concrete cast in one lift beneath the bar exceeds 300 mm.
- (c) Straight bars terminated at a joint shall pass through the confined core of a column or of a boundary member. Any portion of the straight embedment length not within the confined core shall be increased by a factor of 1.6.

### 8.3.8 Shear Strength Requirements

#### 8.3.8.1 Design Forces

- (a) Frame Members Subjected Primarily to Bending: The design shear force  $V_e$  shall be determined from consideration of the statical forces on the portion of the member between faces of the joints. It shall be assumed that moments of opposite sign corresponding to probable strength  $M_{pr}$  act at the joint faces, and that the member is loaded with the factored tributary gravity load along its span.
- (b) Frame Members Subjected to Combined Bending and Axial Load: The design shear force  $V_e$  shall be determined from consideration of the maximum forces that can be generated at the faces of the joints at each end of the member. These joint forces shall be determined using the maximum probable moment strengths  $M_{pr}$  of the member associated with the range of factored axial loads on the member. The member shears need not exceed those determined from joint strengths based on the probable moment strength  $M_{pr}$  of the transverse members framing into the joint. In no case,  $V_e$  shall be less than the factored shear determined by the analysis of the structure.
- (c) Structural Walls and Diaphragms: The design shear force  $V_e$  shall be obtained from the lateral load analysis in accordance with the factored loads and combinations specified in Chapter 2, loads.

#### 8.3.8.2 Transverse Reinforcement in Frame Members

- (a) For determining the required transverse reinforcement in frame members, the quantity  $V_c$  shall be assumed to be zero if the factored axial compressive force including earthquake effects is less than  $0.05A_gf'_c$  when the earthquake-induced shear forces, calculated in accordance with 8.3.8.1(a), represents one-half or more of total design shear.
- (b) Stirrups or ties required to resist shear shall be closed hoops over lengths of members as specified in 8.3.4.3, 8.3.5.4 and 8.3.7.2.

#### 8.3.8.3 Shear Strength of Special Structural Walls and Diaphragms

- (a) Nominal shear strength of structural walls and diaphragms shall be determined using either (b) or (c) below.
- (b) Nominal shear strength,  $V_n$  of structural walls and diaphragms shall be assumed not to exceed the shear force calculated from

$$V_n = A_{cv}(0.17\lambda\sqrt{f'_c} + \rho_n f_y) \quad (8.3.6)$$

- (c) For walls and wall segments having a ratio of  $(h_w/l_w)$  less than 2.0, nominal shear strength of wall and diaphragm shall be determined from

$$V_n = A_{cv}(\alpha_c \lambda \sqrt{f'_c} + \rho_n f_y) \quad (8.3.7)$$

where the coefficient  $\alpha_c$  varies linearly from 0.25 for  $(h_w/l_w) = 1.5$  to 0.17 for  $(h_w/l_w) = 2.0$ . For  $(h_w/l_w) \leq 1.5$ ,  $\alpha_c = 0.25$ .

- (d) Value of ratio  $(h_w/l_w)$  used in (c) above for determining  $V_n$  for segments of a wall or diaphragm shall be the larger of the ratios for the entire wall (diaphragm) and the segment of wall (diaphragm) considered.
- (e) Walls and diaphragms shall have distributed shear reinforcement providing resistance in two orthogonal directions in the plane of the wall. If the ratio  $(h_w/l_w)$  does not exceed 2.0, reinforcement ratio,  $\rho_v$  shall not be less than reinforcement ratio  $\rho_n$ .
- (f) Nominal shear strength of all wall piers sharing a common lateral force shall not be assumed to exceed  $0.67A_{cv}\sqrt{f'_c}$ , where  $A_{cv}$  is the total cross-sectional area, and the nominal shear strength of any one of the individual wall piers shall not be assumed to exceed  $0.83A_{cp}\sqrt{f'_c}$  where  $A_{cp}$  represents the cross-sectional area of the pier considered.

