भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS

MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG, NEW DELHI 110002
Phone: + 91 11 23230131, 23233375, 23239402 Extn 4434; Fax: + 91 11 23235529 email: sak.bis@nic.in

व्यापक परिचालन मसौदा

हमारा संदर्भ : सीईडी 39/टी-8 **28 मार्च 2014**

प्राप्तकर्ता :

1 **सिविल इंजीनियरी विभाग परिषद् के रूचि रखने वाले सदस्य**
2 **सीईडी 39 के सभी सदस्य**
3 **रूचि रखने वाले अन्य निकाय**

महोदय(यों),

निम्नलिखित मानक का मसौदा संलग्न हैं:

प्रलेख संख्या	**शीर्षक**
सीईडी 39(7941)	भूकंपीय बल के प्रभाव के अंर्तगत प्रबलित कंकरीट संरचनाओं का तन्य विस्तार - रीति संहिता [आईएस 13920 का पहला पुनरीक्षण]

कृपया इस मानक के मसौदे का अवलोकन करें और अपनी सम्मतियॉ यह बताते हुए भेजे कि यदि ये मानक के रूप में प्रकाशित हो तो इन पर अमल करने में आपके व्यवसाय अथवा कारोबार में क्या कठिनाइया आ सकती हैं ।

सम्मतिया भेजने की अंतिम तिथि **31-05-2014**

सम्मति यदि कोई हो तो कृपया अधोहस्ताक्षरी को उपरलिखित पते पर संलग्न **फोर्मेट** में भेजें या sak.bis@nic.in पर ईमेल कर दें ।

यदि कोई सम्मति प्राप्त नहीं होती है अथवा सम्मति में केवल भाषा सम्बन्धी त्रुटि हुई तो उपरोक्त प्रलेख को यथावत अंतिम रूप दिया जाएगा । यदि सम्मित तकनीकी प्रकृति की हुई तो विषय समिति के अध्यक्ष के परामर्श से अथवा उनकी इच्छा पर आगे की कार्यवाही के लिए विषय समिति को भेजे जाने के बाद प्रलेख को अंतिम रूप दे दिया जाएगा।

यह प्रलेख भारतीय मानक ब्यूरो की वैबसाइट www.bis.org.in पर भी है ।

धन्यवाद ।

भवदीय,
Sd/-

(जे रॉय चौधरी)
वैज्ञानिक 'एफ' एवं प्रमुख (सिविल इंजीनियरी)

संलग्न : उपरिलखित

भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS

MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG, NEW DELHI 110002
Phone: + 91 11 23230131, 23233375, 23239402 Extn 4434; Fax: + 91 11 23235529 email: sak.bis@nic.in

DRAFT IN WIDE CIRCULATION

DOCUMENT DESPATCH ADVICE

Reference	Date
CED 39/T- 8	28 03 2014

TECHNICAL COMMITTEE: EARTHQUAKE ENGINEERING SECTIONAL COMMITTEE, CED 39

ADDRESSED TO :

1 Interested Members of Civil Engineering Division Council, CEDC
2. All members of CED 39
3. All others interested

Dear Sir,

Please find enclosed the following document:

Doc No.	Title
CED 39 (7941)	**Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces - Code of Practice (First Revision of IS 13920)**

Kindly examine the draft standard and forward your views stating any difficulties which you are likely to experience in your business or profession, if this is finally adopted as National Standard.

Last Date for comments: **31 05 2014**

Comments if any, may please be made in the format as given overleaf and mailed to the undersigned at sak.bis@nic.in .

In case no comments are received or comments received are of editorial nature, you will kindly permit us to presume your approval for the above document as finalized. However, in case of comments of technical in nature are received then it may be finalized either in consultation with the Chairman, Sectional Committee or referred to the Sectional Committee for further necessary action if so desired by the Chairman, Sectional Committee.

The document is also hosted on BIS website **www.bis.org.in**.

Thanking you,

Yours faithfully,

Sd/-

(J. Roy Chowdhury)
Sc `F' & Head (Civil Engg.)

Encl: as above

Draft Indian Standard

(Not to be reproduced without the permission of BIS or used as an Indian Standard)

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

(*First Revision* of IS 13920)

ICS : 91.120.25

Earthquake Engineering Sectional Committee, CED 39	Last Date for Comments **31 May 2014**

FOREWORD

Formal clause will be added later.

After the formulation of IS 4326 'Code of Practice for Earthquake-Resistant Design and Construction of Buildings' which had provisions for addressing special features in the design and construction of earthquake-resistant RC buildings, and certain details for achieving ductility in reinforced concrete (RC) buildings; in order to keep abreast with the rapid developments and extensive research on earthquake-resistant design of RC structures, the technical committee prepared a separate standard for earthquake-resistant design and detailing of RC structures, namely IS 13920:1993.

The first edition (1993) of this standard incorporated some important provisions that were not covered in IS 4326:1976 for design of RC Structures. With the revision of IS 4326 as IS 4326:2013 (*third revision*), the revision of IS 13920 takes importance and hence, this standard addresses the following salient aspects:

(a) Significant experience was gained from performance of reinforced concrete structures (that were designed and detailed as per IS 4326) during past earthquakes. Many deficiencies were identified and corrected.
(b) Provisions on design and detailing of beams and columns as given in IS 4326 were revised with an aim to provide them with adequate stiffness, strength and ductility and to make them capable of undergoing extensive inelastic deformations and dissipating seismic energy in a stable manner.
(c) Specifications were included on lower limits for strengths of material of earthquake-resistant RC structural systems.
(d) Geometric constraints were imposed on cross-sections of flexural members. Provisions were revised on minimum and maximum reinforcement limits. Requirements were made explicit for detailing of longitudinal reinforcement in beams at joint faces, splices and anchorage requirements. Provisions were included for calculating seismic design shear force, and detailing transverse reinforcement in beams.
(e) For members subjected to axial load and bending moment, constraints were imposed on cross-sectional aspect ratio and on absolute dimensions. Also, provisions are included for (i) location of lap splices, (ii) calculation of seismic design shear force of structural walls, and (iii) special confining reinforcement in regions of columns that are expected to undergo cyclic inelastic deformations during a severe earthquake shaking.
(f) Specifications were included on a seismic design and detailing of reinforced concrete structural walls. These provisions assisted in (i) estimation of design shear force and bending moment demand on structural wall sections, (ii) estimation of design moment capacity of wall sections, (iii) detailing of reinforcement in the wall web,

boundary elements, coupling beams, around openings, at construction joints, and (iv) providing sufficient length for development, lap splicing and anchorage of longitudinal steel.

Following the earthquakes that occurred after the release of IS 13920 in 1993 (especially the 1997 Jabalpur, 2001 Bhuj, 2004 Sumatra, 2006 Sikkim, and 2011 Sikkim earthquakes), it was felt that this standard needs further improvement.

In this first revision of IS 13920, the following changes are incorporated:

(a) The title is revised to reflect the “Design” provisions that existed and new ones added, that determine the sizing, proportioning and reinforcement in RC members meant to resist earthquake shaking.
(b) The following new provisions are added:
  (i) Column-to-beam strength ratio provision has been added in keeping with the strong column – weak beam design philosophy for moment resisting frames;
  (ii) Shear design of beam-column joints;
  (iii) Design of slender RC structural walls is improved. The principle of superposition is dropped for estimating the design moment of resistance of structural walls with boundary elements. Instead, procedure is mentioned for estimating the same.
(c) Most provisions that existed earlier have been redrafted. Also, the sequence of sections is re-organised for greater clarity to designers and for removing ambiguities.
(d) The name of the standard has been modified to reflect various provisions included that pertain to “design” of RC components participating in seismic resistance.

Further, while the common methods of design and construction have been covered in this standard for RC structural systems with moment resisting frames and RC structural systems with moment resisting frames and structural walls that participate in resisting earthquake force, design and construction of other lateral load resisting structural systems made of reinforced concrete but not covered by this standard, may be permitted by the approving agency or a committee constituted by the agency only on production of satisfactory evidence from experiments on prototype sub-assemblages and structures, and nonlinear analyses demonstrating their adequacy to resist earthquake shaking expected in the region where the structures are expected to be built. Such nonlinear analyses shall demonstrate that the collapse mechanism of the proposed structure is desirable and that the lateral deformation capacity of the structure is sufficient to resist the ground deformation imposed in the region where the structure is located. The committee of the approving agency shall comprise of competent engineers with the necessary experience and shall have the authority to review the data submitted, ask for additional data, tests and to frame special rules for such structural systems not covered under this standard.

In the formulation of this standard, due weightage has been given to the need for international coordination among standards prevailing in different seismic regions of the world. In the preparation of this standard, assistance has been derived from the following publications:

(i) ACI 318-11, Building code requirements for reinforced concrete and commentary, published by American Concrete Institute.
(ii) IBC 2003 International Building Code, published by International Code Council, Inc.

(iii) EN 1998-1:2003(E) Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, European Committee for Standardization, Brussels.

(iv) NZS 3102:Part 1:1995 Concrete structure standard, published by Standards Council, New Zealand.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (*Revised*)'. The number of significant places retained in the rounded off value should be same as that of the specified value in this standard.

Draft Indian Standard

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

(*First Revision* of IS 13920)

ICS : 91.120.25

Earthquake Engineering Sectional Committee, CED 39	Last Date for Comments **31 May 2014**

1. SCOPE

1.1 This standard covers the requirements for designing and detailing of members of reinforced concrete (RC) structures designed to resist lateral effects of earthquake shaking, so as to give them adequate stiffness, strength and ductility to resist severe earthquake shaking without collapse. Even though the general concepts adopted in this standard for structures are applicable for RC bridge systems, provisions of this standard shall be taken only as a guide for RC bridge piers and wells of large cross-sections, but are not sufficient. This standard addresses lateral load resisting structural systems of RC structures composed of:

(a) RC *Moment Resisting Frames*,
(b) RC *Moment Resisting Frames with Unreinforced Masonry Infill Walls*; and
(c) RC *Moment Resisting Frames with RC Structural Walls*.
(d) RC Structural Walls

1.1.1 Provisions of this standard shall be adopted in all lateral load resisting systems of RC structures located in Seismic Zone III, IV or V.

