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मानक

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Mazdoor Kisan Shakti Sangathan

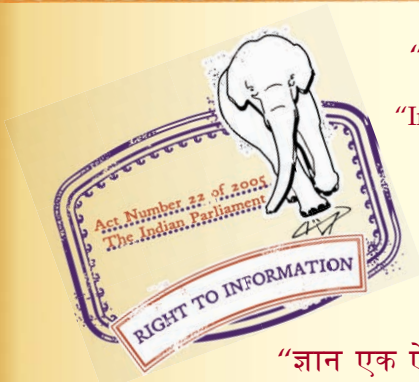
“The Right to Information, The Right to Live”

“पुराने को छोड़ नये के तरफ”

Jawaharlal Nehru

“Step Out From the Old to the New”

IS 13920 (1993, Reaffirmed 2008): Ductile Detailing Of Reinforced Concrete Structures Subjected To Seismic Forces
- Code Of Practice. UDC 69.059.25 (026) : 624.042.7



“ज्ञान से एक नये भारत का निर्माण”

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“ज्ञान एक ऐसा खजाना है जो कभी चुराया नहीं जा सकता है”

Bhartrhari—Nitiśatakam

“Knowledge is such a treasure which cannot be stolen”

(Reaffirmed 2008)

IS 13920 : 1993

(Reaffirmed 2003)

भारतीय मानक

भूकंपीय बल के प्रभाव के अंतर्गत प्रबलित कंकरीट संरचनाओं
का तन्य विस्तार — मार्गदर्शी सिद्धान्त

Indian Standard

DUCTILE DETAILING OF REINFORCED
CONCRETE STRUCTURES SUBJECTED TO
SEISMIC FORCES — CODE OF PRACTICE

(Third Reprint NOVEMBER 1996)

UDC 69-059-25 (026) : 624-042-7

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BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

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AMENDMENT NO. 2 MARCH 2002
TO
IS 13920 : 1993 DUCTILE DETAILING OF REINFORCED
CONCRETE STRUCTURES SUBJECTED TO SEISMIC
FORCES — CODE OF PRACTICE

(*Page 1, clause 1.1.1*) —Substitute the following for the existing:

‘1.1.1 Provisions of this code shall be adopted in all reinforced concrete structures which are located in seismic zone III, IV or V.’

(*Page 3, clause 5.2, line 3*) — Delete the word ‘preferably’.

(*Page 3, clause 5.3*) — Insert the following at the end of the clause:

‘However, high strength deformed steel bars, produced by the thermo-mechanical treatment process, of grades Fe 500 and Fe 550, having elongation more than 14.5 percent and conforming to other requirements of IS 1786 : 1985 may also be used for the reinforcement.’

(CED 39)

FOREWORD

This Indian Standard was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

IS 4326 : 1976 'Code of practice for earthquake resistant design and construction of buildings' while covering certain special features for the design and construction of earthquake resistant buildings included some details for achieving ductility in reinforced concrete buildings. With a view to keep abreast of the rapid developments and extensive research that has been carried out in the field of earthquake resistant design of reinforced concrete structures, the technical committee decided to cover provisions for the earthquake resistant design and detailing of reinforced concrete structures separately.

This code incorporates a number of important provisions hitherto not covered in IS 4326 : 1976. The major thrust in the formulation of this standard is one of the following lines:

- a) As a result of the experience gained from the performance, in recent earthquakes, of reinforced concrete structures that were designed and detailed as per IS 4326 : 1976, many deficiencies thus identified have been corrected in this code.
- b) Provisions on detailing of beams and columns have been revised with an aim of providing them with adequate toughness and ductility so as to make them capable of undergoing extensive inelastic deformations and dissipating seismic energy in a stable manner.
- c) Specifications on a seismic design and detailing of reinforced concrete shear walls have been included.

The other significant changes incorporated in this code are as follows:

- a) Material specifications are indicated for lateral force resisting elements of frames.
- b) Geometric constraints are imposed on the cross section for flexural members. Provisions on minimum and maximum reinforcement have been revised. The requirements for detailing of longitudinal reinforcement in beams at joint faces, splices, and anchorage requirements are made more explicit. Provision are also included for calculation of design shear force and for detailing of transverse reinforcement in beams.
- c) For members subjected to axial load and flexure, the dimensional constraints have been imposed on the cross section. Provisions are included for detailing of lap splices and for the calculation of design shear force. A comprehensive set of requirements is included on the provision of special confining reinforcement in those regions of a column that are expected to undergo cyclic inelastic deformations during a severe earthquake.
- d) Provisions have been included for estimating the shear strength and flexural strength of shear wall sections. Provisions are also given for detailing of reinforcement in the wall web, boundary elements, coupling beams, around openings, at construction joints, and for the development, splicing and anchorage of reinforcement.

Whilst the common methods of design and construction have been covered in this code, special systems of design and construction of any plain or reinforced concrete structure not covered by this code may be permitted on production of satisfactory evidence regarding their adequacy for seismic performance by analysis or tests or both.

The Sectional Committee responsible for the preparation of this standard has taken into consideration the view of manufacturers, users, engineers, architects, builders and technologists and has related the standard to the practices followed in the country in this field. Due weightage has also been given to the need for international co-ordination among standards prevailing in different seismic regions of the world.

In the formulation of this standard, assistance has been derived from the following publications:

- i) ACI 318-89/318R-89, Building code requirements for reinforced concrete and commentary, published by American Concrete Institute.
- ii) ATC-11. Seismic resistance of reinforced concrete shear walls and frame joints : Implications of recent research for design engineers, published by Applied Technology Council, USA.
- iii) CAN3-A23. 3-M84, 1984, Design of concrete structures for buildings, Canadian Standards Association.
- iv) SEADC, 1980, Recommended lateral force requirements and commentary, published by Structural Engineers Association of California, USA

The composition of the technical committees responsible for formulating this standard is given in Annex A.

Indian Standard

DUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES — CODE OF PRACTICE

1 SCOPE

1.1 This standard covers the requirements for designing and detailing of monolithic reinforced concrete buildings so as to give them adequate toughness and ductility to resist severe earthquake shocks without collapse.

1.1.1 Provisions of this code shall be adopted in all reinforced concrete structures which satisfy one of the following four conditions.

- a) The structure is located in seismic zone IV or V;
- b) The structure is located in seismic zone III and has the importance factor (I) greater than 1.0;
- c) The structure is located in seismic zone III and is an industrial structure; and
- d) The structure is located in seismic zone III and is more than 5 storey high.

NOTE — The definition of seismic zone and importance factor are given in IS 1893 : 1984.

1.1.2 The provisions for reinforced concrete construction given herein apply specifically to monolithic reinforced concrete construction. Precast and/or prestressed concrete members may be used only if they can provide the same level of ductility as that of a monolithic reinforced concrete construction during or after an earthquake.

2 REFERENCES

2.1 The Indian Standards listed below are necessary adjunct to this standard:

<i>IS No.</i>	<i>Title</i>
456 : 1978	Code of practice for plain and reinforced concrete (<i>third revision</i>)
1786 : 1985	Specification for high strength deformed steel bars and wires for concrete reinforcement (<i>third revision</i>)
1893 : 1984	Criteria for earthquake design of structures (<i>fourth revision</i>)

3 TERMINOLOGY

3.0 For the purpose of this standard, the following definitions shall apply.

3.1 Boundary Elements

Portions along the edges of a shear wall that are strengthened by longitudinal and transverse reinforcement. They may have the same thickness as that of the wall web.