- (g) Nominal shear strength of horizontal wall segments shall be assumed not to exceed  $0.83A_{cp}\sqrt{f'_c}$ , where  $A_{cp}$  represents the cross-sectional area of a horizontal wall segment.

### 8.3.9 Ordinary Moment Frame Members not Proportioned to Resist Forces Induced by Earthquake Motion

#### 8.3.9.1 Induced moments

Frame members assumed not to contribute to lateral resistance shall be detailed according to (a) or (b) below depending on the magnitude of moments induced in those members when subjected to twice the lateral displacement under the factored lateral forces.

- Members with factored gravity axial forces not exceeding  $0.1A_gf'_c$  shall satisfy 8.3.4.2(a) and 8.3.8.1(a) and members with factored gravity axial forces exceeding  $0.1A_gf'_c$  shall satisfy 8.3.5.4, 8.3.7.2(a) and 8.3.8.1(b) when the induced moment exceeds the design moment strength of the frame member.
- The member shall satisfy 8.3.4.2(a) when the induced moment does not exceed the design moment strength of the frame members.

#### 8.3.9.2 Tie requirements

All frame members with factored axial compressive forces exceeding  $0.1A_gf'_c$  shall satisfy the following special requirements unless they comply with 8.3.5.4.

- Ties shall have hooks not less than  $135^\circ$  with extensions not less than 6 tie bar diameter or 60 mm. Cross ties as defined in 8.3.2 are allowed.
- The maximum tie spacing shall be  $s_o$  over a length  $l_o$  measured from the joint face. The spacing  $s_o$  shall be not more than (i) eight diameters of the smallest longitudinal bar enclosed, (ii) 24 tie bar diameters, and (iii) one-half the least cross-sectional dimension of the column. The length  $l_o$  shall not be less than (i) one-sixth of the clear height of the column, (ii) the maximum cross-sectional dimension of the column, and (iii) 450 mm.
- The first tie shall be within a distance equal to  $0.5s_o$  from the face of the joint.
- The tie spacing shall not exceed  $2s_o$  in any part of the column.

### 8.3.10 Requirements for Intermediate Moment Frames

#### 8.3.10.1 Scope

For structures assigned to SDC C, structural frames proportioned to resist forces induced by earthquake motions shall satisfy the requirements of 8.3.10 in addition to those of Chapter 6.

#### 8.3.10.2 Reinforcement Requirements

Reinforcement details in a frame member shall satisfy 8.3.10.4 below if the factored compressive axial load for the member does not exceed  $0.1A_gf'_c$ . If the factored compressive axial load is larger, frame reinforcement details shall satisfy 8.3.10.5 below unless the member has spiral reinforcement according to Eq (6.3.6). If a two-way slab system without beams is treated as part of a frame resisting earthquake effect, reinforcement details in any span resisting moments caused by lateral force shall satisfy 8.3.10.6 below.

