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

(Incorporating Amendment Nos. 1 & 2)

UDC 69.059.25 (026) : 624.042.7

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

Price Group 7

Earthquake Engineering Sectional Committee, 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.

This edition 1.2 incorporates Amendment No. 1 (November 1995) and Amendment No. 2 (March 2002). Side bar indicates modification of the text as the result of incorporation of the amendments.

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 are located in seismic zone III, IV or V.

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:

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

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.

IS 13920 : 1993

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

A_h — horizontal reinforcement area within spacing S_v

A_k — area of concrete core of column

A_{sd} — reinforcement along each diagonal of coupling beam

A_{sh} — area of cross section of bar forming spiral or hoop

A_{st} — area of uniformly distributed vertical reinforcement

A_v — vertical reinforcement at a joint

C_w — centre to centre distance between boundary elements

D — overall depth of beam

D_k — diameter of column core measured to the outside of spiral or hoop

d — effective depth of member

d_w — effective depth of wall section

E_s — elastic modulus of steel

f_{ck} — characteristic compressive strength of concrete cube

f_y — yield stress of steel

h — longer dimension of rectangular confining hoop measured to its outer face

h_{st} — storey height

L_{AB} — clear span of beam

l_o — length of member over which special confining reinforcement is to be provided

l_w — horizontal length of wall

l_s — clear span of coupling beam

M_u — factored design moment on entire wall section

$M_{u,lim}^{Ah}$ — hogging moment of resistance of beam at end A

$M_{u,lim}^{As}$ — sagging moment of resistance of beam at end A

$M_{u,lim}^{Bh}$ — hogging moment of resistance of beam at end B

$M_{u,lim}^{Bs}$ — sagging moment of resistance of beam at end B

$M_{u,lim}^{bL}$ — moment of resistance of beam framing into column from the left

$M_{u,lim}^{bR}$ — moment of resistance of beam framing into column from the right

M_{uv} — flexural strength of wall web

P_u — factored axial load

S — pitch of spiral or spacing hoops

S_v — vertical spacing of horizontal reinforcement in web

t_w — thickness of wall web

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

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

V_j — shear resistance at a joint

V_u — factored shear force

V_{us} — shear force to be resisted by reinforcement

x_u, x_u^* — depth of neutral axis from extreme compression fibre

α — inclination of diagonal reinforcement in coupling beam

ρ — vertical reinforcement ratio

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 be M20 (f_{ck} = 20 MPa).

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

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.

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.
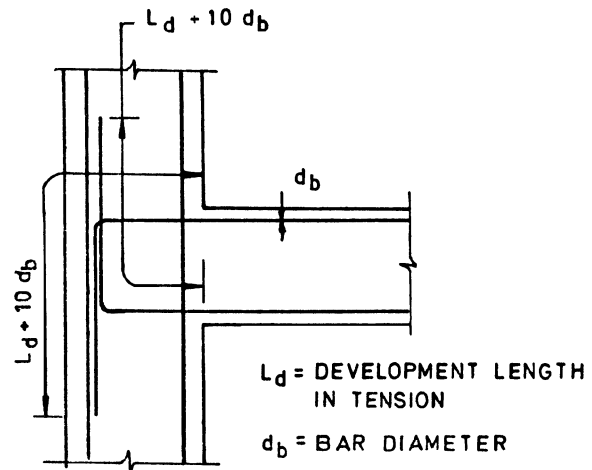

FIG. 1 ANCHORAGE OF BEAM BARS IN AN EXTERNAL JOINT

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 lengh 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.
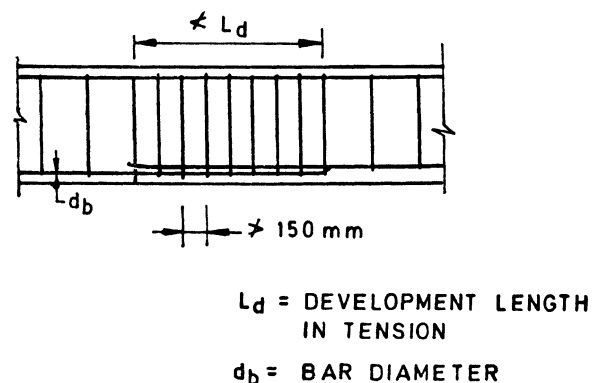