1.1.2 The provisions for RC structures given herein apply specifically to monolithic RC construction, and not for precast RC structures. Precast and/or prestressed concrete members may be used, only if they are designed to provide similar level of ductility as that of monolithic RC structures during or after an earthquake. Specialist literature must be referred to for design and construction of such structures. The adequacy of such designs shall be demonstrated by adequate, appropriate experimentation and nonlinear dynamic structural analyses.

1.1.3 RC monolithic members assumed not to participate in the lateral force resisting system (**3.7**) shall be permitted provided that their effect on the seismic response of the system is accounted for. Consequence of failure of structural and non-structural members not part of the lateral force resisting system shall also be considered in design.

2. REFERENCES

2.1 The standards listed below contain provision which through reference in this text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision and parties to agreement based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated therein.

IS No.	***Title***
456 : 2000	Code of Practice for Plain and Reinforced Concrete (*Fourth Revision*)
800 : 2007	Code of Practice for Structural Steel
1343 : 2012	Code of Practice for Prestressed Concrete Structures (*Second Revision*)
1786 : 2008	Specification for High Strength Deformed Steel Bars and Wires for Concrete Reinforcement (*Fourth Revision*)
1893 (Part 1) : 2002	Criteria for Earthquake Design of Structures (*Fifth Revision*)
xxxxx : 201x	Specification for Reinforcement couplers for mechanical splices of bars for concrete reinforcement *(under print)*

3. TERMINOLOGY

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

3.1 Beams
These are horizontal members of the moment resisting frames with flexural and shearing actions.

3.2 Boundary Elements
These are portions along the ends of a structural wall that are strengthened by longitudinal and transverse reinforcement. They may have the same thickness as that of the wall web.

3.3 Columns
These are vertical members of the moment resisting frames with axial, flexural and shearing actions.

3.4 Cover Concrete
It is that concrete which is not confined by transverse reinforcement.

3.5 Cross-tie
It is a continuous bar having a 135^o hook with an extension of 6 times diameter (but not < 75 mm) at one end and a hook not less than 90^o with an extension of 6 times diameter (but not < 65 mm) at the other end. The hooks shall engage peripheral longitudinal bars. In general, the 90^o hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end.

3.6 Gravity Columns in Buildings
It is a column, which is not part of the lateral load resisting system and designed only for force actions (*i.e.,* axial force, shear force and bending moments) due to gravity loads. But, it should be able to resist the gravity loads at lateral displacement imposed by the earthquake forces.

3.7 Lateral Force Resisting System
It is that part of the structural system which participates in resisting forces induced by earthquake.

3.8 Moment-Resisting Frame
It is a three-dimensional structural system composed of interconnected members, without structural walls, so as to function as a complete self-contained unit with or

without the aid of horizontal diaphragms or floor bracing systems, in which the members resist gravity and lateral forces primarily by flexural actions.

3.8.1 Special Moment Resisting Frame (SMRF)

It is a moment-resisting frame specially detailed to provide ductile behaviour as per the requirements specified in **5**, **6**, **7** and **8** of this standard.

3.8.2 Ordinary Moment Resisting Frame (OMRF)

It is a moment-resisting frame not meeting special detailing requirements for ductile behaviour specified in this standard.

3.9 Stirrup

It is a single steel bar bent into a closed loop having a 135° hook with an extension of 6 times diameter (but not < 65 mm) at each end, which is embedded in the confined core of the section, and placed normal to the longitudinal axis of the RC beam or column.

3.10 Structural Wall

It is a vertically oriented planar element that is primarily designed to resist lateral force effects (axial force, shear force and bending moment) in its own plane. It is commonly known as *Shear Wall*.

3.11 Special Structural Wall

It is a structural wall meeting special detailing requirements for ductile behaviour specified in **10**.

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, unless otherwise specified.

A_e – effective cross sectional area of a joint

A_{ej} – effective shear area of a joint

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 stirrup

A_{st} – area of uniformly distributed vertical reinforcement

A_v – vertical reinforcement at a joint

b_b – width of beam

b_c – width of column

b_j – effective width of a joint

D – overall depth of beam

D_k – diameter of column core measured to the outside of spiral or stirrup

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 (in MPa)

f_y – yield stress of steel reinforcing bars, or 0.2% proof strength of steel reinforcing steel (in MPa)

h – longer dimension of rectangular confining stirrup measured to its outer face

h_c – depth of column

h_j – effective depth of a joint

h_{st} – clear storey height

h_w – overall height of RC structural wall

L_{AB} – clear span of beam

L_d – development length of bar in tension

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 couplings beam

M_u – design moment of resistance of entire RC beam, column or wall section

M_{c1} – Design moment of resistance of column section

M_{c2} – Design moment of resistance of column section

M_{g1} – Design moment of resistance of beam section

M_{g2} – Design moment of resistance of beam section

M_u^{Ah} – Hogging design moment of resistance of beam at end A

M_u^{As} – Sagging design moment of resistance of beam at end A

M_u^{Bh} – Hogging design moment of resistance of beam at end B

M_u^{Bs} – Sagging design moment of resistance of beam at end B

M_u^{BL} – Design moment of resistance of beam framing into column from the left

M_u^{BR} – Design moment of resistance of beam framing into column from the right

M_{uw} – Design moment of resistance of web of RC structural wall alone

P_u – Factored axial load

s_v – Spacing of stirrups along the longitudinal direction of beam or column

t_w – Thickness of web of RC structural wall

${V_{u,a}}^{D+L}$ – Factored shear force demand at end A of beam due to dead and live loads

${V_{u,b}}^{D+L}$ – Factored shear force demand at end B of beam due to dead and live loads

V_j – Design shear resistance of a joint

V_u – Factored shear force

V_{us} – *Design* shear resistance offered at a section by steel stirrups

x_u, ${x_u}^*$ – Depth of neutral axis from extreme compression fibre

α – Inclination of diagonal reinforcement in coupling beam

ρ – Area of longitudinal reinforcement as a fraction of gross area of cross-section in a RC beam, column or structural wall

ρ_c – Area of longitudinal reinforcement on the compression face of a beam as a fraction of gross area of cross-section

$(\rho_h)_{min}$ – Minimum area of horizontal reinforcement of a structural wall as a fraction of gross area of cross-section

$(\rho_{v,be})_{min}$ – Minimum area of vertical reinforcement in each boundary element of a structural wall as a fraction of gross area of cross-section of each boundary element

$(\rho_{v,net})_{min}$ – Minimum area of vertical reinforcement of a structural wall as a fraction of gross area of cross-section of the wall

$(\rho_{v,web})_{min}$ Minimum area of vertical reinforcement in web of a structural wall as a fraction of gross area of cross-section of web

ρ_{max} – Maximum area of longitudinal reinforcement permitted on the tension face of a beam as a fraction of gross area of cross-section

ρ_{min} – Minimum area of longitudinal reinforcement to be ensured on the tension face of a beam as a fraction of gross area of cross-section

τ_c – Design shear strength of concrete

$\tau_{c,\max}$ – Maximum nominal shear stress permitted at a section of RC beam, column or structural wall

τ_v – Nominal shear stress at a section of RC beam, column or structural wall

5. GENERAL SPECIFICATIONS

5.1 The design and construction of reinforced concrete buildings shall be governed by provisions of IS 456, except as modified by the provisions of this standard for those elements participating in lateral force resistance.

5.2 Minimum grade of structural concrete shall be M20, but M25 for buildings

(i) more than 15m in height in Seismic Zones III, IV and V,
(ii) but not less than that required by IS 456 based on exposure conditions.

5.3 Steel reinforcement resisting earthquake-induced forces in RC frame members and in boundary elements of RC structural walls shall comply with **5.3.1**, **5.3.2** and **5.3.3.**

5.3.1 Steel reinforcements used shall be
(i) of grade Fe 415 (conforming to IS 1786), and
(ii) of grade Fe 500 and Fe 550, i.e., high strength deformed steel bars produced by thermo-mechanical treatment process having elongation more than 14.5 percent, and conforming to IS 1786.

5.3.2 The actual 0.2% proof strength of steel bars based on tensile test must not exceed their characteristic 0.2% proof strength by more than 20%.

5.3.3 The ratio of the actual ultimate strength to the actual 0.2% proof strength shall be at least 1.25.

5.4 In RC frame buildings, lintel beams shall preferably not be integrated into the columns to avoid short column effect. When integrated, they shall be included in the analytical model for structural analysis. Similarly, plinth beams (where provided), and staircase beams and slabs framing into columns shall be included in the analytical model for structural analysis.

5.5 RC regular moment-resisting frame buildings shall have planar frames oriented along the two principal plan directions of buildings. Irregularities listed in IS 1893 (Part 1) shall be avoided. Buildings with any of the listed irregularities perform poorly during earthquake shaking; in addition, buildings with floating columns and set-back columns also perform poorly. When any such irregularities are adopted, detailed nonlinear analyses shall be performed to demonstrate that there is no threat to loss of life and property.

6. BEAMS

6.1 General

Requirements of this section shall apply to beams resisting earthquake-induced effects, in which the factored axial compressive stress due to gravity and earthquake effects does not exceed $0.08f_{ck}$. Beams, in which the factored axial compressive stress exceeds $0.08f_{ck}$, shall be designed as per requirements of **7**.

6.1.1 Beams shall preferably have width-to-depth ratio of more than 0.3.

6.1.2 Beams shall not have width less than 200 mm.

6.1.3 Beams shall not have depth *D* more than 1/4th of clear span. This may not apply to the floor beam of frame staging of elevated RC water tanks.

6.1.4 Width of beam b_w shall not exceed the width of supporting member c_2 plus distance on either side of supporting member equal to the smaller of (a) and (b)

(a) Width of supporting member c_2
(b) 0.75 times breadth of supporting member c_1 (*see* Fig. 1a and 1b)

Transverse reinforcement for the width of a beam that exceeds width of the column c_2 shall be provided as shown in Fig. 1b throughout the beam span including within the beam column joint.