3.2 Crosstie

Is a continuous bar having a 135° hook with a 10-diameter extension (but not < 75 mm) at each end. The hooks shall engage peripheral longitudinal bars.

3.3 Curvature Ductility

Is the ratio of curvature at the ultimate strength of the section to the curvature at first yield of tension steel in the section.

3.4 Hoop

Is a closed stirrup having a 135° hook with a 10-diameter extension (but not < 75 mm) at each end, that is embedded in the confined core of the section. It may also be made of two pieces of reinforcement; a U-stirrup with a 135° hook and a 10-diameter extension (but not < 75 mm) at each end, embedded in the confined core and a crosstie.

3.5 Lateral Force Resisting System

Is that part of the structural system which resists the forces induced by earthquake.

3.6 Shear Wall

A wall that is primarily designed to resist lateral forces in its own plane.

3.7 Shell Concrete

Concrete that is not confined by transverse reinforcement, is also called concrete cover.

3.8 Space Frame

A three dimensional structural system composed of interconnected members, without shear or bearing walls, so as to function as a complete

self-contained unit with or without the aid of horizontal diaphragms or floor bracing systems.

3.8.1 Vertical Load Carrying Space Frame

A space frame designed to carry all vertical loads.

3.8.2 Moment Resisting Space Frame

A vertical load carrying space frame in which the members and joints are capable of resisting forces primarily by flexure.

4 SYMBOLS

For the purpose of this standard, the following letter symbols shall have the meaning indicated against each; where other symbols are used, they are explained at the appropriate place. All dimensions are in mm, loads in Newton and stresses in MPa (N/sq mm) unless otherwise specified.

A_g	— gross cross sectional area of column, wall	M_u	— factored design moment on entire wall section
A_h	— horizontal reinforcement area within spacing S_v	$M_{u, \text{lim}}^{Ah}$	— hogging moment of resistance of beam at end A
A_k	— area of concrete core of column	$M_{u, \text{lim}}^{As}$	— sagging moment of resistance of beam at end A
A_{sd}	— reinforcement along each diagonal of coupling beam	$M_{u, \text{lim}}^{Bh}$	— hogging moment of resistance of beam at end B
A_{sh}	— area of cross section of bar forming spiral or hoop	$M_{u, \text{lim}}^{Bs}$	— sagging moment of resistance of beam at end B
A_{st}	— area of uniformly distributed vertical reinforcement	$M_{u, \text{lim}}^{bL}$	— moment of resistance of beam framing into column from the left
A_v	— vertical reinforcement at a joint	$M_{u, \text{lim}}^{bR}$	— moment of resistance of beam framing into column from the right
C_w	— centre to centre distance between boundary elements	M_{uv}	— flexural strength of wall web
D	— overall depth of beam	P_u	— factored axial load
D_k	— diameter of column core measured to the outside of spiral or hoop	S	— pitch of spiral or spacing hoops
d	— effective depth of member	S_v	— vertical spacing of horizontal reinforcement in web
d_w	— effective depth of wall section	t_w	— thickness of wall web
E_s	— elastic modulus of steel	V_a^{D+L}	— shear at end A of beam due to dead and live loads with a partial factor of safety of 1.2 on loads
f_{ck}	— characteristic compressive strength of concrete cube	V_b^{D+L}	— shear at end B of beam due to dead and live loads with a partial factor of safety of 1.2 on loads
f_y	— yield stress of steel	V_j	— shear resistance at a joint
h	— longer dimension of rectangular confining hoop measured to its outer face	V_u	— factored shear force
h_{st}	— storey height	V_{us}	— shear force to be resisted by reinforcement
L_{AB}	— clear span of beam	x_u, x_u^*	— depth of neutral axis from extreme compression fibre
l_o	— length of member over which special confining reinforcement is to be provided	α	— inclination of diagonal reinforcement in coupling beam
l_w	— horizontal length of wall	ρ	— vertical reinforcement ratio
l_s	— clear span of coupling beam	ρ_c	— compression reinforcement ratio in a beam
		ρ_{max}	— maximum tension reinforcement ratio for a beam
		ρ_{min}	— minimum tension reinforcement ratio for a beam
		τ_c	— shear strength of concrete
		$\tau_{c, max}$	— maximum permissible shear stress in section
		τ_v	— nominal shear stress

5 GENERAL SPECIFICATION

5.1 The design and construction of reinforced concrete buildings shall be governed by the provisions of IS 456 : 1978, except as modified by the provisions of this code.

5.2 For all buildings which are more than 3 storeys in height, the minimum grade of concrete shall preferably be M20 ($f_{ck} = 20$ MPa).

5.3 Steel reinforcements of grade Fe 415 (see IS 1786 : 1985) or less only shall be used.

6 FLEXURAL MEMBERS

6.1 General

These requirements apply to frame members resisting earthquake induced forces and designed to resist flexure. These members shall satisfy the following requirements.

6.1.1 The factored axial stress on the member under earthquake loading shall not exceed $0.1 f_{ck}$.

6.1.2 The member shall preferably have a width-to-depth ratio of more than 0.3.

6.1.3 The width of the member shall not be less than 200 mm.

6.1.4 The depth D of the member shall preferably be not more than $1/4$ of the clear span.

6.2 Longitudinal Reinforcement

6.2.1 a) The top as well as bottom reinforcement shall consist of at least two bars throughout the member length.

b) The tension steel ratio on any face, at any section, shall not be less than $\rho_{min} = 0.24 \sqrt{f_{ck}/f_y}$; where f_{ck} and f_y are in MPa.

6.2.2 The maximum steel ratio on any face at any section, shall not exceed $\rho_{max} = 0.025$.

6.2.3 The positive steel at a joint face must be at least equal to half the negative steel at that face.

6.2.4 The steel provided at each of the top and bottom face of the member at any section along its length shall be at least equal to one-fourth of the maximum negative moment steel provided at the face of either joint. It may be clarified that redistribution of moments permitted in IS 456 : 1978 (clause 36.1) will be used only for vertical load moments and not for lateral load moments.

6.2.5 In an external joint, both the top and the bottom bars of the beam shall be provided with anchorage length, beyond the inner face of the column, equal to the development length in tension plus 10 times the bar diameter minus the allowance for 90 degree bend(s) (see Fig. 1). In an internal joint, both face bars of the beam shall be taken continuously through the column.