FIG. 2 LAP, SPLICE IN BEAM

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 :

a) calculated factored shear force as per analysis, and

b) 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.
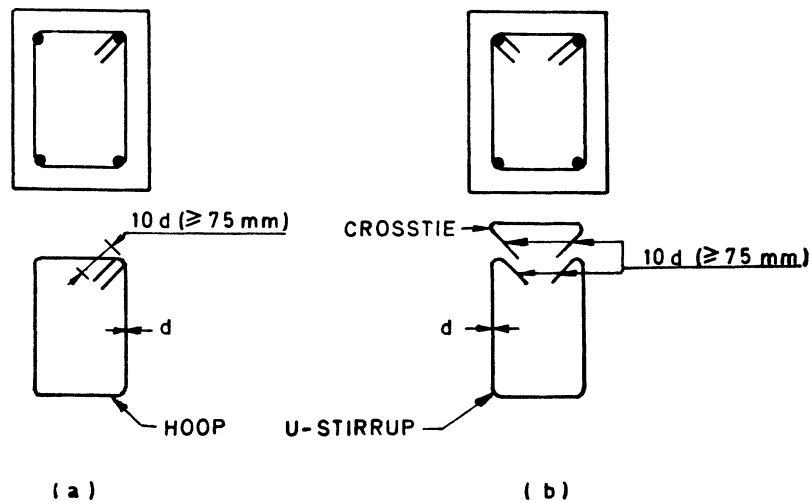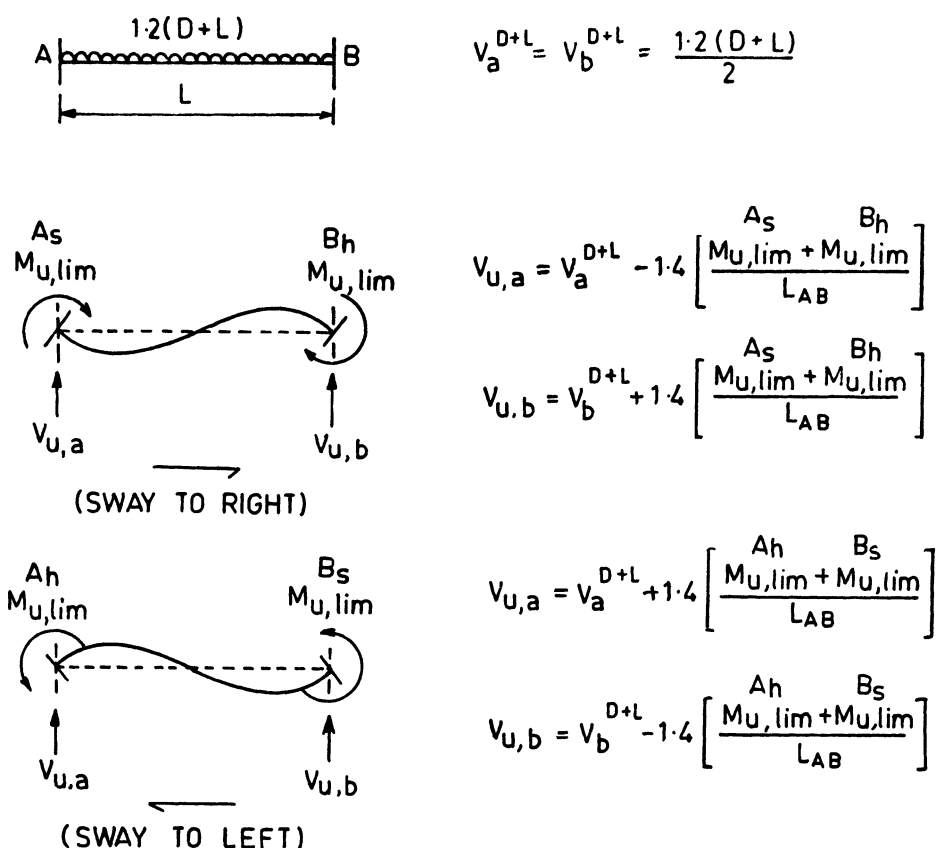

FIG. 3   BEAM WEB REINFORCEMENT

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.