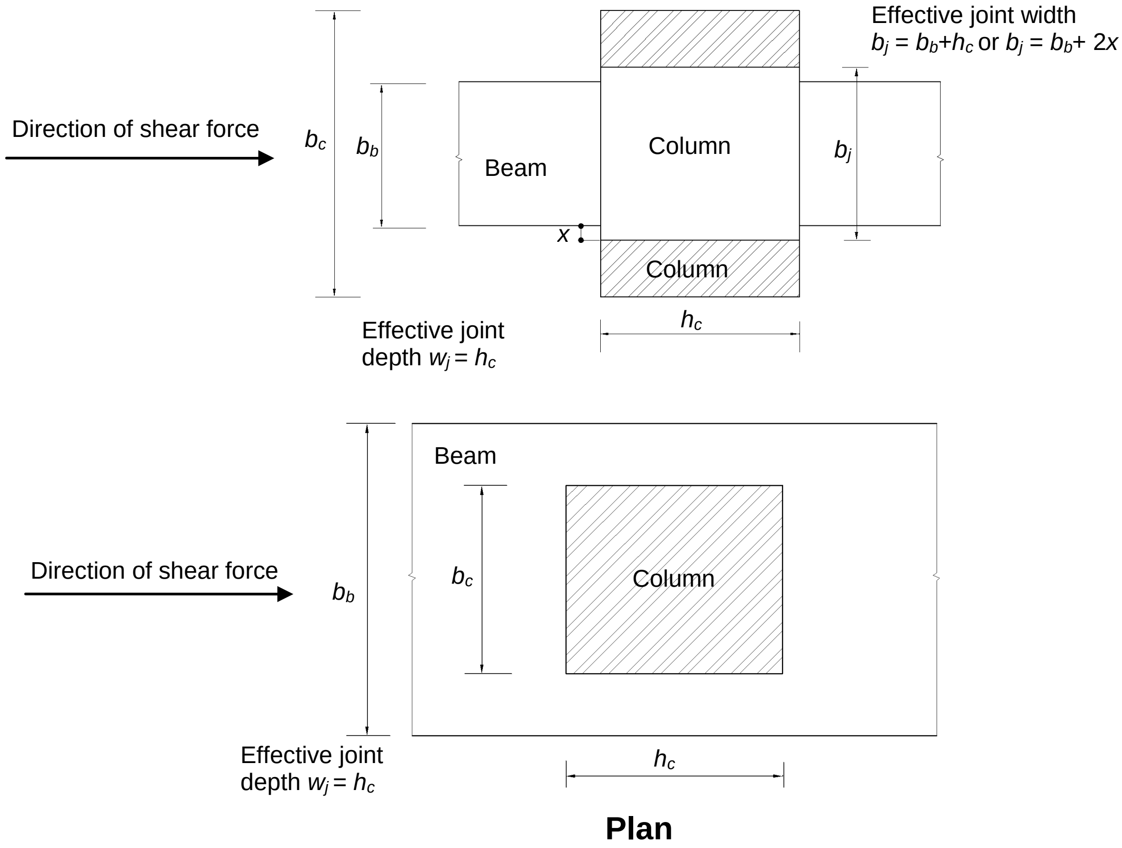

FIG. 1(A) PLAN VIEW OF A BEAM COLUMN JOINT SHOWING EFFECTIVE BREADTH AND WIDTH OF JOINT

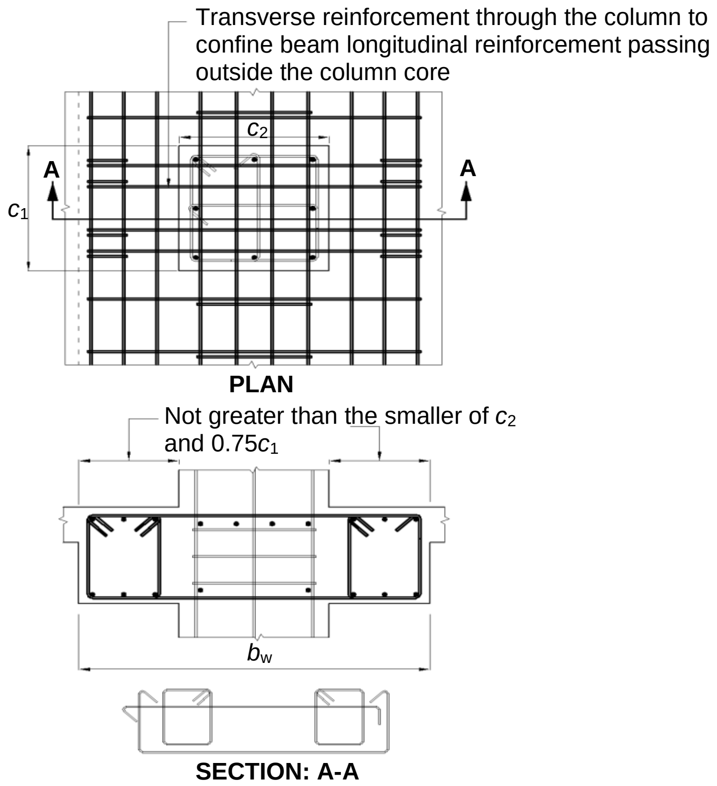

FIG. 1(B) MAXIMUM EFFECTIVE WIDTH OF WIDE BEAM AND REQUIRED TRANSVERSE REINFORCEMENT

6.2 Longitudinal Reinforcement

6.2.1 (a) Beams shall have at least two 12 mm diameter bars each at the top and bottom faces.

(b) Minimum longitudinal steel ratio ρ_{min} required on any face at any section is

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

6.2.2 Maximum longitudinal steel ratio $\rho_{\max}$ provided on any face at any section is 0.025.

6.2.3 Longitudinal steel on bottom face of a beam framing into a column (at the face of the column) shall be at least half the steel on its top face at the same section. At exterior joints, the anchorage length calculation shall consider this bottom steel to be tension steel.

6.2.4 Longitudinal steel in beams at any section on top or bottom face shall be at least $1/4^{th}$ of longitudinal steel provided at the top face of the beam at the face of the column; when the top longitudinal steel in the beam at the two supporting column faces is different, the larger of the two shall be considered.

6.2.5 At an exterior joint, top and bottom bars of beams shall be provided with anchorage length beyond the inner face of the column, equal to development length of the bar in tension plus 10 times bar diameter minus the allowance for 90° bends (*see* Fig. 2); here,

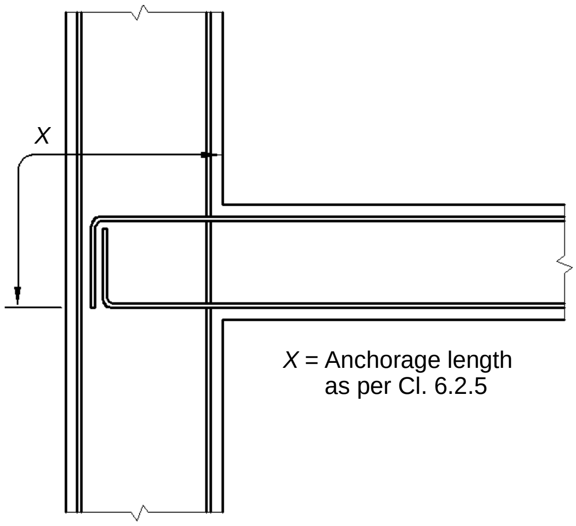

FIG. 2 ANCHORAGE OF LONGITUDINAL BEAM BARS AT EXTERIOR BEAM-COLUMN JOINT

6.2.6 Splicing of Longitudinal Bars

6.2.6.1 *Lap Splices*

When adopted, closed stirrups shall be provided over the entire length over which the longitudinal bars are spliced and,

(a) the spacing of these stirrups shall not exceed 150 mm (*see* Fig. 3).
(b) the lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.
(c) lap splices shall not be provided
 (i) within a joint,
 (ii) within a distance of 2*d* from face of the column, and
 (iii) within a quarter length of the beam adjoining the location where flexural yielding may occur under earthquake effects.
(d) not more than 50% of area of steel bars on either top or bottom face shall be spliced at any one section.

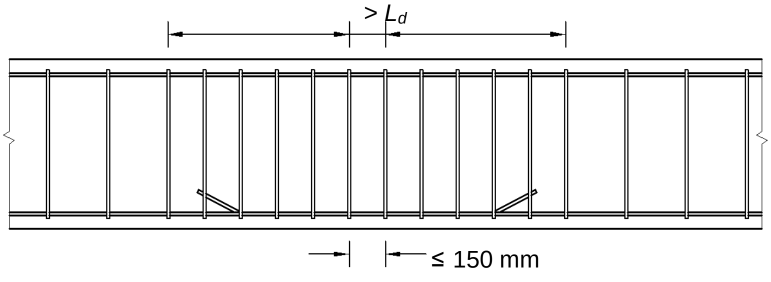

FIG. 3 LAP LENGTH AT LOCATION OF SPLICING OF LONGITUDINAL BARS IN BEAM

6.2.6.2 *Mechanical Couplers*

Mechanical couplers [conforming to IS xxxxx (*under print*)] shall be used when longitudinal steel bars have to be continued for beam spans larger than their manufacture lengths. Further,

(a) only those mechanical splices conforming to the above standard and capable of developing the specified tensile strength of spliced bar shall be permitted within a distance equal to two times the depth of the member from the member face or in any location where yielding of reinforcement is likely to take place

(b) the spacing between adjacent longitudinal bars shall be based also on the outer size of the coupler to allow easy flow of concrete.

6.2.6.3 *Welded Splices*

Welded splices shall not be used in beams for a distance equal to two times the depth of the member from the member face or in any location where yielding of reinforcement is likely to take place at any location, not more than 50% of area of steel bars shall be spliced at any one section.

Welding of stirrups, ties, inserts or other similar elements to vertical reinforcement bars required as per design is not permitted, in any seismic zone.

6.3 Transverse Reinforcement

6.3.1 Only vertical stirrups shall be used in beams (*see* Fig. 4a); inclined stirrups shall not be used.

(a) In normal practice, a stirrup is made of a single bent bar. But, it may be made of two bars also, namely a U-stirrup with a 135^{o} hook with an extension of 6 times diameter (but not less than 65 mm) at each end, embedded in the core concrete, and a cross-tie (*see* Fig. 4b).
(b) The hooks of the stirrups and cross-ties shall engage around peripheral longitudinal bars. Consecutive crossties engaging the same longitudinal bars shall have their 90^{o} hooks at opposite sides of the beam. When the longitudinal reinforcement bars are secured by cross-ties in beams that have a slab on one side, the 90^{o} hooks of the cross-ties shall be placed on that side.