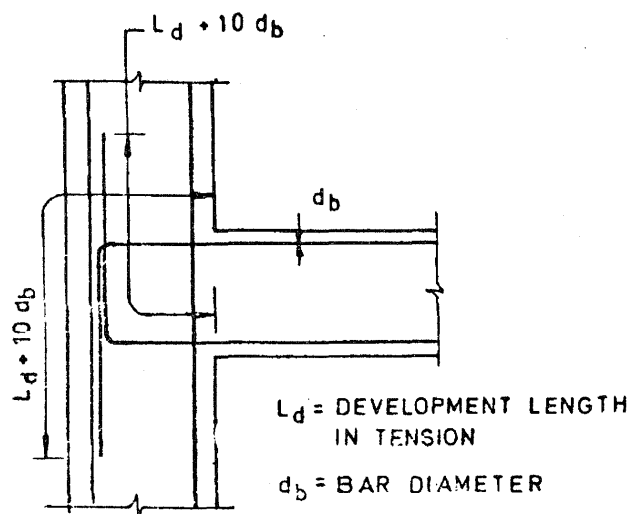
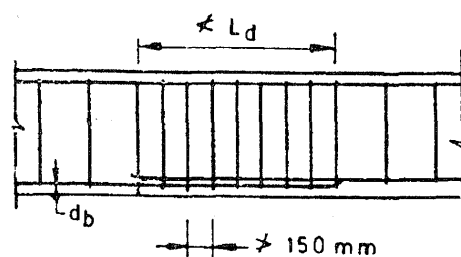


FIG. 1 ANCHORAGE OF BEAM BARS IN AN EXTERNAL JOINT

6.2.6 The longitudinal bars shall be spliced, only if hoops are provided over the entire splice length, at a spacing not exceeding 150 mm (see Fig. 2). The lap length shall not be less than the bar development length in tension. Lap splices shall not be provided (a) within a joint, (b) within a distance of $2d$ from joint face, and (c) within a quarter length of the member where flexural yielding may generally occur under the effect of earthquake forces. Not more than 50 percent of the bars shall be spliced at one section.



$L_d = \text{DEVELOPMENT LENGTH IN TENSION}$

$d_b = \text{BAR DIAMETER}$

FIG. 2 LAP, SPLICE IN BEAM

6.2.7 Use of welded splices and mechanical connections may also be made, as per 25.2.5.2 of IS 456 : 1978. However, not more than half the reinforcement shall be spliced at a section where flexural yielding may take place. The location of splices shall be governed by 6.2.6.

6.3 Web Reinforcement

6.3.1 Web reinforcement shall consist of vertical hoops. A vertical hoop is a closed stirrup having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end that is embedded in the confined core (see Fig. 3a). In compelling circumstances, it may also be made up of two pieces of reinforcement; a U-stirrup with a 135° hook and a 10 diameter extension (but not < 75 mm) at each end, embedded in the confined core and a crosstie (see Fig. 3b). A crosstie is a bar having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end. The hooks shall engage peripheral longitudinal bars.

6.3.2 The minimum diameter of the bar forming a hoop shall be 6 mm. However, in beams with clear span exceeding 5 m, the minimum bar diameter shall be 8 mm.

6.3.3 The shear force to be resisted by the vertical hoops shall be the maximum of :

- calculated factored shear force as per analysis, and
- shear force due to formation of plastic hinges at both ends of the beam plus the factored gravity load on the span. This is given by (see Fig. 4):

i) for sway to right:

$$V_{u,a} = V_a^{D+L} - 1.4 \left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}} \right]$$

$$\text{and } V_{u,b} = V_b^{D+L} + 1.4 \left[\frac{M_{u,lim}^{As} + M_{u,lim}^{Bh}}{L_{AB}} \right], \text{ and}$$

ii) for sway to left:

$$V_{u,a} = V_a^{D+L} + 1.4 \left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}} \right]$$

$$\text{and } V_{u,b} = V_b^{D+L} - 1.4 \left[\frac{M_{u,lim}^{Ah} + M_{u,lim}^{Bs}}{L_{AB}} \right],$$

where $M_{u,lim}^{As}$, $M_{u,lim}^{Ah}$ and $M_{u,lim}^{Bs}$, $M_{u,lim}^{Bh}$ are the sagging and hogging moments of resistance of the beam section at ends A and B, respectively. These are to be calculated as per IS 456 : 1978. L_{AB} is clear span of beam. V_a^{D+L} and V_b^{D+L} are the shears at ends A and B, respectively, due to vertical loads with a partial safety factor of 1.2 on loads. The design shear at end A shall be the larger of the two values of $V_{u,a}$ computed above. Similarly, the design shear at end B shall be the larger of the two values of $V_{u,b}$ computed above.