MIN 2 BARS FOR FULL LENGTH
ALONG TOP AND BOTTOM FACE
AS ≥ ℓ MIN. Bd
AS ≤ ℓ MAX. Bd

50 mm MAX

50 mm MAX

d

db

2d

2d

HOOP SPACING
≯ $\frac{d}{2}$

HOOP SPACING
≤ d/4 AND 8 db

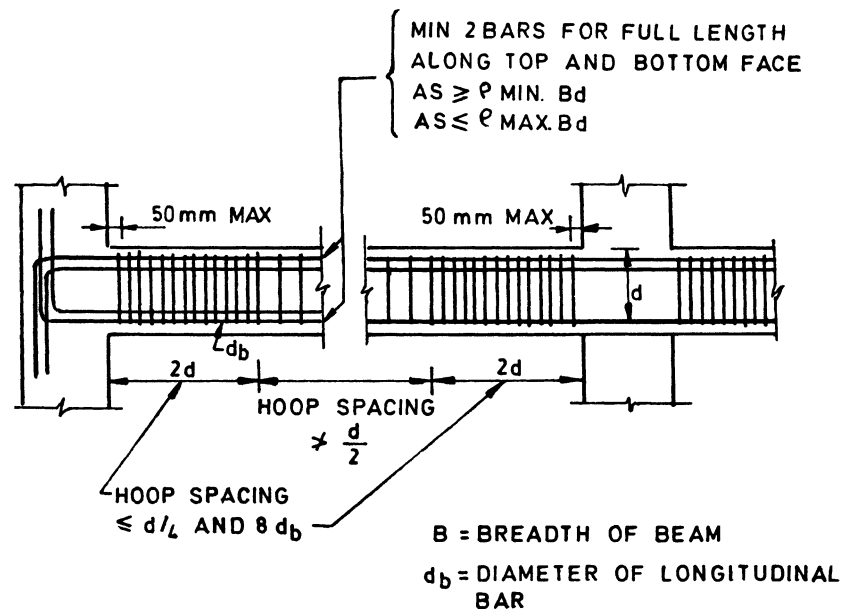B = BREADTH OF BEAM

db = DIAMETER OF LONGITUDINAL
BAR

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).
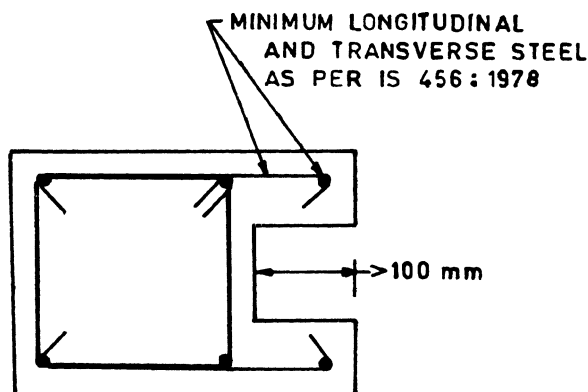

MINIMUM LONGITUDINAL AND TRANSVERSE STEEL AS PER IS 456 : 1978

>100 mm

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 hoop 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:

a) calculated factored shear force as per analysis, and

b) a factored shear force given by

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

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_{st} 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.
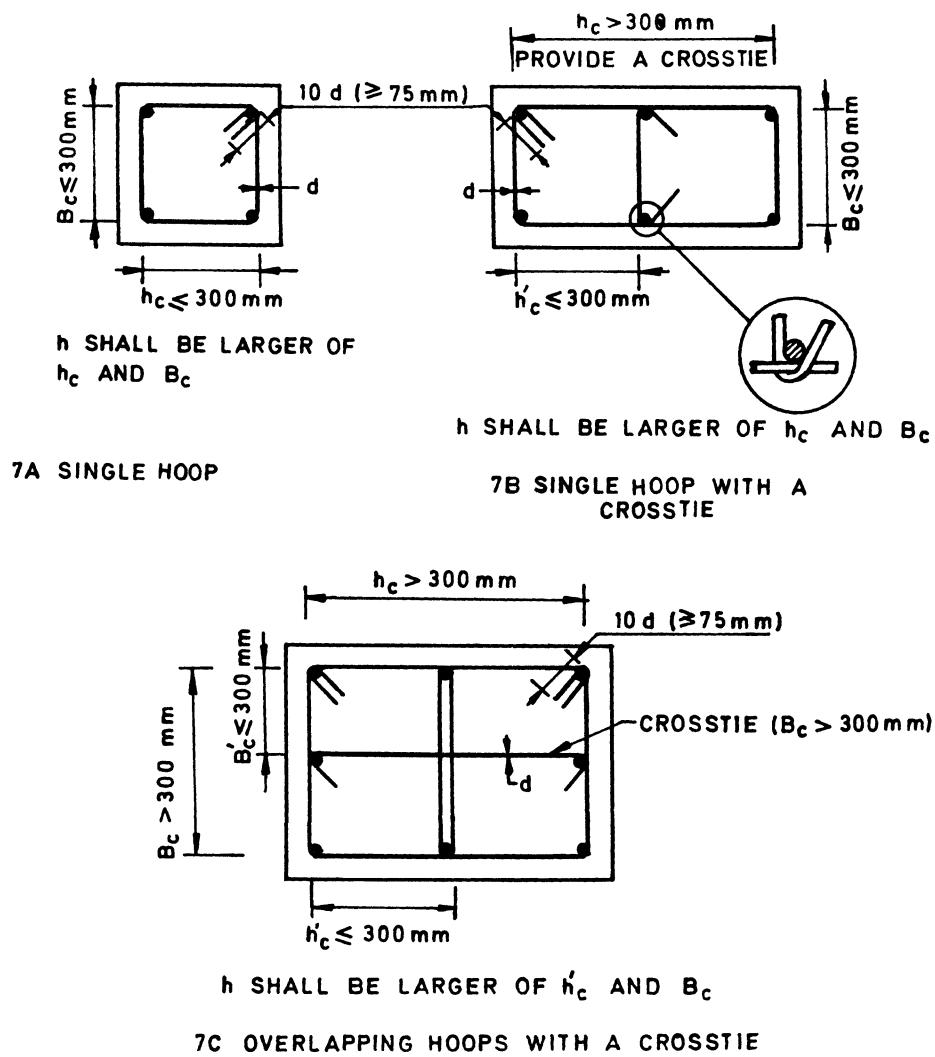
6

h SHALL BE LARGER OF
h$_c$ AND B$_c$

7A SINGLE HOOP

h SHALL BE LARGER OF h$_c$ AND B$_c$

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_0 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_0' 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 contra-flexure, 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