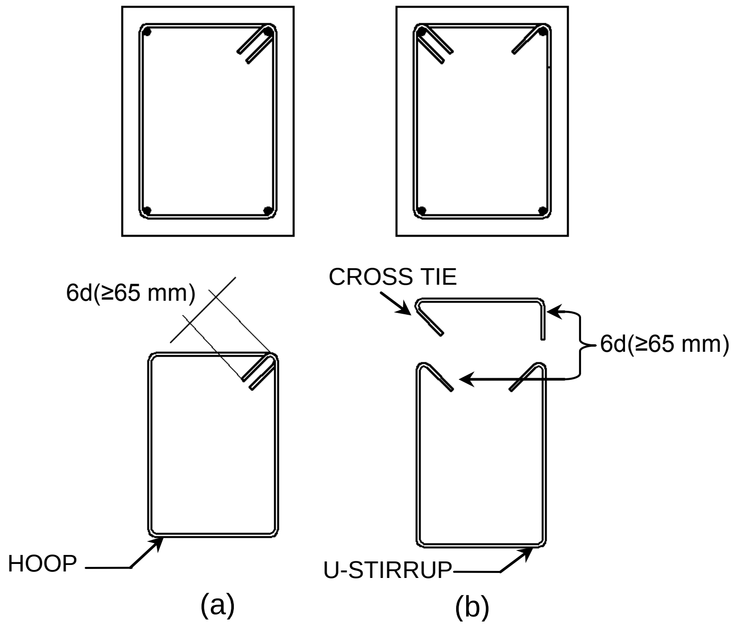

FIG. 4 DETAILS OF TRANSVERSE REINFORCEMENT IN BEAMS

6.3.2 The minimum diameter of a stirrup shall be 8 mm.

6.3.3 Shear force capacity of a beam shall be more than larger of:
(a) Factored shear force as per linear structural analysis, and
(b) Factored gravity shear force, plus equilibrium shear force when plastic hinges are formed at both ends of the beam (*see* Fig. 5) given by.

(i) For sway to right:

$$V_{u,a} = V_{u,a}^{D+L} - 1.4 \frac{M_u^{As} + M_u^{Bh}}{L_{AB}} \quad \text{and}$$

$$V_{u,b} = V_{u,b}^{D+L} + 1.4 \frac{M_u^{As} + M_u^{Bh}}{L_{AB}}.$$

(ii) For sway to left:

$$V_{u,a} = V_{u,a}^{D+L} - 1.4 \frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}} \quad \text{and}$$

$$V_{u,b} = V_{u,b}^{D+L} + 1.4 \frac{M_u^{Ah} + M_u^{Bs}}{L_{AB}}$$

where M_u^{As}, M_u^{Ah}, M_u^{Bs} *and* M_u^{Bh} are sagging and hogging moments of resistance of the beam section at ends *A* and *B*, respectively. These shall be calculated as per IS 456. L_{AB} is clear span of beam. $V_{u,a}^{D+L}$ and $V_{u,b}^{D+L}$ are the factored shear forces at ends *A* and *B*, respectively, due to vertical loads acting on the span; the partial safety factor for dead and live loads shall be 1.2, and the beam shall be considered to be simply supported for this estimation.

The design shear force demand at end *A* of the beam shall be the larger of the two values of $V_{u,a}$ computed above. Similarly, the design shear force demand at end *B* shall be the larger of the two values of $V_{u,b}$ computed above.

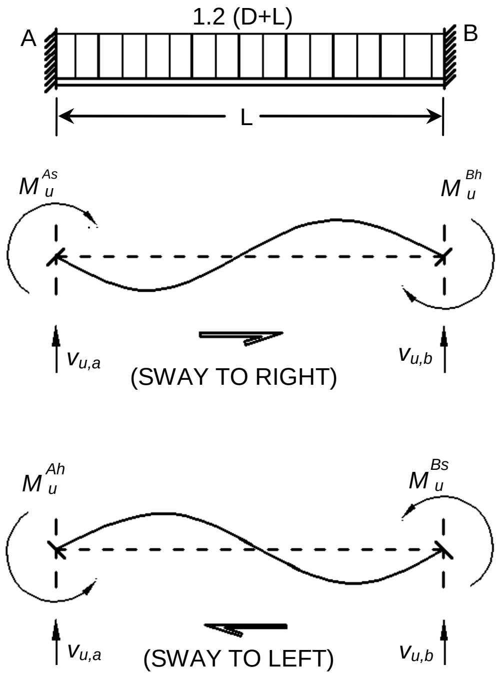

FIG. 5 CALCULATION OF DESIGN SHEAR FORCE DEMAND ON BEAMS UNDER PLASTIC HINGE ACTION AT THEIR ENDS

6.3.4 In the calculation of design shear force capacity of RC beams, contributions of the following shall NOT be considered:
(a) bent up bars ,
(b) inclined stirrups, and
(c) concrete in the RC section.

6.3.5 Close Spacing of Stirrups

Spacing of stirrups over a length of 2d at either end of a beam shall not exceed

(a) d/4,
(b) 8 times the diameter of the smallest longitudinal bar; and
(c) 100 mm (*see* Fig. 6).

6.3.5.1 The first stirrup shall be at a distance not exceeding 50 mm from the joint face.

6.3.5.2 Stirrups shall be provided over a length equal to 2d on either side of a section where flexural yielding may occur under earthquake effects. Over the remaining length of the beam, vertical stirrups shall be provided at a spacing not exceeding d/2.

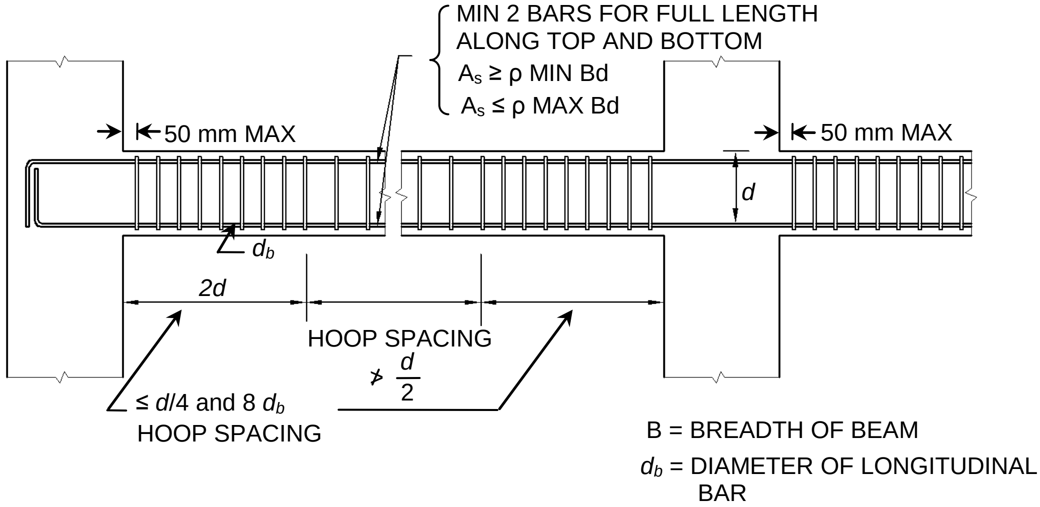

FIG. 6 DETAILS OF TRANSVERSE REINFORCEMENT IN BEAMS

7. COLUMNS AND INCLINED MEMBERS

7.1 Geometry

Requirements of this section shall apply to columns and inclined members resisting earthquake-induced effects, in which the factored axial compressive stress due to gravity and earthquake effects exceeds $0.08f_{ck}$.
The factored axial compressive stress considering all load combinations relating to seismic loads shall be limited to $0.40f_{ck}$ in all such members, except in those covered under **10**.

7.1.1 The minimum dimension of a column shall not be less than

(a) $20d_b$, where d_b is diameter of the largest diameter longitudinal reinforcement bar in the beam passing through or anchoring into the column at the joint, or
(b) 300 mm (*see* Fig. 7).

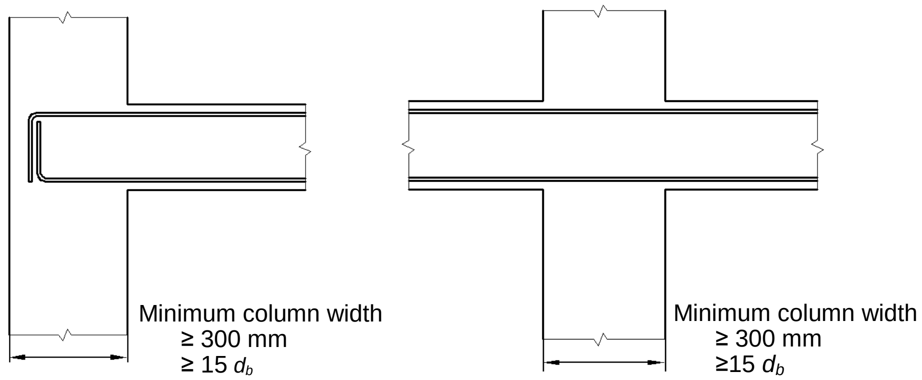

FIG. 7 MINIMUM SIZE OF RC COLUMNS BASED ON DIAMETER OF LARGEST LONGITUDINAL REINFORCEMENT BAR IN BEAMS FRAMING INTO IT

7.1.2 The cross-section aspect ratio (i.e., ratio of smaller dimension to larger dimension of the cross section of a column or inclined member) shall not be less than 0.4. Vertical members of RC buildings whose cross-section aspect ratio is less than 0.4 shall be designed as per requirements of **9**.

7.2 Relative Strengths of Beams and Columns at a Joint

7.2.1 At each beam-column joint of a moment-resisting frame, the sum of nominal design strength of columns meeting at that joint (with nominal strength calculated for the factored axial load in the direction of the lateral force under consideration so as to give least column nominal design strength) along each principal plane shall be at least 1.4 times the sum of nominal design strength of beams meeting at that joint in the same plane (*see* Fig. 8).

In the event of a beam-column joint not conforming to above, the columns at the joint shall be considered to be gravity columns only and shall not be considered as part of the lateral load resisting system.

7.2.1.1 The design moments of resistance of a beam shall be estimated based on the principles of mechanics and the limiting strain states of the limit state design method enunciated in IS 456. The design moment of resistance of a column shall be estimated as in case of beams corresponding to zero axial force on the design *P-M* interaction diagram.