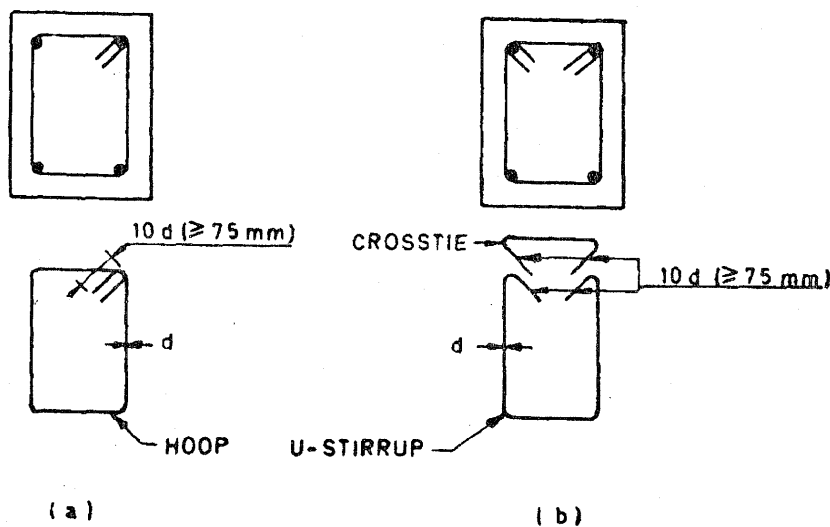


FIG. 3 BEAM WEB REINFORCEMENT

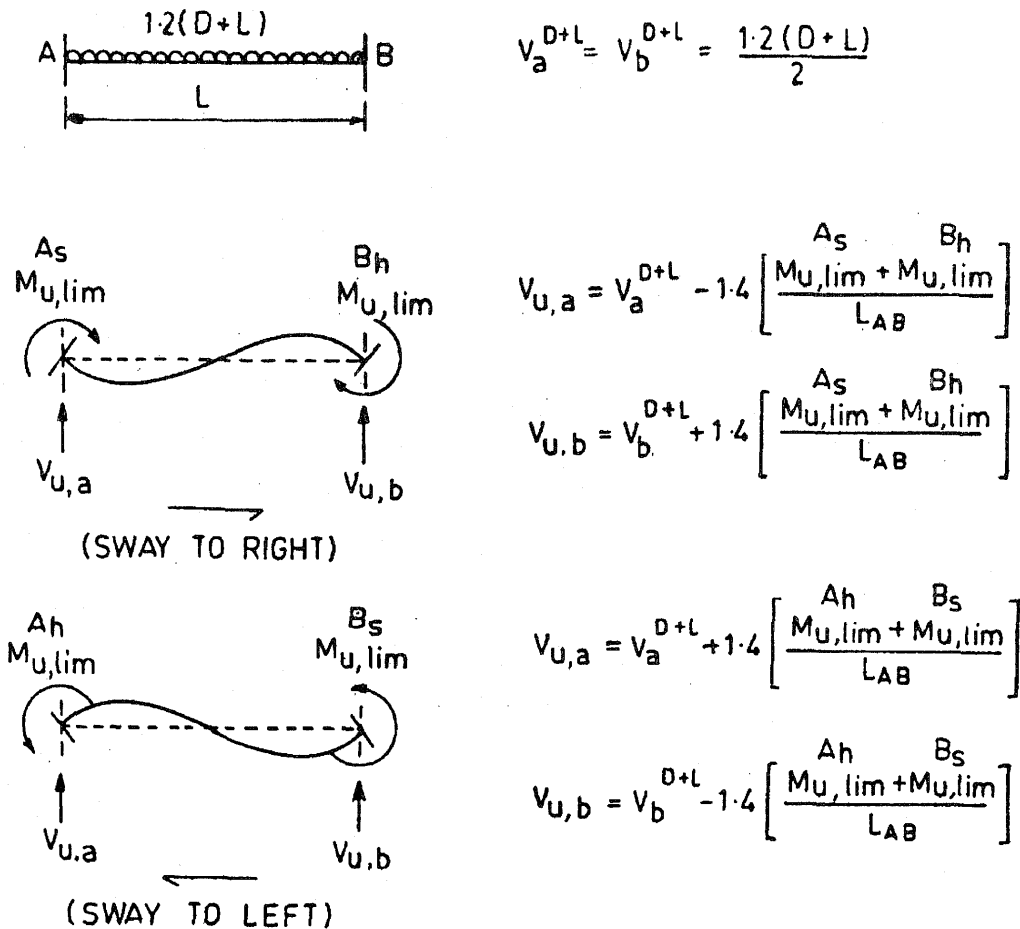


FIG. 4 CALCULATION OF DESIGN SHEAR FORCE FOR BEAM

6.3.4 The contribution of bent up bars and inclined hoops to shear resistance of the section shall not be considered.

6.3.5 The spacing of hoops over a length of $2d$ at either end of a beam shall not exceed (a) $d/4$, and (b) 8 times the diameter of the smallest longitudinal bar; however, it need not be less than 100 mm (see Fig. 5). The first hoop shall be at a distance not exceeding 50 mm from the joint face. Vertical hoops at the same spacing as above, shall also be provided over a length equal to $2d$ on either side of a section where flexural yielding may occur under the effect of earthquake forces. Elsewhere, the beam shall have vertical hoops at a spacing not exceeding $d/2$.

7 COLUMNS AND FRAME MEMBERS SUBJECTED TO BENDING AND AXIAL LOAD

7.1 General

7.1.1 These requirements apply to frame members which have a factored axial stress in excess of $0.1 f_{ck}$ under the effect of earthquake forces.

7.1.2 The minimum dimension of the member shall not be less than 200 mm. However, in frames which have beams with centre to centre span exceeding 5 m or columns of unsupported length exceeding 4 m, the shortest dimension of the column shall not be less than 300 mm.

7.1.3 The ratio of the shortest cross sectional dimension to the perpendicular dimension shall preferably not be less than 0.4.

7.2 Longitudinal Reinforcement

7.2.1 Lap splices shall be provided only in the central half of the member length. It should be proportioned as a tension splice. Hoops shall be provided over the entire splice length at spacing not exceeding 150 mm centre to centre. Not more than 50 percent of the bars shall be spliced at one section.

7.2.2 Any area of a column that extends more than 100 mm beyond the confined core due to architectural requirements, shall be detailed in the following manner. In case the contribution of this area to strength has been considered, then it will have the minimum longitudinal and transverse reinforcement as per this code.

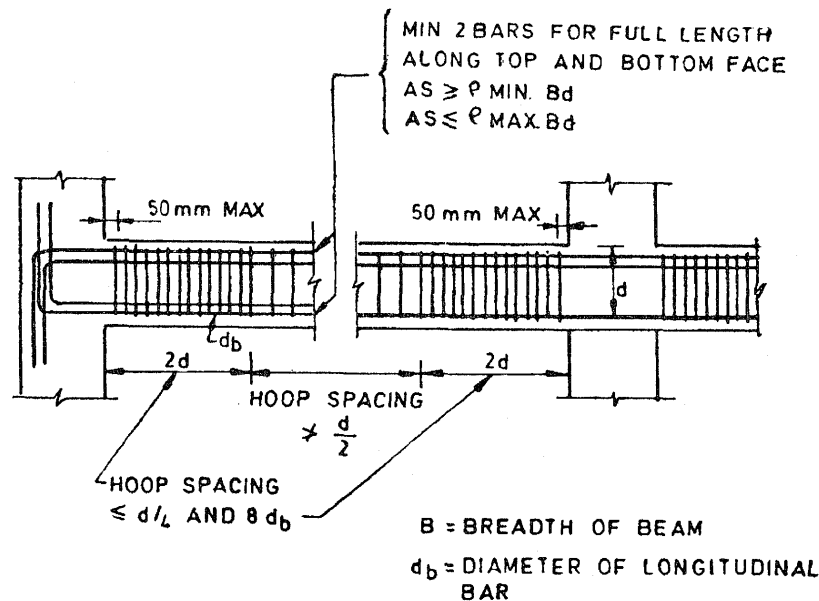


FIG. 5 BEAM REINFORCEMENT

However, if this area has been treated as non-structural, the minimum reinforcement requirements shall be governed by IS 456 : 1978 provisions minimum longitudinal and transverse reinforcement, as per IS 456 : 1978 (see Fig. 6).

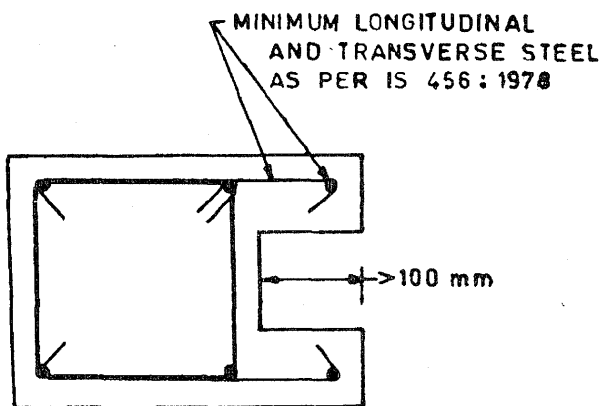


FIG. 6 REINFORCEMENT REQUIREMENT FOR COLUMN WITH MORE THAN 100 mm PROJECTION BEYOND CORE

7.3 Transverse Reinforcement

7.3.1 Transverse reinforcement for circular columns shall consist of spiral or circular hoops. In rectangular columns, rectangular hoops may be used. A rectangular hoop is a closed stirrup, having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end, that is embedded in the confined core (see Fig 7A).

7.3.2 The parallel legs of rectangular hoops shall be spaced not more than 300 mm centre to centre. If the length of any side of the hoop exceeds 300 mm, a crosstie shall be provided (Fig. 7B). Alternatively, a pair of overlapping hoops may be provided within the column (see Fig. 7C). The hooks shall engage peripheral longitudinal bars.

7.3.3 The spacing of hoops shall not exceed half the least lateral dimension of the column, except where special confining reinforcement is provided, as per 7.4.