$$V_u = 1.4 \left(\frac{M_{u,lim}^{bL} + M_{u,lim}^{bR}}{h_{st}} \right)$$
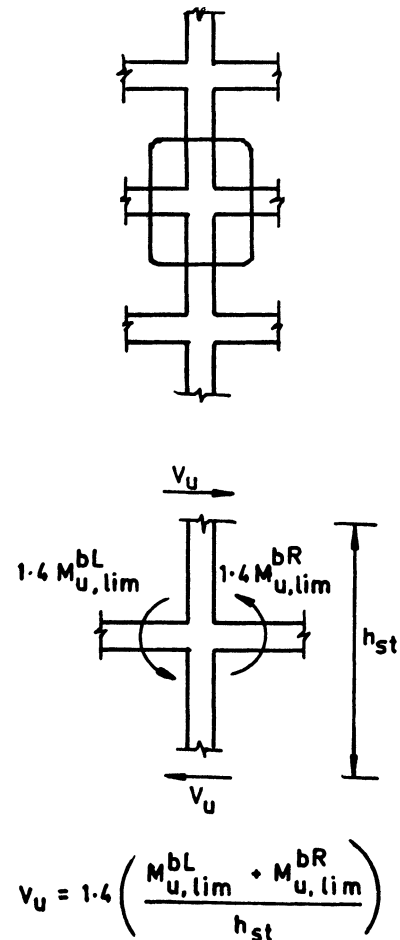
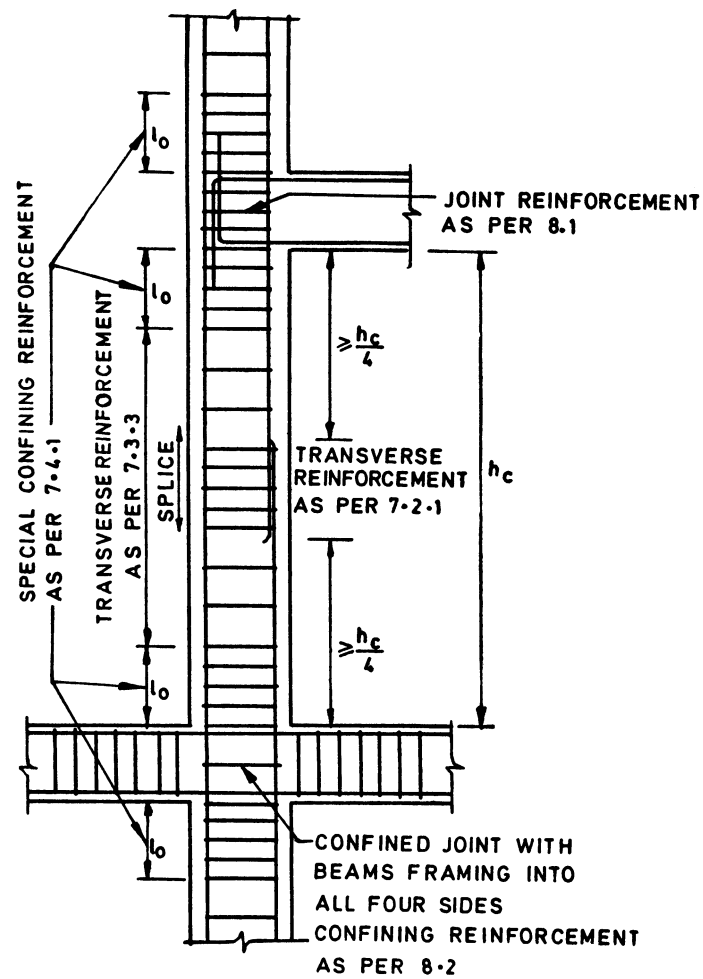Fig. 8   Calculation of Design Shear Force for Column