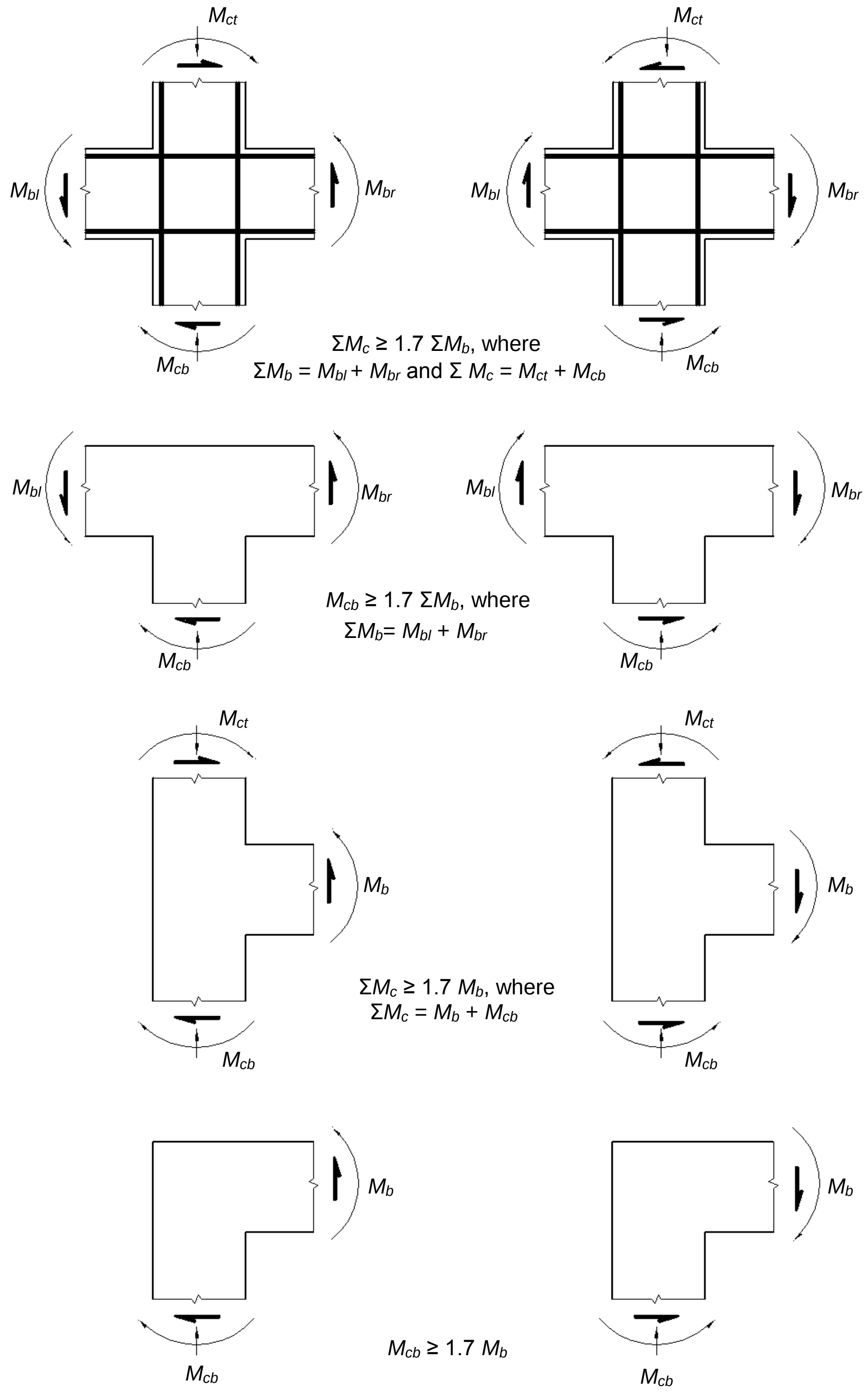

FIG. 8 STRONG COLUMN – WEAK BEAM REQUIREMENT

7.2.1.2 This check shall be performed at each joint for both positive and negative directions of shaking in the plane under consideration. Further, in this check, design moments of resistance in beam(s) meeting at a joint shall be considered in the same direction, and similarly the design moments of resistance of column(s) at the same joint shall be considered to be in the direction opposite to that of the moments in the beams.

7.2.1.3 This check shall be waived at all joints at roof level only, in buildings more than 4 storeys tall.

7.3 Longitudinal Reinforcement

7.3.1 Circular columns shall have minimum of 6 bars.

7.3.2 Splicing of Longitudinal Bars

7.3.2.1 Lap Splices

When adopted, closed stirrups shall be provided over the entire length over which the longitudinal bars are spliced.

(a) The spacing of these stirrups shall not exceed 150 mm.
(b) The lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.
(c) Lap splices shall be provided only in the central half of clear column height, and not
  (i) within a joint, or
  (ii) within a distance of 2*d* from face of the beam.
(d) Not more than 50% of area of steel bars shall be spliced at any one section.

7.3.2.2 *Mechanical couplers*

Mechanical couplers [conforming to IS xxxxx (*under print*)] shall be used. Further, only those mechanical splices conforming to the above standard and capable of developing the specified tensile strength of spliced bar shall be permitted within a distance equal to two times the depth of the member from the column face or in any location where yielding of reinforcement is likely to take place

7.3.2.3 *Welded Splices*

Welded splices shall not be used in columns for a distance equal to two times the depth of the member from the member face or in any location where yielding of reinforcement is likely to take place. At any location, not more than 50% of area of steel bars shall be spliced at any one section.

But, welding of stirrups, ties, inserts or other similar elements to vertical reinforcement bars required as per design is not permitted, in any seismic zone.

7.3.3 A column that extends more than 100 mm beyond the confined core owing to architectural requirement (*see* Fig. 9).shall be detailed in the following manner.

(a) When the contribution of this area is considered in the estimate of strength of columns, it shall have at least the minimum longitudinal and transverse reinforcement given in this standard.
(b) When the contribution of this area is NOT considered in the estimate of strength of columns, it shall have at least the minimum longitudinal and transverse reinforcement given in IS 456.

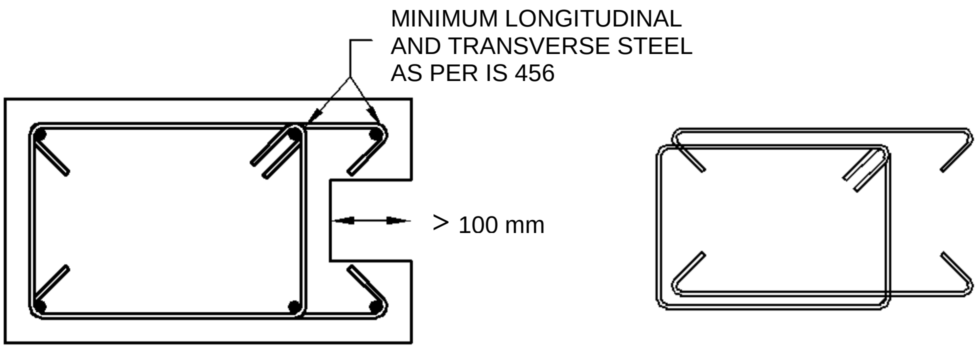

FIG. 9 REINFORCEMENT REQUIREMENT IN COLUMNS WITH PROJECTION MORE THAN 100 MM BEYOND CORE

7.4 Transverse Reinforcement

7.4.1 Transverse reinforcement shall consist of closed loop

(a) spiral or circular stirrups in circular columns, and
(b) rectangular stirrups in rectangular columns,

In either case, the closed stirrup shall have 135° hook ends with an extension of 6 times its diameter (but not < 65 mm) at each end, which are embedded in the confined core of the column (*see* Fig. 10a).

7.4.2 When rectangular stirrups are used,

(a) the minimum diameter permitted of transverse reinforcement bars is 8 mm, when diameter of longitudinal bar is less than or equal to 32 mm, and 10 mm, when diameter of longitudinal bar is more than 32 mm;
(b) the maximum spacing of parallel legs of stirrups shall be 300 mm centre to centre;
(c) a cross-tie shall be provided, if the length of any side of the stirrup exceeds 300 mm (*see* Fig. 10b); the cross-tie shall be placed perpendicular to this stirrup whose length exceeds 300mm. Alternatively, a pair of overlapping stirrups may be provided within the column (*see* Fig. 10c). In either case, the hook ends of the stirrups and cross-ties shall engage around peripheral longitudinal bars. Consecutive cross-ties engaging the same longitudinal bars shall have their 90°hooks on opposite sides of the column; and
(d) the maximum spacing of stirrups shall be half the least lateral dimension of the column, except where special confining reinforcement is provided as per **8**.