7.3.4 The design shear force for columns shall be the maximum of:

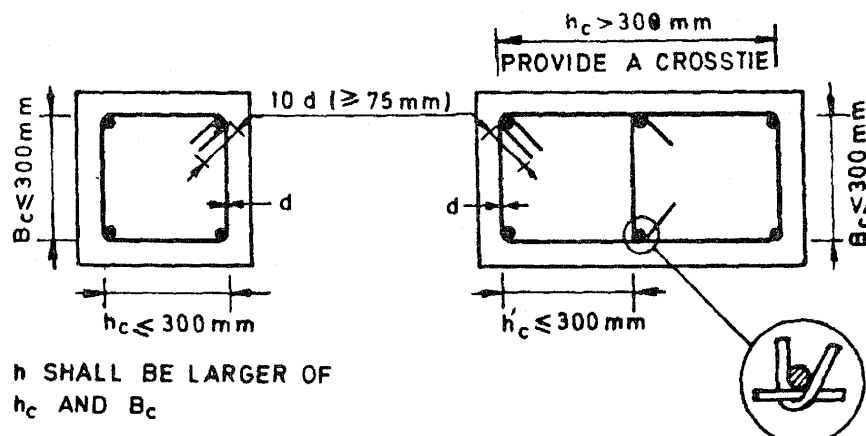
- calculated factored shear force as per analysis, and
- a factored shear force given by

$$V_u = 1.4 \left[\frac{M_{u,lim}^{bL} + M_{u,lim}^{bR}}{h_{at}} \right]$$

where $M_{u,lim}^{bL}$ and $M_{u,lim}^{bR}$ are moment of resistance, of opposite sign, of beams framing into the column from opposite faces (see Fig. 8); and h_{at} is the storey height. The beam moment capacity is to be calculated as per IS 456 : 1978.

7.4 Special Confining Reinforcement

This requirement shall be met with, unless a larger amount of transverse reinforcement is required from shear strength considerations.

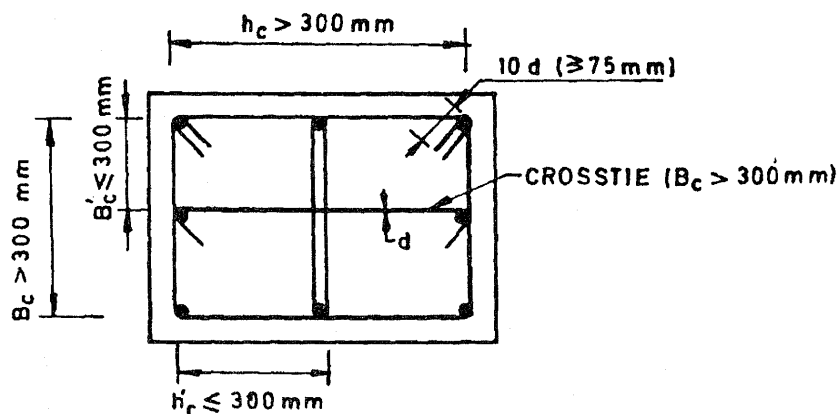


h SHALL BE LARGER OF
 h_c AND B_c

h SHALL BE LARGER OF h_c AND B_c

7A SINGLE HOOP

7B SINGLE HOOP WITH A
CROSSTIE



h SHALL BE LARGER OF h'_c AND B_c

7C OVERLAPPING HOOPS WITH A CROSSTIE

FIG. 7 TRANSVERSE REINFORCEMENT IN COLUMN

7.4.1 Special confining reinforcement shall be provided over a length l_o from each joint face, towards midspan, and on either side of any section, where flexural yielding may occur under the effect of earthquake forces (see Fig. 9.). The length ' l_o ' shall not be less than (a) larger lateral dimension of the member at the section where yielding occurs, (b) $1/6$ of clear span of the member, and (c) 450 mm.

7.4.2 When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat (see Fig. 10).

7.4.3 When the calculated point of contraflexure, under the effect of gravity and earthquake loads, is not within the middle half of the member clear height, special confining reinforcement shall be provided over the full height of the column.

7.4.4 Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height (see Fig. 11). This reinforcement shall also be placed above the discontinuity for at least the development length of the largest longitudinal bar in the column. Where the column is supported on a wall, this reinforcement shall be provided over the full height of the column; it shall also be provided below the discontinuity for the same development length.

7.4.5 Special confining reinforcement shall be provided over the full height of a column which has significant variation in stiffness along its height. This variation in stiffness may result