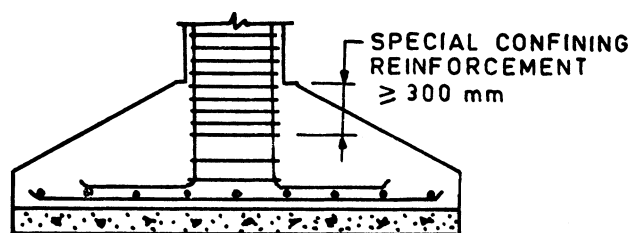FIG. 9   COLUMN AND JOINT DETAILING

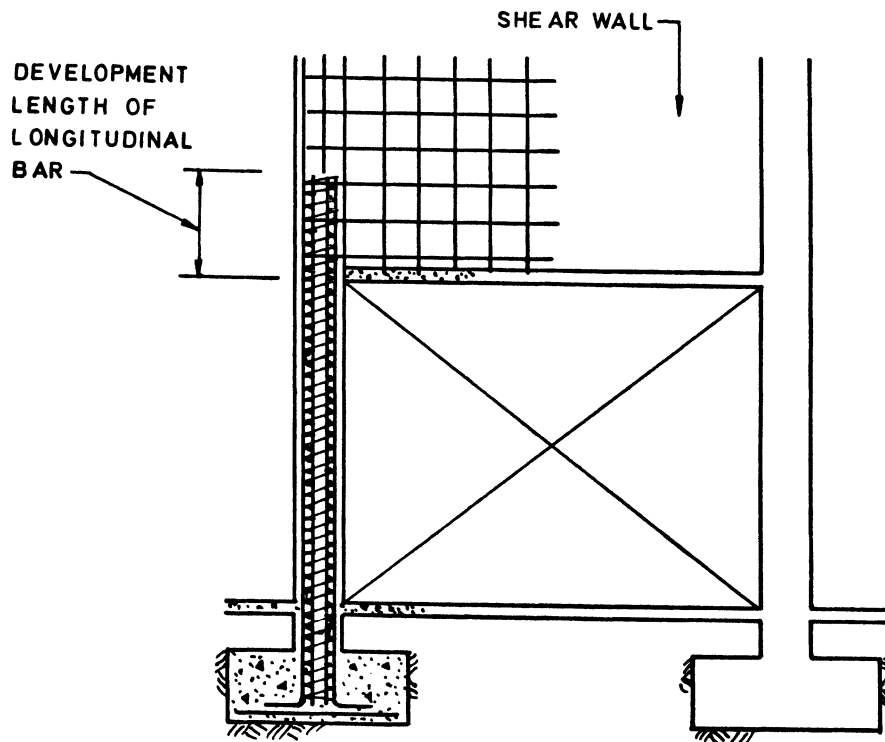FIG. 10   PROVISION OF SPECIAL CONFINING REINFORCEMENT IN FOOTINGS

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

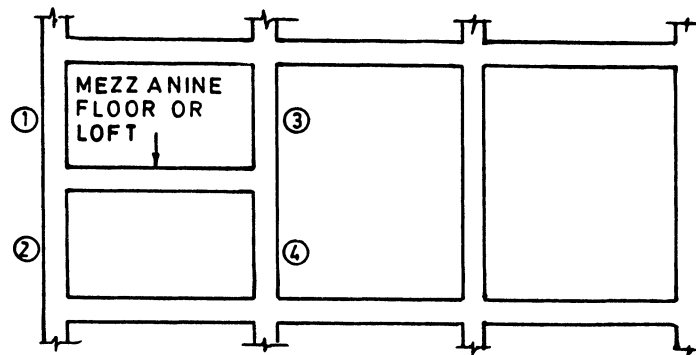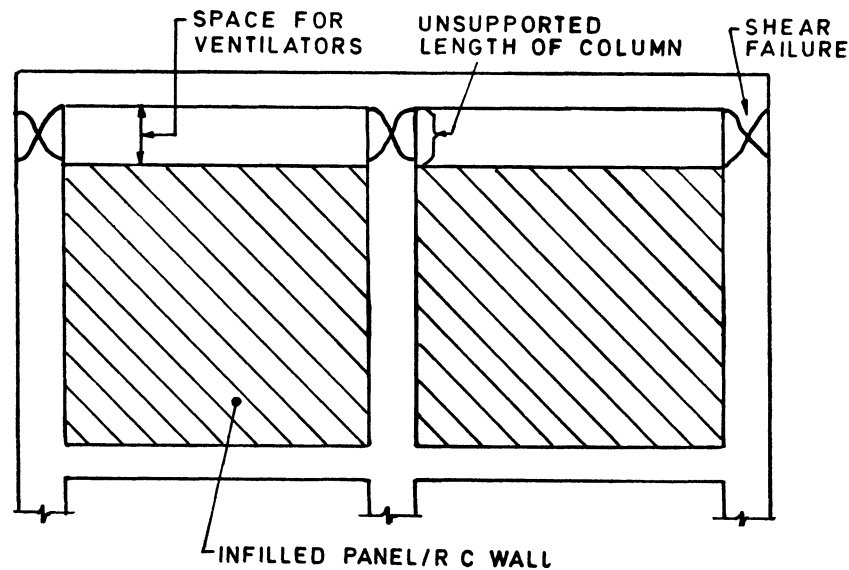A_k = area of the concrete core = $\frac{\pi}{4}D_k^2$

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 × 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 × 40 + 2 × 8 = 236 mm. Thus, the area of cross section of the bar forming circular hoop works out to be 47.28 mm². This is less than the cross sectional area of 8 mm bar (50.27 mm²). 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

10

INFILLED PANEL/R C WALL

(1), 2), (3) and (4) relatively stiff columns — They attract parge 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 crossties, 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 crossties 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 crossties, 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 × 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