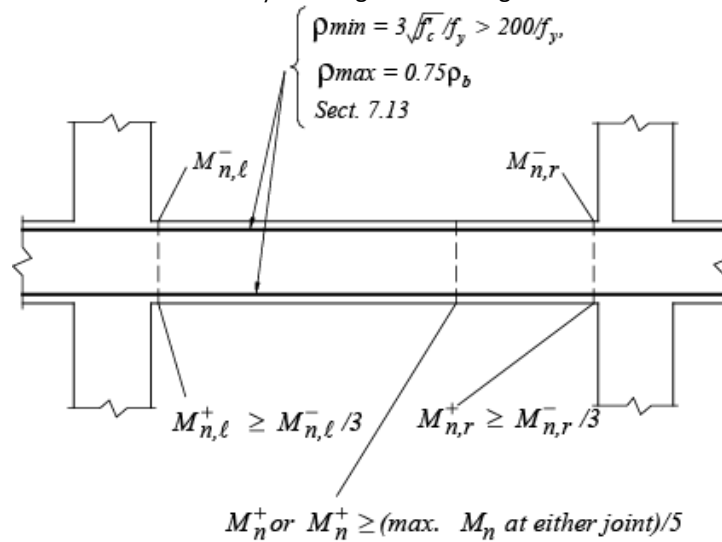
#### 8.3.10.3 Shear requirements

Design shear strength of beams, columns, and two-way slabs resisting earthquake effect shall not be less than either (a) the sum of the shear associated with development of nominal moment strengths of the member at each restrained end of the clear span and the shear calculated for factored gravity loads, or (b) the maximum shear obtained from design load combinations which include earthquake effect.

#### 8.3.10.4 Beams

- The positive moment strength at the face of the joint shall not be less than one-third the negative moment strength provided at that face (Fig.8.3.12). Neither the negative nor positive moment strength at any section along the length of the member shall be less than one-fifth of the maximum moment strength provided at the face of either joint.

- (b) At both ends of the member, stirrups shall be provided over lengths equal to twice the member depth measured from the face of the supporting member toward midspan (Fig.8.3.13). The first stirrup shall be located not more than 50 mm from the face of the supporting member. Maximum stirrup spacing shall not exceed (a)  $d/4$ , (b) 8 times the diameter of the smallest longitudinal bar enclosed, (c) 24 times the diameter of the stirrup bar, and (d) 300 mm.
- (c) Stirrups shall be placed at not more than  $d/2$  throughout the length of the member.



Note: transverse reinforcement not shown for clarity

Fig.8.3.12 Flexural Requirements for Beams

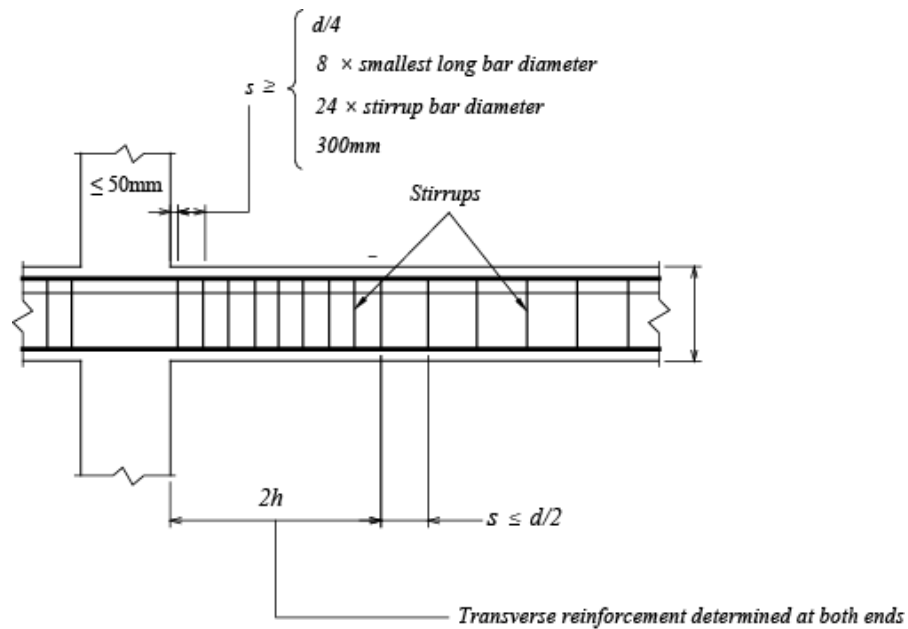


Fig.8.3.13 Transverse reinforcement Requirements for Beams

### 8.3.10.5 Columns

- (a) Maximum tie spacing shall not exceed  $s_o$  so over a length  $l_o$  measured from the joint face. The spacing  $s_o$  shall not exceed (i) 8 times the diameter of the smallest longitudinal bar enclosed, (ii) 24 times the diameter of the tie bar, (iii) one-half of the smallest cross-sectional dimension of the frame member, and (iv) 300 mm. The length  $l_o$  shall not be less than (i) one-sixth of the clear span of the member, (ii) maximum cross-sectional dimension of the member, and (iii) 450 mm.
- (b) The first tie shall be located not more than  $s_o/2$  from the joint face.
- (c) Joint reinforcement shall conform to 6.4.9.