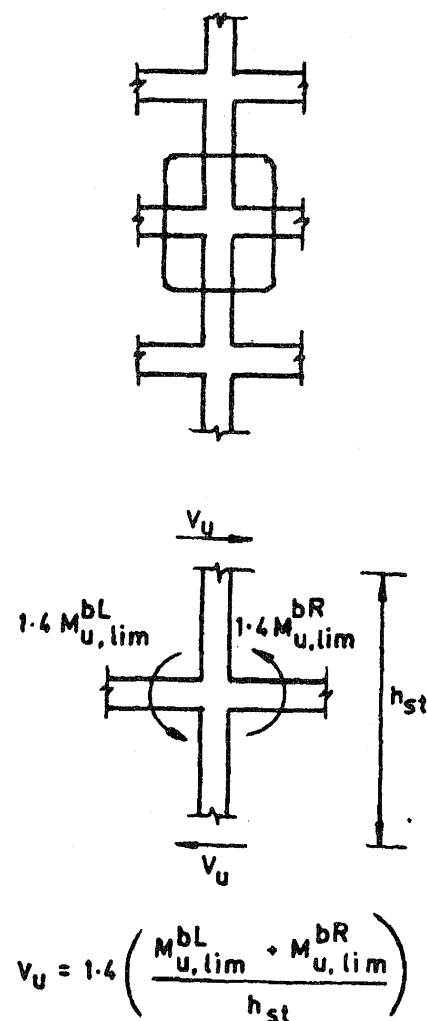


FIG. 8 CALCULATION OF DESIGN SHEAR FORCE FOR COLUMN

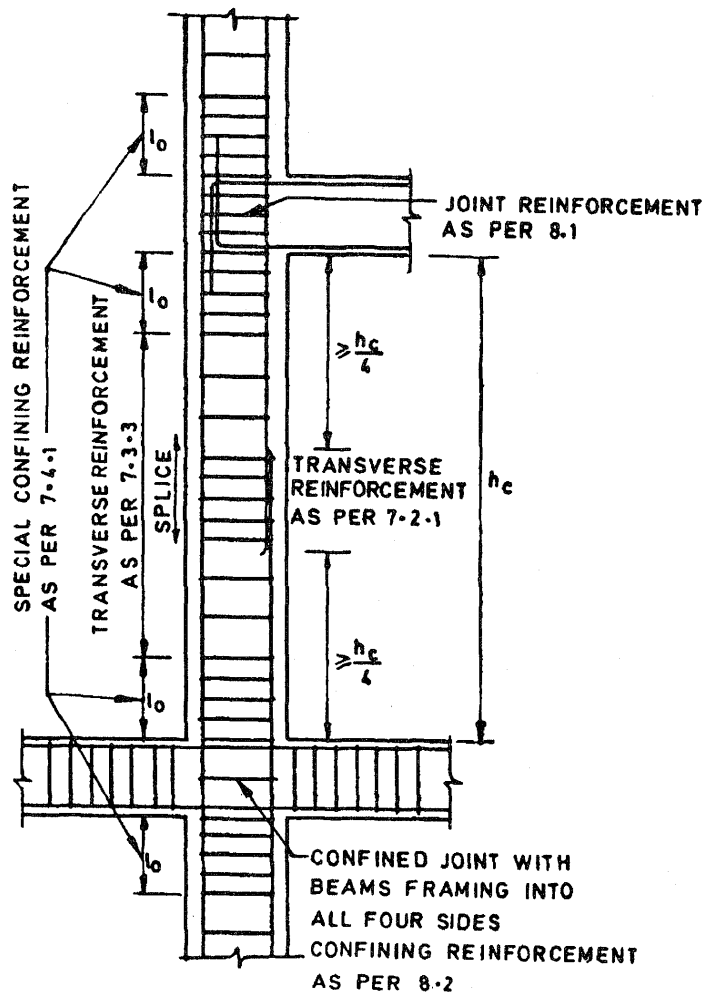


FIG. 9 COLUMN AND JOINT DETAILING

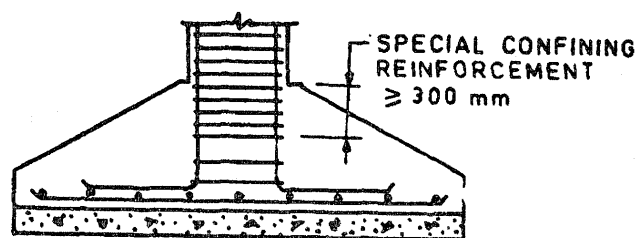


FIG. 10 PROVISION OF SPECIAL CONFINING REINFORCEMENT IN FOOTINGS

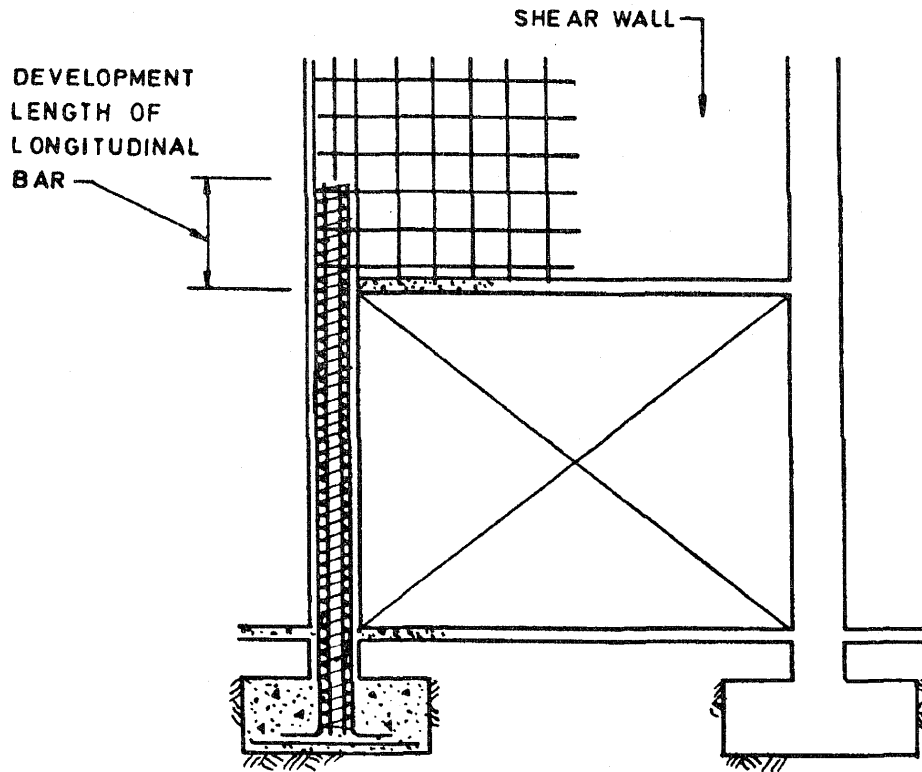


FIG. 11 SPECIAL CONFINING REINFORCEMENT REQUIREMENT FOR COLUMNS UNDER DISCONTINUED WALLS

due to the presence of bracing, a mezzanine floor or a R.C.C. wall on either side of the column that extends only over a part of the column height (see Fig. 12).

7.4.6 The spacing of hoops used as special confining reinforcement shall not exceed $1/4$ of minimum member dimension but need not be less than 75 mm nor more than 100 mm.

7.4.7 The area of cross section, A_{sh} , of the bar forming circular hoops or spiral, to be used as special confining reinforcement, shall not be less than

$$A_{sh} = 0.09 SD_k \frac{f_{ck}}{f_y} \left[\frac{A_g}{A_k} - 1.0 \right]$$

where

- A_{sh} = area of the bar cross section,
- S = pitch of spiral or spacing of hoops,
- D_k = diameter of core measured to the outside of the spiral or hoop,
- f_{ck} = characteristic compressive strength of concrete cube,
- f_y = yield stress of steel (of circular hoop or spiral),
- A_g = gross area of the column cross section, and

$$A_k = \text{area of the concrete core} = \frac{\pi}{4} D_k^2$$

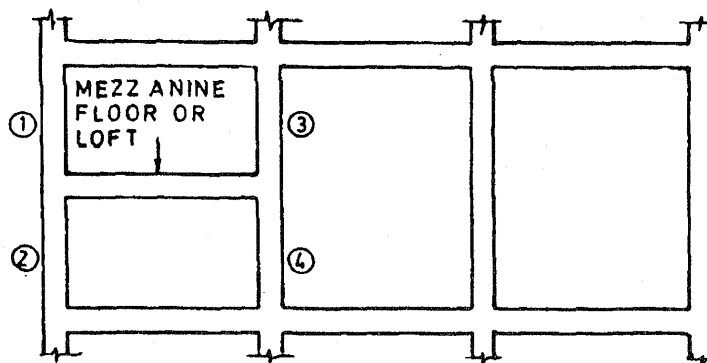
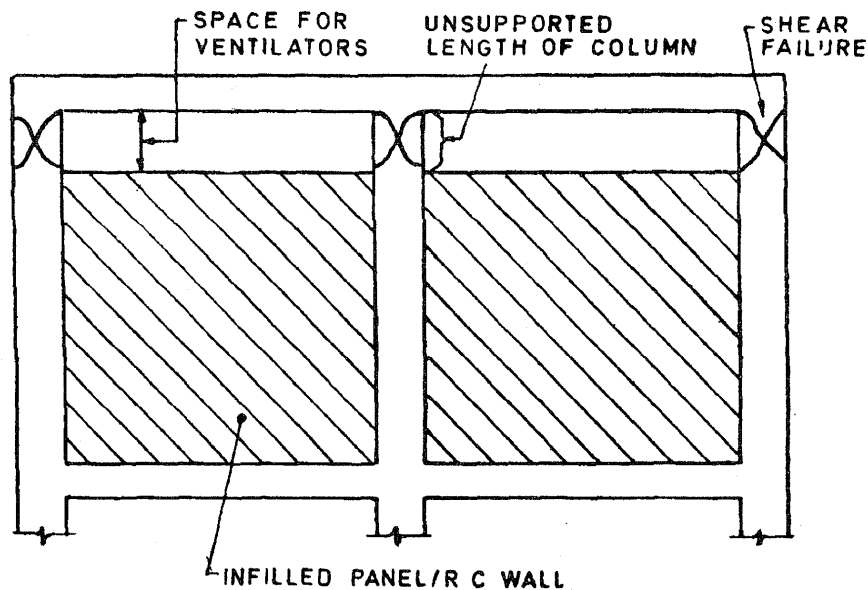
Example : Consider a column of diameter 300 mm. Let the grade of concrete be M20, and that of steel Fe 415, for longitudinal and confining reinforcement. The spacing of circular hoops, S , shall not exceed the smaller of (a) $1/4$ of minimum member dimension = $1/4 \times 300 = 75$ mm, and (b) 100 mm. Therefore, $S = 75$ mm. Assuming 40 mm clear cover to the longitudinal reinforcement and circular hoops of diameter 8 mm, $D_k = 300 - 2 \times 40 + 2 \times 8 = 236$ mm. Thus, the area of cross section of the bar forming circular hoop works out to be 47.28 mm^2 . This is less than the cross sectional area of 8 mm bar (50.27 mm^2). Thus, circular hoops of diameter 8 mm at a spacing of 75 mm centre to centre will be adequate.