11

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 by 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_{uv}, 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 reieforcement 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 $0.2f_{ck}$, boundaty 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

$$\frac{0 \cdot 1 \, l_s \, \sqrt{f_{ck}}}{D}$$

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:

$$A_{sd} = \frac{V_u}{1 \cdot 74 \, f_y \sin \alpha}$$

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:

$$\frac{0 \cdot 92}{f_y} \left(\tau_v - \frac{P_u}{A_g} \right)$$

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 $2 \, l_w$. 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 confirm 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 $x_u/l_w < x_u^*/l_w$,

$$\frac{M_{uv}}{f_{ck}\, t_w\, l^2_w} = \phi \left[\left(1 + \frac{\lambda}{\phi} \right) \left(\frac{1}{2} - 0{\cdot}416\, \frac{x_u}{l_w} \right) - \left(\frac{x_u}{l_w} \right)^2 \left(0{\cdot}168 + \frac{\beta^2}{3} \right) \right]$$

where

$$\frac{x_u}{l_w} = \left(\frac{\phi + \lambda}{2\,\phi + 0{\cdot}36} \right); \quad \frac{x_u^*}{l_w} = \left(\frac{0{\cdot}003\,5}{0{\cdot}003\,5 + 0{\cdot}87\, f_y/E_s} \right);$$

$$\phi = \left(\frac{0{\cdot}87\, f_y\, \rho}{f_{ck}} \right); \quad \lambda = \left(\frac{P_u}{f_{ck}\, t_w\, l_w} \right);$$

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

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

β = 0.87 $f_y/(0.003\,5\, E_s)$,

E_s = elastic modulus of steel, and

P_u = axial compression on wall.

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

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

where

$$\alpha_1 = \left[0{\cdot}36 + \phi \left(1 - \frac{\beta}{2} - \frac{1}{2\,\beta} \right) \right]$$

$$\alpha_2 = \left[0{\cdot}15 + \frac{\phi}{2} \left(1 - \beta - \frac{\beta^2}{2} - \frac{1}{3\,\beta} \right) \right]; \text{ and } \alpha_3 = \frac{\phi}{6\,\beta} \left(\frac{1}{(\, x_u/l_w\,)} - 3 \right)$$

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*)

COMMITTEE COMPOSITION

Earthquake Engineering Sectional Committee, CED 39

Chairman	*Representing*
Dr A. S. Arya	72/6 Civil Line, Roorkee
Members	
Shri O. P. Aggarwal	Indian Roads Congress, New Delhi
Shri G. Sharan (*Alternate*)	
Dr K. G. Bhatia	Bharat Heavy Electricals Ltd, New Delhi
Dr C. Kameshwara Rao (*Alternate*)	
Shri A. K. Singh (*Alternate*)	
Shri S. C. Bhatia	National Geophysical Research Institute (CSIR), Hyderabad
Dr B. K. Rastogi (*Alternate*)	
Dr A. R. Chandrasekaran	Department of Earthquake Engineering, University of Roorkee, Roorkee
Dr Brijesh Chandra (*Alternate*)	
Dr B. V. K. Lavania (*Alternate*)	
Dr S. N. Chatterjee	Indian Meterological Department, New Delhi
Shri S. K. Nag (*Alternate*)	
Shri K. T. Chaubal	North Eastern Council, Shillong
Dr B. K. Paul (*Alternate*)	
Dr A. V. Chummar	Indian Society of Earthquake Technology, Roorkee
Dr S. K. Kaushik (*Alternate*)	
Director Embankment (N & W)	Central Water Commission (ERDD), New Delhi
Director CMDD (NW & S) (*Alternate*)	
Director Standards (B & S), RDSO	Railway Board, Ministry of Railways
Joint Director Standards (B & S) CB-I, RDSO, Lucknow (*Alternate*)	
Kumari E. Divatia	National Hydro-Electric Power Corporation Ltd, New Delhi
Shri C. R. Venkatesha (*Alternate*)	
Shri I. D. Gupta	Central Water & Power Research Station, Pune
Shri J. G. Padale (*Alternate*)	
Shri V. K. Kulkarni	Department of Atomic Energy, Bombay
Shri P. C. Koteswara Rao (*Alternate*)	
Shri V. Kumar	National Thermal Power Corporation Ltd, New Delhi
Shri R. S. Bajaj (*Alternate*)	
Shri M. Z. Kurien	Tata Consulting Engineers, Bombay
Shri K. V. Subramanian (*Alternate*)	
Shri A. K. Lal	National Buildings Organization, New Delhi
Shri T. R. Bhatia (*Alternate*)	
Shri S. K. Mittal	Central Building Research Institute, Roorkee
Shri S. S. Narang	Central Water Commission (CMDD), New Delhi
Shri A. D. Narian	Ministry of Transport, Department of Surface Transport (Roads Wing), New Delhi
Shri O. P. Aggarwal (*Alternate*)	
Shri P. L. Narula	Geological Survey of India, Calcutta
Shri A K. Srivastava (*Alternate*)	
Research Officer	Irrigation Department, Govt of Maharashtra, Nasik
Dr D. Sengupta	Engineers India Ltd, New Delhi
Shri R. K. Grover (*Alternate*)	
Dr R. D. Sharma	Nuclear Power Corporation, Bombay
Shri U. S. P. Verma (*Alternate*)	
Col R. K. Singh	Engineer-in-Chief's Branch, Army Headquarters, New Delhi
Lt-Col B. D. Bhattopadhyaya (*Alternate*)	
Dr P. Srinivasulu	Structural Engineering Research Centre (CSIR), Madras
Dr N. Lakshmanan (*Alternate*)	
Superintending Engineer (D)	Central Public Works Department, New Delhi
Executive Engineer (D) II (*Alternate*)	
Dr A. N. Tandon	In personal capacity (*B-7/50 Safdarjung Development Area, New Delhi*)
Shri J. Venkataraman, Director (Civ Engg)	Director General, BIS (*Ex-officio Member*)