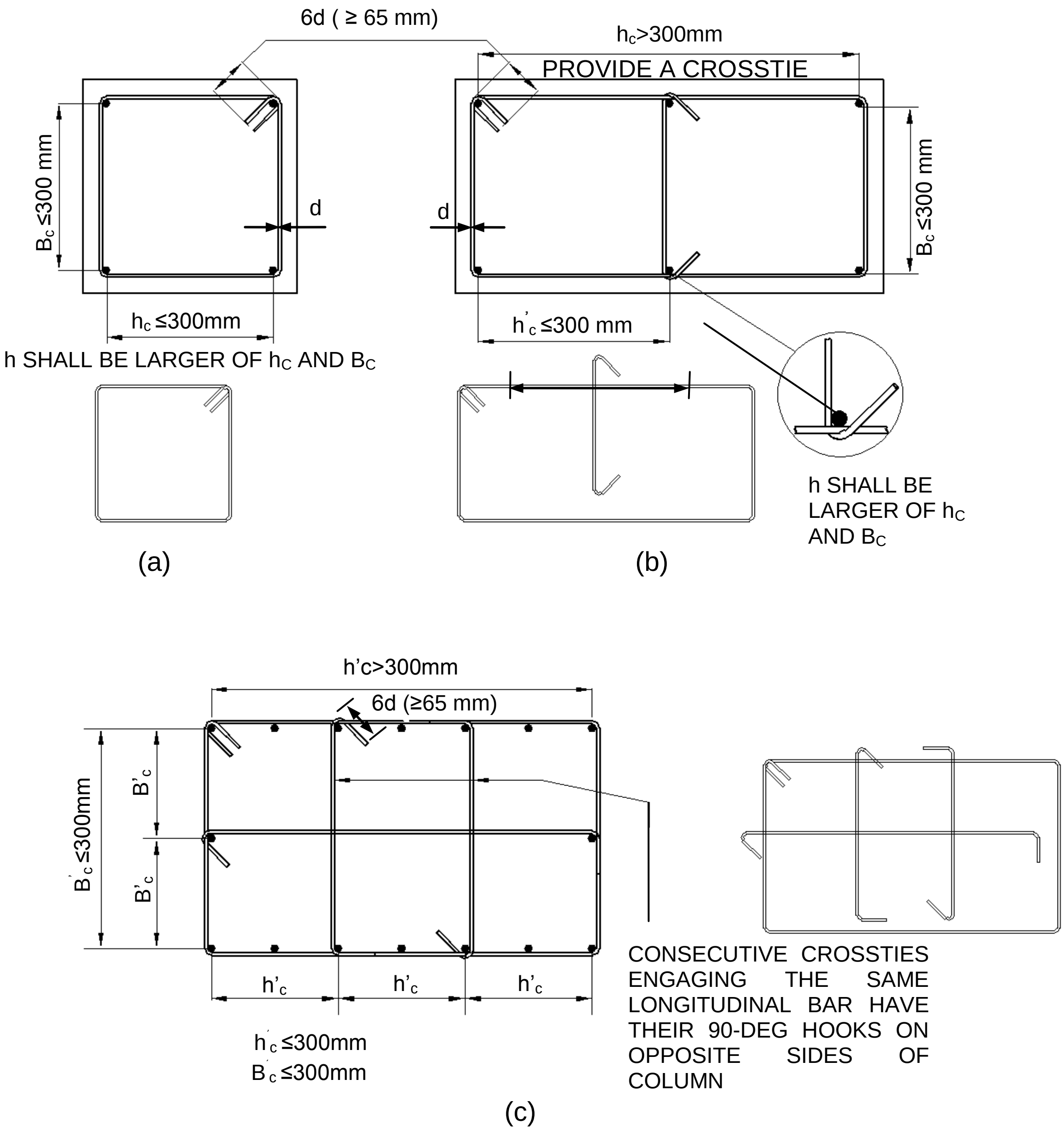

FIG. 10 DETAILS OF TRANSVERSE REINFORCEMENT IN COLUMNS.

7.5 Design Shear Force in Columns

The design shear force demand on columns is the larger of:

(a) Factored shear force demand as per linear structural analysis, and
(b) factored equilibrium shear force demand when plastic hinges are formed at both ends of the beams given by

(i) For sway to right:

$$V_u = 1.4\frac{\left(M_u^{As} + M_u^{Bh}\right)}{h_{st}},$$

(ii) For sway to left:

$$V_u = 1.4\frac{\left(M_u^{Ah} + M_u^{Bs}\right)}{h_{st}},$$

where M_u^{As}, M_u^{Ah}, M_u^{Bs} *and* M_u^{Bh} are design sagging and hogging moments of resistance of beams framing into the column on opposite faces *A* and *B*, respectively, with one hogging moment and the other sagging (*see* Fig. 11); and h_{st} the storey height. The design moments of resistance of beam sections shall be calculated as per IS 456.

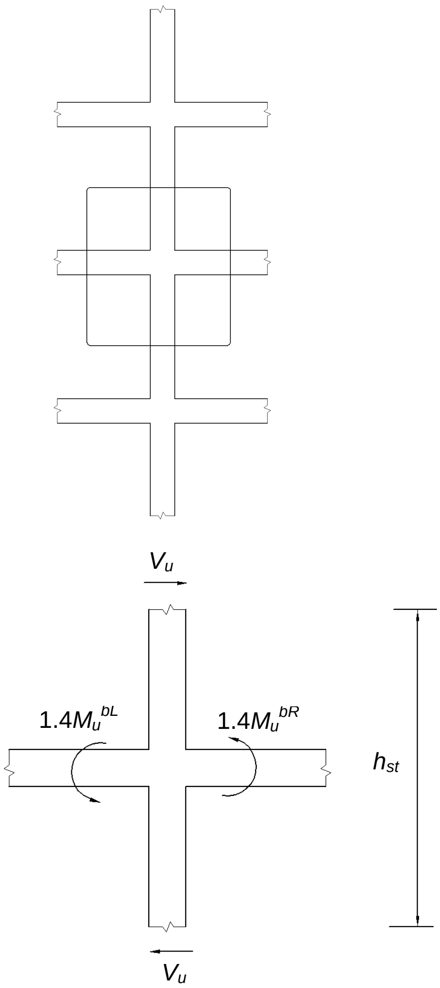

FIG. 11 EQUILIBRIUM DESIGN SHEAR FORCE DEMAND ON COLUMN WHEN PLASTIC HINGES ARE FORMED AT BEAM ENDS

7.5.1 The calculation of design shear force capacity of RC columns shall be calculated as per IS 456.

8. SPECIAL CONFINING REINFORCEMENT

The requirements of this section shall be met with in beams and columns, unless a larger amount of transverse reinforcement is required from shear strength considerations given in **6.3.3** for beams and **7.5** for columns.

8.1 Flexural yielding is likely in beams during strong earthquake shaking and in columns when the shaking intensity exceeds the expected intensity of earthquake shaking (*see* Fig. 12). This special confining reinforcement shall

(a) be provided over a length l_o from the face of the joint towards mid-span of beams and mid heights of columns, on either side of the joint; where l_o is not less than
   (i) larger lateral dimension of the member at the section where yielding occurs,
   (ii) 1/6 of clear span of the member, or
   (iii) 450 mm.

(b) have a spacing not more than
   (i) 1/4 of minimum member dimension of the beam or column,
   (ii) 6 times diameter of the smallest longitudinal reinforcement bars,
   (iii) 100 mm,
   but need not be less than 75 mm.

(c) have area A_{sh} of cross section of the bar forming stirrups or spiral of at least
   (i) in circular stirrups or spirals:

$$A_{sh} = Max\begin{bmatrix} 0.090 s_v D_k \dfrac{f_{ck}}{f_y}\left(\dfrac{A_g}{A_k} - 1\right) \\ 0.024 s_v D_k \dfrac{f_{ck}}{f_y} \end{bmatrix}$$

where
s_v = pitch of spiral or spacing of stirrups,
D_k = diameter of core of circular column measured to outside of spiral/stirrup,
f_{ck} = characteristic compressive strength of concrete cube,
f_y = 0.2% Proof Strength of transverse steel reinforcement bars,
A_g = gross area of column cross-section, and
A_k = area of concrete core of column = $\dfrac{\pi}{4} D_k^2$

   (ii) in rectangular stirrups:

$$A_{sh} = Max\begin{bmatrix} 0.180 s_v h \dfrac{f_{ck}}{f_y}\left(\dfrac{A_g}{A_k} - 1\right) \\ 0.05 s_v h \dfrac{f_{ck}}{f_y} \end{bmatrix}$$

where
h = longer dimension of rectangular stirrup measured to its outer face, which does not exceed 300 mm (*see* Fig. 10b), and
A_k= area of confined concrete core in rectangular stirrup measured to its outer dimensions.

h of the stirrup could be reduced by introducing crossties (*see* Fig. 10c). In such cases, A_k shall be measured as overall core area, regardless of stirrup arrangement. Hooks of cross-ties shall engage peripheral longitudinal bars.

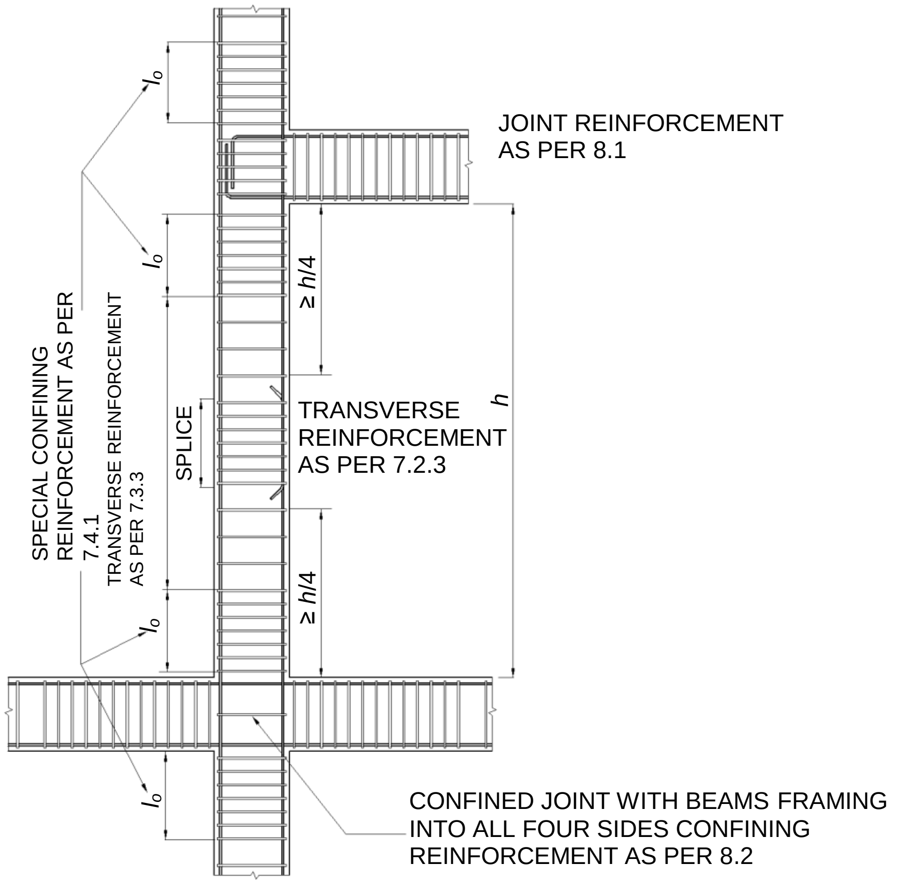

FIG. 12 COLUMN AND JOINT DETAILING

8.2 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. 13).

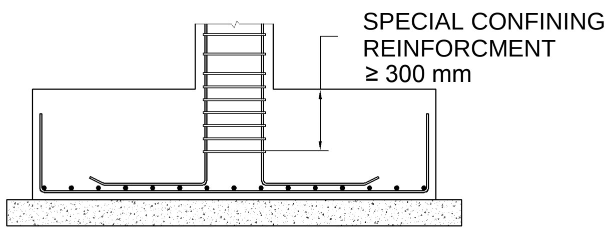

FIG. 13 PROVISION OF SPECIAL CONFINING REINFORCEMENT IN FOOTING

8.3 When the calculated point of contra-flexure, under the effect of gravity and earthquake effects, is not within the middle half of the member clear height, special confining reinforcement shall be provided over the full height of the column.