7.4.8 The area of cross section, A_{sh} , of the bar forming rectangular hoop, to be used as special confining reinforcement shall not be less than

$$A_{sh} = 0.18 Sh \frac{f_{ck}}{f_y} \left[\frac{A_g}{A_k} - 1.0 \right]$$

where

- h = longer dimension of the rectangular confining hoop measured to its outer



(1), (2), (3) and (4) relatively stiff columns — They attract large seismic shear force.

FIG. 12 COLUMNS WITH VARYING STIFFNESS

face. It shall not exceed 300 mm (see Fig. 7), and

A_k = area of confined concrete core in the rectangular hoop measured to its outside dimensions.

NOTE: The dimension 'h' of the hoop could be reduced by introducing cross-ties, as shown in Fig. 7B. In this case, A_k shall be measured as the overall core area, regardless of the hoop arrangement. The hooks of cross-ties shall engage peripheral longitudinal bars.

Example: Consider a column of 650 mm × 500 mm. Let the grade of concrete be M20 and that of steel Fe 415, for the longitudinal and confining reinforcement. Assuming clear cover of 40 mm to the longitudinal reinforcement and rectangular hoops of diameter 10 mm, the size of the core is 590 mm × 440 mm. As both these dimensions are greater than 300 mm,

either a pair of overlapping hoops or a single hoop with cross-ties, in both directions, will have to be provided. Thus, the dimension 'h' will be the larger of (i) $590/2 = 295$ mm, and (ii) $440/2 = 220$ mm. The spacing of hoops, S , shall not exceed the smaller of (a) $1/4$ of minimum member dimensions = $1/4 \times 500 = 125$ mm, and (b) 100 mm. Thus, $S = 100$ mm. The area of cross section of the bar forming rectangular hoop works out to be 64.47 mm^2 . This is less than the area of cross section of 10 mm bar (78.54 mm^2). Thus, 10 mm diameter rectangular hoops at 100 mm c/c will be adequate. Similar calculations indicate that, as an alternative, one could also provide 8 mm diameter rectangular hoops at 70 mm c/c.

8 JOINTS OF FRAMES

8.1 The special confining reinforcement as required at the end of column shall be provided

through the joint as well, unless the joint is confined as specified by 8.2.

8.2 A joint which has beams framing into all vertical faces of it and where each beam width is at least 3/4 of the column width, may be provided with half the special confining reinforcement required at the end of the column. The spacing of hoops shall not exceed 150 mm.

9 SHEAR WALLS

9.1 General Requirements

9.1.1 The requirements of this section apply to the shear walls, which are part of the lateral force resisting system of the structure.

9.1.2 The thickness of any part of the wall shall preferably, not be less than 150 mm.

9.1.3 The effective flange width, to be used in the design of flanged wall sections, shall be assumed to extend beyond the face of the web for a distance which shall be the smaller of (a) half the distance to an adjacent shear wall web, and (b) 1/10 th of the total wall height.

9.1.4 Shear walls shall be provided with reinforcement in the longitudinal and transverse directions in the plane of the wall. The minimum reinforcement ratio shall be 0.002 5 of the gross area in each direction. This reinforcement shall be distributed uniformly across the cross section of the wall.

9.1.5 If the factored shear stress in the wall exceeds $0.25 \sqrt{f_{ck}}$ or if the wall thickness exceeds 200 mm, reinforcement shall be provided in two curtains, each having bars running in the longitudinal and transverse directions in the plane of the wall.

9.1.6 The diameter of the bars to be used in any part of the wall shall not exceed 1/10th of the thickness of that part.

9.1.7 The maximum spacing of reinforcement in either direction shall not exceed the smaller of $l_w/5$, $3 t_w$, and 450 mm; where l_w is the horizontal length of the wall, and t_w is the thickness of the wall web.

9.2 Shear Strength

9.2.1 The nominal shear stress, τ_v , shall be calculated as:

$$\tau_v = \frac{V_u}{t_w d_w}$$

where

V_u = factored shear force,

t_w = thickness of the web, and

d_w = effective depth of wall section. This may be taken as $0.8 l_w$ for rectangular sections.

9.2.2 The design shear strength of concrete, τ_c , shall be calculated as per Table 13 of IS 456 : 1978.

9.2.3 The nominal shear stress in the wall, τ_v , shall not exceed $\tau_{c, \max}$, as per Table 14 of IS 456 : 1978.

9.2.4 When τ_v is less than τ_c shear reinforcement shall be provided in accordance with 9.1.4, 9.1.5 and 9.1.7.

9.2.5 When τ_v is greater than τ_c , the area of horizontal shear reinforcement, A_h , to be provided within a vertical spacing, S_v , is given by

$$V_{us} = \frac{0.87 f_y A_h d_w}{S_v}$$

where $V_{us} = (V_u - \tau_c t_w d_w)$, is the shear force to be resisted by the horizontal reinforcement. However, the amount of horizontal reinforcement provided shall not be less than the minimum, as per 9.1.4.

9.2.6 The vertical reinforcement, that is uniformly distributed in the wall, shall not be less than the horizontal reinforcement calculated as per 9.2.5.

9.3 Flexural Strength

9.3.1 The moment of resistance, M_{uw} , of the wall section may be calculated as for columns subjected to combined bending and axial load as per IS 456 : 1978. The moment of resistance of slender rectangular shear wall section with uniformly distributed vertical reinforcement is given in Annex A.

9.3.2 The cracked flexural strength of the wall section should be greater than its uncracked flexural strength.

9.3.3 In walls that do not have boundary elements, vertical reinforcement shall be concentrated at the ends of the wall. Each concentration shall consist of a minimum of 4 bars of 12 mm diameter arranged in at least 2 layers.

9.4 Boundary Elements

Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement. Though they may have the same thickness as that of the wall web it is advantageous to provide them with greater thickness.

9.4.1 Where the extreme fibre compressive stress in the wall due to factored gravity loads plus factored earthquake force exceeds $0.2 f_{ck}$, boundary elements shall be provided along the vertical boundaries of walls. The boundary

elements may be discontinued where the calculated compressive stress becomes less than $0.15f_{ck}$. The compressive stress shall be calculated using a linearly elastic model and gross section properties.