(d) Tie spacing shall not exceed  $2s_o$  throughout the length of the member. These requirements are shown in Fig. 8.3.14.

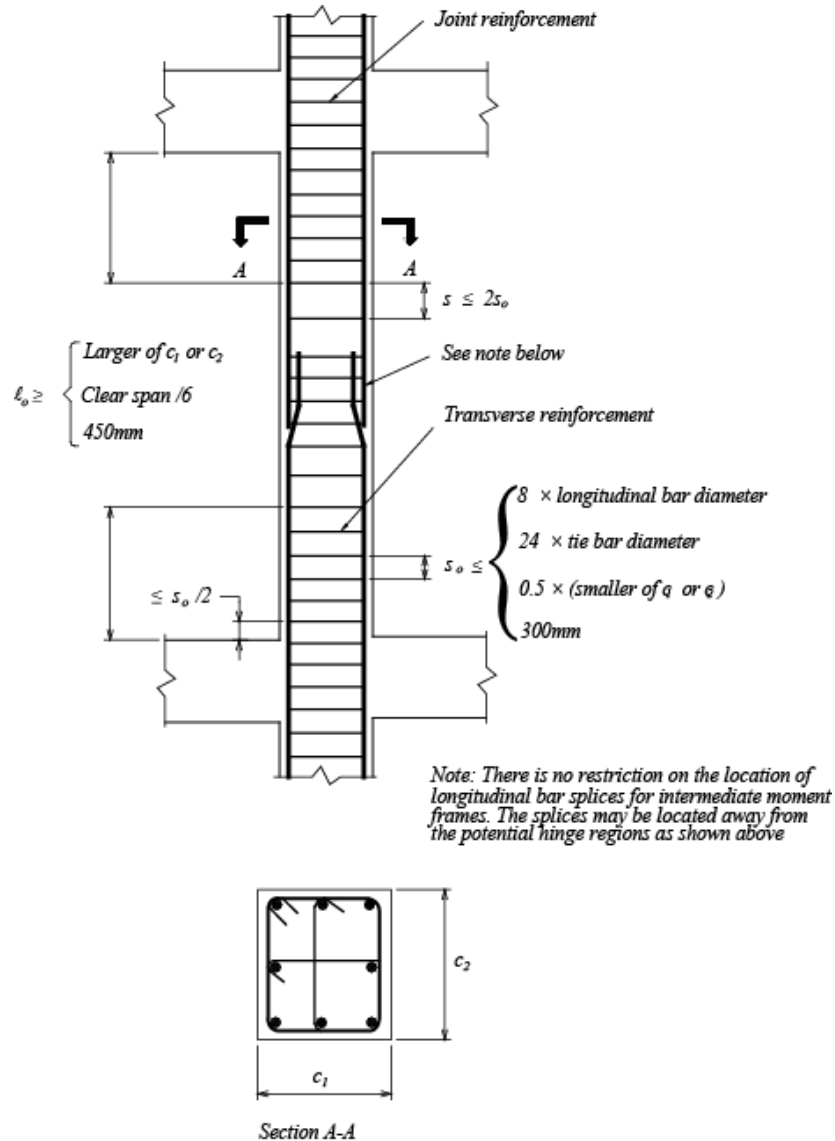
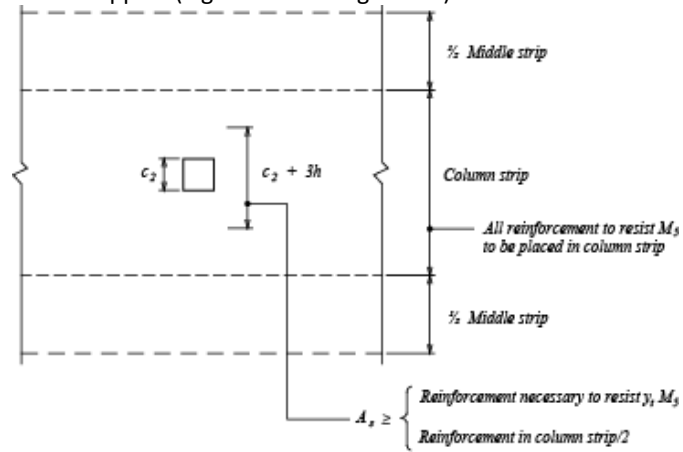