8.4 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 due to abrupt changes in cross-section size, or unintended restraint to the column provided by stair-slab, mezzanine floor, plinth or lintel beams framing into the columns, RC wall or masonry wall adjoining column and extending only for partial column height

8.5 Columns carrying relatively small lateral forces are termed as gravity columns. Such columns supporting discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height (*see* Fig. 14).This reinforcement shall 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; also it shall be provided below the discontinuity for the same development length.

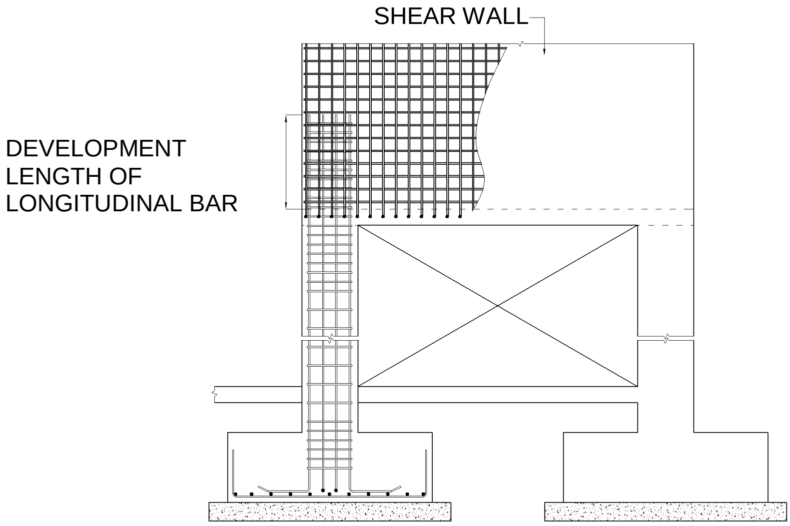

FIG. 14 COLUMNS WITH VARIABLE STIFFNESS

9. BEAM-COLUMN JOINTS OF MOMENT-RESISTING FRAMES

9.1 Design of Beam-Column Joint for Distortional Shear

9.1.1 *Shear Strength of Concrete in a Joint*

The nominal shear strength τ_{jc} of concrete in a beam-column joint shall be taken as

$$\tau_{jc} = \begin{cases} 1.5 A_{ej}\sqrt{f_{ck}} & \text{for joints confined by beams on all four faces} \\ 1.2 A_{ej}\sqrt{f_{ck}} & \text{for joints confined by beams on three faces} \\ 1.0 A_{ej}\sqrt{f_{ck}} & \text{for other joints} \end{cases}$$

where A_{ej} is effective shear area of joint given by $b_j w_j$, in which b_j is the effective breadth of joint perpendicular to the direction of shear force and w_j the effective width of joint along the direction of shear force (*see* Fig. 15). In no case shall the area of joint be greater than the column cross-sectional area

9.1.2 *Design Shear Stress Demand on a Joint*

(a) Design Shear Stress Demand acting horizontally along each of the two principal plan directions of the joint shall be estimated from earthquake shaking considered along each of these directions, using

$$\tau_{jdX} = \frac{V_{djX}}{b_j w_j} \quad \text{for shaking along plan direction X of earthquake shaking}$$
$$\tau_{jdY} = \frac{V_{djY}}{b_j w_j} \quad \text{for shaking along plan direction Y of earthquake shaking},$$

It shall be ensured that the joint shear capacity of joint concrete estimated using **9.1.1** exceeds both τ_{jdX} and τ_{jdY}.

(b) Design Shear Force Demands V_{jdX} and V_{jdY} acting horizontally on the joint in principal plan directions X and Y shall be estimated considering that the longitudinal beam bars in tension reach a stress of $1.25f_y$ (when overstrength plastic moment hinges are formed at beam ends).

9.1.3 Width of Beam Column Joint

When beam reinforcement extends through beam-column joint, the minimum width of the column parallel to beam shall be 20 times the diameter of the largest longitudinal beam bar.

9.2 Transverse Reinforcement

9.2.1 *Confining Reinforcement in Joints*

(a) When all four vertical faces of the joint are having beams framing into them covering at least 75% of the width on each face,
   (i) At least half the special confining reinforcement required as per **8** at the two ends of columns, shall be provided through the joint within the depth of the shallowest beam framing into it, and
   (ii) Spacing of these transverse stirrups shall not exceed 150mm.

(b) When all four vertical faces of the joint are NOT having beams framing into them or when all four vertical faces have beams framing into them but do not cover at least 75% of the width on any face,

(i) Special confining reinforcement required as per **8** at the two ends of columns shall be provided through the joint within the depth of the shallowest beam framing into it, and

(ii) Spacing of these transverse stirrups shall not exceed 150mm.

9.2.2 In the exterior and corner joints, all 135° hooks of cross-ties should be along the outer face of columns.

10. SPECIALSTRUCTURAL WALLS

10.1 General Requirements

10.1.1 The requirements of this section apply to special structural walls that are part of lateral force resisting system of earthquake-resistant RC buildings.

10.1.2 The minimum thickness of special structural walls shall not be less than
(a) 150 mm
(b) 300 mm for buildings with coupled structural walls in any Seismic Zone.

10.1.3 Special structural walls shall be classified as squat, intermediate or slender depending on the overall height h_w to Length L_w ratio as
(a) Squat walls: $h_w / L_w < 1$,
(b) Intermediate walls: $1 \leq h_w / L_w \leq 2$, and
(c) Slender walls: $h_w / L_w > 2$.

10.1.4 In the design of flanged wall sections, only that part of the flange shall be considered which extends beyond the face of the web of the structural wall at least for a distance equal to smaller of
(a) Actual width available,
(b) Half the distance to the adjacent structural wall web, and
(c) 1/10th of the total wall height.

10.1.5 Special structural walls shall be provided with uniformly spaced reinforcement in its cross-section along vertical and horizontal directions. At least a minimum area of reinforcement bars as indicated in Table 1 shall be provided along vertical and horizontal directions.

10.1.6 Reinforcement bars shall be provided in two curtains within the cross-section of the wall, with each curtain having bars running along vertical and horizontal directions, when
(a) Factored shear stress demand in the wall exceeds $0.25\sqrt{f_{ck}}$ MPa, or
(b) Wall thickness is 200 mm or higher.
When steel is provided in two layers, all vertical steel bars shall be contained within the horizontal steel bars; the horizontal bars shall form a closed core concrete area with closed loops and cross-ties.

10.1.7 The largest diameter of longitudinal steel bars used in any part of a wall shall not exceed 1/10th of the thickness of that part.

10.1.8 The maximum spacing of vertical or horizontal reinforcement shall not exceed smaller of
(a) 1/5th horizontal length L_w of wall,
(b) 3 times thickness t_w of web of wall, and
(c) 450 mm.

10.1.9 Special structural walls shall be founded on properly designed foundations and shall not be discontinued to rest on beams, columns or inclined members.

Table 1: Minimum Reinforcement in RC Structural Walls

Type of Wall	*Reinforcement Details*
Squat Walls	$(\rho_h)_{min} = 0.0025$ $(\rho_v)_{min} = 0.0025 + 0.5\left(1 - \frac{h_w}{t_w}\right)(\rho_h - 0.0025)$ $(\rho_{v,net}) = (\rho_{v,web}) + \left(\frac{t_w}{L_w}\right).[0.02 - 2.5(\rho_{v,web})]$ $(\rho_v)_{provided} < (\rho_h)_{provided}$
Intermediate Walls	$(\rho_h)_{min} = 0.0025$ $(\rho_{v,be})_{min} = 0.0080$ $(\rho_{v,web})_{min} = 0.0025$ $(\rho_{v,net})_{\min} = 0.0025 + 0.01375\left(\frac{t_w}{L_w}\right).$
Slender Walls	$(\rho_h)_{\min} = 0.0025 + 0.5\left(\frac{h_w}{L_w} - 2\right)(\rho_h - 0.0025)$ $(\rho_{v,be})_{min} = 0.0080$ $(\rho_{v,web})_{min} = 0.0025$ $(\rho_{v,net})_{\min} = 0.0025 + 0.01375\left(\frac{t_w}{L_w}\right).$

10.2 Design for Shear Force

10.2.1 Nominal shear stress demand τ_v on a wall shall be estimated as:

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

where V_u is factored shear force, t_w thickness of the web, and d_w effective depth of wall section (along the length of the wall), which may be taken as $0.8L_w$ for rectangular sections.

10.2.2 Design shear strength τ_c of concrete shall be calculated as per Table 19 of IS 456.

10.2.3 When nominal shear stress demand τ_v on a wall is

(a) more than maximum design shear strength $\tau_{c,\max}$ of concrete (given in Table 20 of IS 456), the wall section shall be redesigned;

(b) less than maximum design shear strength $\tau_{c,\max}$ of concrete and more than design shear strength τ_c of concrete, design horizontal shear reinforcement shall be provided of area A_h given by

$$A_h = \frac{V_{us}}{0.87 f_y \left(\frac{d}{s_v}\right)_{Integral}} = \frac{V_u - \tau_c t_w d_w}{0.87 f_y \left(\frac{d}{s_v}\right)_{Integral}},$$

which shall not be less than the minimum area of horizontal steel per Clause 10.1.5; and

(c) less than design shear strength τ_c of concrete, horizontal shear reinforcement shall be the minimum area of horizontal steel per **10.1.5**.

10.3 Design for Axial Force and Bending Moment

10.3.1 Design moment of resistance M_u of the wall section subjected to combined bending moment and compressive axial load shall be estimated in accordance with requirements of limit state design method given in IS 456, using the principles of mechanics involving equilibrium equations, strain compatibility conditions and constitutive laws.

The moment of resistance of slender rectangular structural wall section with uniformly distributed vertical reinforcement may be estimated using expressions given in Annex A. Expressions given in Annex A are not applicable for structural walls with boundary elements,

10.3.2 The cracked flexural strength of a wall section shall be greater than its uncracked flexural strength.

10.3.3 In structural walls that do not have boundary elements, at least a minimum of 4 bars of 12 mm diameter arranged in 2 layers, shall be concentrated as vertical reinforcement at the ends of the wall over a length not exceeding twice the thickness of RC wall.