9.4.2 A boundary element shall have adequate axial load carrying capacity, assuming short column action, so as to enable it to carry an axial compression equal to the sum of factored gravity load on it and the additional compressive load induced by the seismic force. The latter may be calculated as:

$$\frac{M_u - M_{uv}}{C_w}$$

where

M_u = factored design moment on the entire wall section,

M_{uv} = moment of resistance provided by distributed vertical reinforcement across the wall section, and

C_w = center to center distance between the boundary elements along the two vertical edges of the wall.

9.4.3 If the gravity load adds to the strength of the wall, its load factor shall be taken as 0.8.

9.4.4 The percentage of vertical reinforcement in the boundary elements shall not be less than 0.8 percent, nor greater than 6 percent. In order to avoid congestion, the practical upper limit would be 4 percent.

9.4.5 Boundary elements, where required, as per 9.4.1, shall be provided throughout their height, with special confining reinforcement, as per 7.4.

9.4.6 Boundary elements need not be provided, if the entire wall section is provided with special confining reinforcement, as per 7.4.

9.5 Coupled Shear Walls

9.5.1 Coupled shear walls shall be connected by ductile coupling beams. If the earthquake induced shear stress in the coupling beam exceeds

$$\frac{0.1 l_s \sqrt{f_{ck}}}{D}$$

where l_s is the clear span of the coupling beam and D is its overall depth, the entire earthquake induced shear and flexure shall, preferably, be resisted by diagonal reinforcement.

9.5.2 The area of reinforcement to be provided along each diagonal in a diagonally reinforced coupling beam shall be:

$$A_{sd} = \frac{V_u}{1.74 f_y \sin \alpha}$$

where V_u is the factored shear force, and α is the angle made by the diagonal reinforcement with the horizontal. At least 4 bars of 8 mm diameter shall be provided along each diagonal. The reinforcement along each diagonal shall be enclosed by special confining reinforcement, as per 7.4. The pitch of spiral or spacing of ties shall not exceed 100 mm.

9.5.3 The diagonal or horizontal bars of a coupling beam shall be anchored in the adjacent walls with an anchorage length of 1.5 times the development length in tension.

9.6 Openings in Walls

9.6.1 The shear strength of a wall with openings should be checked along critical planes that pass through openings.

9.6.2 Reinforcement shall be provided along the edges of openings in walls. The area of the vertical and horizontal bars should be such as to equal that of the respective interrupted bars. The vertical bars should extend for the full storey height. The horizontal bars should be provided with development length in tension beyond the sides of the opening.

9.7 Discontinuous Walls

Columns supporting discontinuous walls shall be provided with special confining reinforcement, as per 7.4.4.

9.8 Construction Joints

The vertical reinforcement ratio across a horizontal construction joint shall not be less than:

$$\frac{0.92}{f_y} \left(\tau_v - \frac{P_u}{A_g} \right)$$

where τ_v is the factored shear stress at the joint, P_u is the factored axial force (positive for compression), and A_g is the gross cross sectional area of the joint.

9.9 Development, Splice and Anchorage Requirement

9.9.1 Horizontal reinforcement shall be anchored near the edges of the wall or in the confined core of the boundary elements.

9.9.2 Splicing of vertical flexural reinforcement should be avoided, as far as possible, in regions where yielding may take place. This zone of flexural yielding may be considered to extend for a distance of l_w above the base of the wall or one sixth of the wall height, whichever is more. However, this distance need not be greater than $2l_w$. Not more than one third of this vertical reinforcement shall be spliced at such a section. Splices in adjacent bars should be staggered by a minimum of 600 mm.

9.9.3 Lateral ties shall be provided around lapped spliced bars that are larger than 16 mm in diameter. The diameter of the tie shall not be less than one fourth that of the spliced bar nor less than 6 mm. The spacing of ties shall not exceed 150 mm center to center.

9.9.4 Welded splices and mechanical connections shall conform to 25.2.5.2 of IS 456 : 1978. However, not more than half the reinforcement shall be spliced at a section, where flexural yielding may take place.

ANNEX A

(Clause 9.3.1)

MOMENT OF RESISTANCE OF RECTANGULAR SHEAR WALL SECTION

A-1 The moment of resistance of a slender rectangular shear wall section with uniformly distributed vertical reinforcement may be estimated as follows:

(a) For $x_u/l_w \leq x_u^*/l_w$,

$$\frac{M_{uv}}{f_{ck} t_w l_w^2} = \phi \left[\left(1 + \frac{\lambda}{\phi} \right) \left(\frac{1}{2} - 0.416 \frac{x_u}{l_w} \right) - \left(\frac{x_u}{l_w} \right)^2 \left(0.168 + \frac{\beta^2}{3} \right) \right]$$

where

$$\frac{x_u}{l_w} = \left(\frac{\phi + \lambda}{2\phi + 0.36} \right); \quad \frac{x_u^*}{l_w} = \left(\frac{0.0035}{0.0035 + 0.87 f_y/E_s} \right);$$

$$\phi = \left(\frac{0.87 f_y \rho}{f_{ck}} \right); \quad \lambda = \left(\frac{P_u}{f_{ck} t_w l_w} \right);$$

ρ = vertical reinforcement ratio = $A_{st}/(t_w l_w)$,

A_{st} = area of uniformly distributed vertical reinforcement,

$\beta = 0.87 f_y/(0.0035 E_s)$,

E_s = elastic modulus of steel, and

P_u = axial compression on wall.

(b) For $x_u^*/l_w < x_u/l_w < 1.0$,

$$\frac{M_{uv}}{f_{ck} t_w l_w^2} = \alpha_1 \left(\frac{x_u}{l_w} \right) - \alpha_2 \left(\frac{x_u}{l_w} \right)^2 - \alpha_3 - \frac{\lambda}{2}$$

where

$$\alpha_1 = \left[0.36 + \phi \left(1 - \frac{\beta}{2} - \frac{1}{2\beta} \right) \right]$$

$$\alpha_2 = \left[0.15 + \frac{\phi}{2} \left(1 - \beta - \frac{\beta^2}{2} - \frac{1}{3\beta} \right) \right]; \text{ and } \alpha_3 = \frac{\phi}{6\beta} \left(\frac{1}{(x_u/l_w)} - 3 \right)$$

The value of x_u/l_w to be used in this equation, should be calculated from the quadratic equation

$$\alpha_1 \left(\frac{x_u}{l_w} \right)^2 + \alpha_4 \left(\frac{x_u}{l_w} \right) - \alpha_5 = 0,$$

where

$$\alpha_4 = \left(\frac{\phi}{\beta} - \lambda \right); \text{ and } \alpha_5 = \left(\frac{\phi}{2\beta} \right).$$

These equations were derived, assuming a rectangular wall section of depth l_w and thickness t_w that is subjected to combined uni-axial bending and axial compression. The vertical reinforcement is represented by an equivalent steel plate along the length of the section. The stress-strain curve assumed for concrete is as per IS 456 : 1978 whereas that for steel is assumed to be bi-linear. Two equations are given for calculating the flexural strength of the section. Their use depends on whether the section fails in flexural tension or in flexural compression.

ANNEX B

(Foreword)

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TO SEISMIC FORCES — CODE OF PRACTICE**

[*Page 3, clause 6.2.1(b)*] — Substitute the following for the existing formula:

$$\rho_{\min} = 0.24\sqrt{f_{ck}}/f_y$$

(CED 39)

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