Fig.8.3.14 Transverse Reinforcement Requirements for Columns

#### 8.3.10.6 Two-way Slabs without Beams

- The factored slab moment at the supports relating to earthquake effect shall be determined for load combinations specified in Chapter 2, Loads. All reinforcement provided to resist the portion of slab moment balanced by support moment shall be placed within the column strip defined in 6.5.2.1 (Fig.8.3.15).
- The fractional part of the column strip moment shall be resisted by reinforcement placed within the effective width (Fig.8.3.15) specified in 6.5.5.3.2.
- Not less than one-half of the total reinforcement in the column strip at the support shall be placed within the effective slab width (Fig.8.3.15) specified in 6.5.5.3.2.
- Not less than one-quarter of the top steel at the support in the column strip shall be continuous throughout the span (Fig.8.3.16).
- Continuous bottom reinforcement in the column strip shall be not less than one-third of the top reinforcement at the support in the column strip.
- Not less than one-half of all bottom reinforcement at midspan shall be continuous and shall develop its yield strength at the face of support (Fig.8.3.17).

- (g) At discontinuous edges of the slab all top and bottom reinforcement at the support shall be developed at the face of the support (Fig.8.3.16 and Fig.8.3.17).



Note: applies to both top and bottom reinforcement

Fig.8.3.15 Reinforcement Details at Support of Two-way Slabs without beams

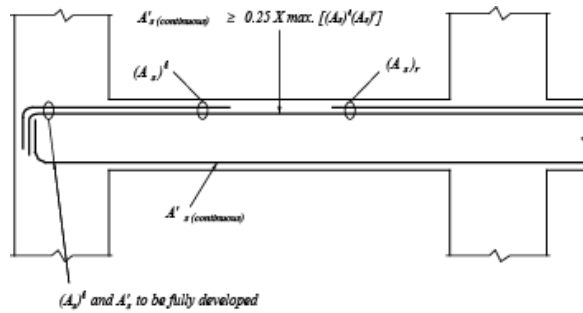


Fig.8.3.16 Reinforcement Details in Two-way Slabs without beams: Column Strip

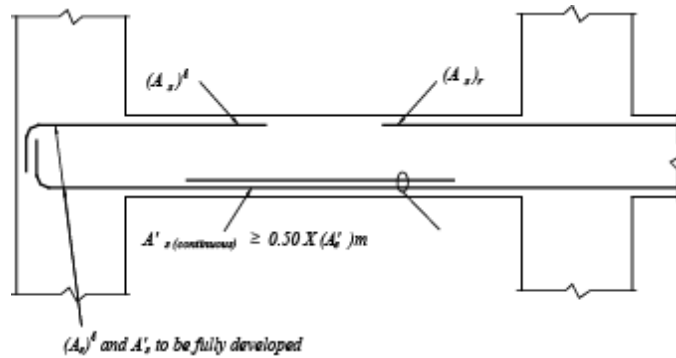


Fig.8.3.17 Reinforcement Details in Two-way Slabs without beams: Middle Strip