10.4 Boundary Elements

Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement even if they have the same thickness as that of the wall web. It is advantageous to provide boundary elements with dimension greater than thickness of the wall web.

10.4.1 Boundary elements shall be provided along the vertical boundaries of walls, when the extreme fiber compressive stress in the wall exceeds $0.2f_{ck}$ due to factored gravity loads plus factored earthquake force. Boundary elements may be discontinued at elevations where extreme fiber compressive stress becomes less than $0.15f_{ck}$. Extreme fiber compressive stress shall be estimated using a linearly elastic model and gross section properties.

10.4.2 A boundary element shall have adequate axial load carrying capacity, assuming short column action, so as to enable it to carry axial compression arising from factored gravity load and lateral seismic shaking effects.

10.4.2.1 The load factor for gravity load shall be taken as 0.8, if gravity load gives higher axial compressive strength of the boundary element.

10.4.3 The vertical reinforcement in the boundary elements shall not be less than 0.8% and not greater than 6 %; the practical upper limit would be 4 % to avoid congestion.

10.4.4 Boundary elements, where required as per **10.4.1**, shall be provided with special confining reinforcement throughout their height, given by

$$A_{sh} = 0.05 s_v h \frac{f_{ck}}{f_y}.$$

10.4.6 Boundary elements need not be provided, if the entire wall section is provided with special confining reinforcement, as per **8**.

10.5 Coupling Beams

10.5.1 Coplanar Special Structural walls may be connected by means coupling beams.

10.5.2 If earthquake induced shear stress τ_{ve} in coupling beam exceeds

$$\tau_{ve} > 0.1\sqrt{f_{ck}}\left(\frac{L_s}{D}\right),$$

where L_s is clear span of coupling beam and D overall depth, the entire earthquake-induced shear, bending moment and axial compression shall be resisted by diagonal reinforcement alone.

(a) Area of this diagonal reinforcement along each diagonal shall be estimated as:

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

where V_u is factored shear force on the coupling beam and α the angle made by diagonal reinforcement with the horizontal.

(b) At least 4 bars of 8 mm diameter shall be provided along each diagonal. All longitudinal bars along each diagonal shall be enclosed by special confining transverse reinforcement as per Clause 8 at a spacing not exceeding 100 mm.

10.5.3 The diagonal of a coupling beam shall be anchored in the adjacent walls with an anchorage length of 1.5 times the development length in tension. (*see* Fig. 15)

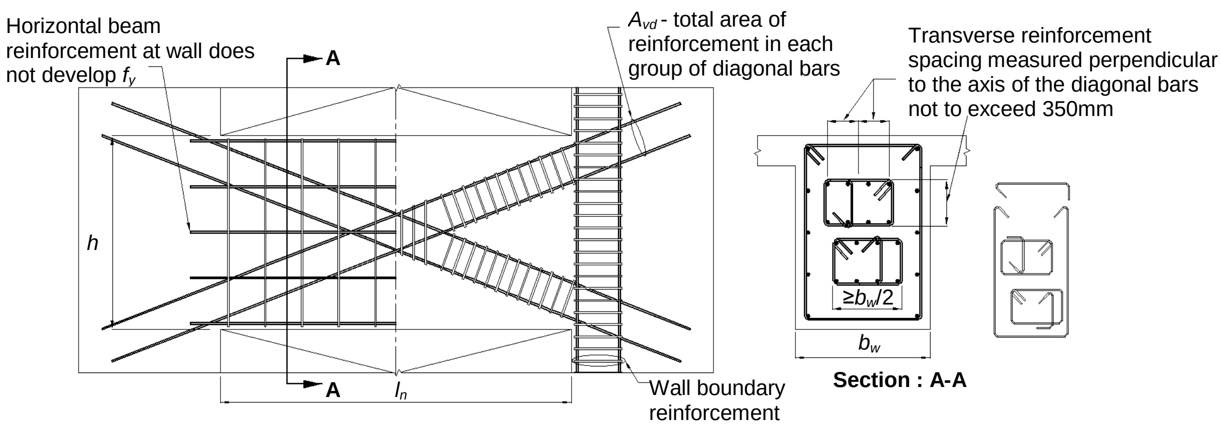

FIG. 15 COUPLING BEAMS WITH DIAGONAL REINFORCEMENT

10.6 Openings in Walls

10.6.1 Shear strength of a wall with openings should be checked at critical horizontal planes passing through openings.

10.6.2 Additional steel reinforcement shall be provided along all four edges of openings in walls.

(a) The area of these vertical and horizontal steel should be equal to that of the respective interrupted bars, provided half on either side of the wall in each direction.
(b) These vertical bars should extend for full height of the storey in which this opening is present.
(c) The horizontal bars should be provided with development length in tension beyond the edge of the opening.

10.7 Construction Joints

Vertical reinforcement across a horizontal construction joint shall have area A_{st} given by:

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

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

10.8 Development, Splice and Anchorage Requirement

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

10.8.2 In slender walls (H/L_w > 2), splicing of vertical flexural reinforcement should be avoided, as far as possible, in regions where flexural yielding may take place, which extends for a distance larger of
(a) L_w above the base of the wall, and
(b) 1/6th of the wall height,
but not larger than $2L_w$.

10.8.3 Splices

10.8..3.1 Lap Splices

When adopted, closed stirrups shall be provided over the entire length over which the longitudinal bars are spliced.

(a) The spacing of these stirrups shall not exceed 150 mm.
(b) The lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.
(c) Lap splices shall be provided only in the central half of clear wall height, and not
 (i) within a joint, or
 (ii) within a distance of 2*d* from face of the beam.
(d) Not more than 50% of area of steel bars shall be spliced at any one section.

10.8.3.2 Mechanical couplers [conforming to IS xxxxx (*under print*)] shall be used. Further, only those mechanical splices conforming to the above standard and capable of developing the specified tensile strength of spliced bar shall be permitted within a distance equal to two times the depth of the member from the beam-column joint or in any location where yielding of reinforcement is likely to take place.

10.8.3.4 *Welded Splices*

Welded splices shall be avoided as far as possible. In no case shall they be used for a distance equal to two times the depth of the member from the member face or in any location where yielding of reinforcement is likely to take place. At any location, not more than 50% of area of steel bars shall be spliced at any one section.

Welding of stirrups, ties, inserts or other similar elements to vertical reinforcement bars required as per design is not permitted, in any seismic zone.

10.8.4 In buildings located in Seismic Zones II and III, closed loop transverse stirrups shall be provided around lapped spliced bars larger than 16 mm in diameter. The minimum diameter of such stirrups shall be $1/4^{th}$ of diameter of spliced bar but not less than 8 mm at spacing not exceeding 150 mm centers.

11. GRAVITY COLUMNS IN BUILDINGS

Gravity columns in buildings shall be detailed according to Clauses **11.1** and **11.2** for bending moments induced when subjected to twice the design lateral displacement under the factored equivalent static design seismic loads given by IS 1893 (Part 1).

11.1 Clauses **11.1.1** and **11.1.2** shall be satisfied, when induced bending moments and horizontal shear forces under the said lateral displacement combined with factored gravity bending moment and shear force *do not* exceed the design moment of resistance and design lateral shear capacity of the column.

11.1.1 Gravity columns shall satisfy **7.3.2**, **7.4.1** and **7.4.2.** But, spacing of stirrups along the full column height shall not exceed 6 times diameter of smallest longitudinal bar or 150 mm.

11.1.2 Gravity columns with factored gravity axial stress exceeding 0.4 f_{ck} shall satisfy **11.1.1** and shall have transverse reinforcement at least one half of special confining reinforcement required by **8**.

11.2 When induced bending moments and shear forces under said lateral displacement combined with factored gravity bending moment and shear force exceed design moment and shear strength of the frame, **11.2.1** and **11.2.2** shall be satisfied.

11.2.1 Mechanical and welded splices shall satisfy **7.3.2.2** and **7.3.2.3.**

11.2.2 Gravity columns shall satisfy **7.4** and **8**.

ANNEX A
(*Clause* 10.3.1)

A.1 MOMENT OF RESISTANCE OF RECTANGULAR SHEAR WALL SECTION

The moment of resistance M_u of a slender rectangular structural wall section with uniformly distributed vertical reinforcement may be estimated as:

(a) For $(x_u/l_w) < (x_u^*/l_w)$

$$\frac{M_u}{f_{ck}t_w L_w^2} = \varphi\left[\left(1+\frac{\lambda}{\varphi}\right)\left(\frac{1}{2}-0.416\frac{x_u}{L_w}\right)-\left(\frac{x_u}{L_w}\right)^2\left(0.168+\frac{\beta^2}{3}\right)\right]$$

where

$$\frac{x_u}{L_w} = \left(\frac{\varphi+\lambda}{2\varphi+0.36}\right);$$

$$\frac{x_u^*}{L_w} = \frac{0.0035}{0.0035+\left(0.002+0.87f_y/E_s\right)};$$

$$\varphi = \left(\frac{0.87f_y\rho}{f_{ck}}\right);$$

$$\lambda = \left(\frac{P_u}{f_{ck}t_w L_w}\right);$$

ρ = vertical reinforcement ratio = $\left(\dfrac{A_{st}}{t_w L_w}\right)$,

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

$$\beta = \frac{\left(0.002+0.87f_y/E_s\right)}{0.0035},$$

E_s = elastic modulus of steel, and

P_u = factored compressive axial force on wall.

(b) For $(x_u^*/L_w) < (x_u/L_w) < 1.0$

$$\frac{M_u}{f_{ck}t_w L_w^2} = \left(\frac{x_u}{L_w}\right)-\alpha_2\left(\frac{x_u}{L_w}\right)^2-\alpha_3-\frac{\lambda}{2}$$

where

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

$$\alpha_2 = \left[0.15+\frac{\varphi}{2}\left(1-\beta+\frac{\beta^2}{3}-\frac{1}{3\beta}\right)\right] \text{ and}$$

$$\alpha_3 = \frac{\varphi}{6\beta}\left(\frac{1}{x_u/L_w}-3\right).$$

x_u/L_w to be used in this expression shall be obtained by solving the equation:

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

where

$\alpha_4 = \left(\frac{\varphi}{\beta} - \lambda\right)$ and

$\alpha_5 = \left(\frac{\varphi}{2\beta}\right)$.

....