Secretary

Shri S. S. Sethi
Director (Civ Engg), BIS

Earthquake Resistant Construction Subcommittee, CED 39 : 1

Bureau of Indian Standards

BIS is a statutory institution established under the *Bureau of Indian Standards Act*, 1986 to promote harmonious development of the activities of standardization, marking and quality certification of goods and attending to connected matters in the country.

Copyright

Review of Indian Standards

Amendments are issued to standards as the need arises on the basis of comments. Standards are also reviewed periodically; a standard along with amendments is reaffirmed when such review indicates that no changes are needed; if the review indicates that changes are needed, it is taken up for revision. Users of Indian Standards should ascertain that they are in possession of the latest amendments or edition by referring to the latest issue of 'BIS Catalogue' and 'Standards : Monthly Additions'.

This Indian Standard has been developed from Doc : No. CED 39 (5263).

Amendments Issued Since Publication

Amend No.	Date of Issue
Amd. No. 1	November 1995
Amd. No. 2	March 2002

BUREAU OF INDIAN STANDARDS

Headquarters:

Manak Bhavan, 9 Bahadur Shah Zafar Marg, New Delhi 110002. Telegrams: Manaksanstha
Telephones: 323 01 31, 323 33 75, 323 94 02 (Common to all offices)

Regional Offices: Telephone

Central : Manak Bhavan, 9 Bahadur Shah Zafar Marg ⎰ 323 76 17
NEW DELHI 110002 ⎱ 323 38 41

Eastern : 1/14 C. I. T. Scheme VII M, V. I. P. Road, Kankurgachi ⎰ 337 84 99, 337 85 61
KOLKATA 700054 ⎱ 337 86 26, 337 91 20

Northern : SCO 335-336, Sector 34-A, CHANDIGARH 160022 ⎰ 60 38 43
⎱ 60 20 25

Southern : C. I. T. Campus, IV Cross Road, CHENNAI 600113 ⎰ 235 02 16, 235 04 42
⎱ 235 15 19, 235 23 15

Western : Manakalaya, E9 MIDC, Marol, Andheri (East) ⎰ 832 92 95, 832 78 58
MUMBAI 400093 ⎱ 832 78 91, 832 78 92

Branches : AHMEDABAD. BANGALORE. BHOPAL. BHUBANESHWAR. COIMBATORE. FARIDABAD. GHAZIABAD. GUWAHATI. HYDERABAD. JAIPUR. KANPUR. LUCKNOW. NAGPUR. NALAGARH. PATNA. PUNE. RAJKOT. THIRUVANANTHAPURAM. VISHAKHAPATNAM.