भारतीय मानक ब्यूरो
(उपभोक्ता मामले, खाद्य एवं सार्वजनिक वितरण मंत्रालय, भारत सरकार)
BUREAU OF INDIAN STANDARDS
(Ministry of Consumer Affairs, Food & Public Distribution, Govt. of India)

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व्यापक परिचालन मसौदा

हमारा संदर्भ : सीईडी 39/टी-32 **04 जून 2024**

तकनीकी समिति : भूकंप इंजीनियरिंग अनुभागीय समिति , सीईडी 39

प्राप्तकर्ता :

1. सिविल अभियांत्रिकी विभाग परिषद, सीईडीसी के सभी सदस्य
2. भूकंप इंजीनियरिंग अनुभागीय समिति, सीईडी 39 के सभी सदस्य
3. सीईडी 39 की उपसमितियों और अन्य कार्यदल के सभी सदस्य
4. रुचि रखने वाले अन्य निकाय।

महोदय/महोदया,

निम्नलिखित मानक का मसौदा संलग्न है:

प्रलेख संख्या	शीर्षक
सीईडी 39(25408)WC	**संरचनाओं का भूकंप प्रतिरोधी डिज़ाइन और विवरण — रीति संहिता भाग 2 भवन का भारतीय मानक मसौदा** (IS 13920 का *दूसरा पुनराभ्यास*) (आईसीएस 91.120.25)

कृपया इस मसौदे का अवलोकन करें और अपनी सम्मतियाँ यह बताते हुए भेजे कि यह मसौदा प्रकाशित हो तो इन पर अमल करने में आपको व्यवसाय अथवा कारोबार में क्या कठिनाइयां आ सकती हैं।

सम्मतियाँ भेजने की अंतिम तिथि: 03 अगस्त 2024

सम्मति यदि कोई हो तो कृपया अधोहस्ताक्षरी को ई-मेल द्वारा **ced39@bis.gov.in** पर या उपरलिखित पते पर, संलग्न फोर्मेट में भेजें। सम्मतियाँ बीआईएस ई-गवर्नेंस पोर्टल, **www.manakonline.in** के माध्यम से ऑनलाइन भी भेजी जा सकती हैं।

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यह प्रालेख भारतीय मानक ब्यूरो की वेबसाइट www.bis.gov.in पर भी उपलब्ध हैं।

धन्यवाद।

भवदीय
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(द्वैपायन भद्र)
वैज्ञानिक ई एवं प्रमुख
सिविल अभियांत्रिकी विभाग

संलग्न: उपरऽलिखित

भारतीय मानक ब्यूरो
(उपभोक्ता मामले, खाद्य एवं सार्वजनिक वितरण मंत्रालय, भारत सरकार)
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WIDE CIRCULATION DRAFT

Our Reference: CED 39/T- 32 **04 June 2024**

TECHNICAL COMMITTEE: EARTHQUAKE ENGINEERING SECTIONAL COMMITTEE, CED 39

ADDRESSED TO:

1. All Members of Civil Engineering Division Council, CEDC
2. All Members of Earthquake Engineering Sectional Committee, CED 39
3. All Members of Subcommittees, Panels and Working Groups under CED 39
4. All others interested.

Dear Sir/Madam,

Please find enclosed the following draft:

Doc No.	Title
CED 39(25408)WC	**Draft Indian Standard** **Earthquake Resistant Design and Detailing of Structures — Code of Practice** Part 2 Buildings (*Second Revision* of IS 13920) ICS 91.120.25

Kindly examine the attached draft 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: 03 August 2024

Comments if any, may please be made in the enclosed format and emailed at ced39@bis.gov.in or sent at the above address. Additionally, comments may be sent online through the BIS e-governance portal, www.manakonline.in.

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

The document is also hosted on the BIS website www.bis.gov.in.

Thanking you,

Yours faithfully,
Sd/-
Dwaipayan Bhadra
Scientist 'E' & Head
Civil Engineering Department

Encl: As above

FORMAT FOR SENDING COMMENTS ON THE DOCUMENT

[Please use A4 size sheet of paper only and type within fields indicated. Comments on each clause/sub-clause/ table/Fig., etc, be stated on a fresh row. Information/comments should include reasons for comments, technical references and suggestions for modified wordings of the clause. **Comments through e-mail to** ced39@bis.gov.in **shall be appreciated**.]

Doc. No.: CED 39(25408)WC

BIS Letter Ref: CED 39/T-32

Title: Draft Indian Standard Earthquake Resistant Design and Detailing of Structures — Code of Practice
Part 2 Buildings (Second Revision of IS 13920) ICS 91.120.25

Last date of comments: 03 August 2024

Name of the Commentator/ Organization: ______________________________________

S. No.	Clause/ Para/ Table/ Fig. No. commented	Type of Comment (General/ Technical/ Editorial)	Comments/ Modified Wordings	Justification of Proposed Change

NOTE- Kindly insert more rows as necessary for each clause/table, etc

BUREAU OF INDIAN STANDARDS

DRAFT STANDARD FOR COMMENTS ONLY
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Draft Indian Standard
EARTHQUAKE RESISTANT DESIGN AND DETAILING OF STRUCTURES — CODE OF PRACTICE
PART 2 BUILDINGS

(*Second Revision* of IS 13920)

Earthquake Engineering Sectional Committee, CED 39	Last Date for Comments: **03 August 2024**

FOREWORD

(*Formal Clauses will be added later*).

This standard should be read in conjunction with CED 39 (22343) and CED 39 (22345), which provides design earthquake hazard and general provisions for earthquake resistant design of structures, and criteria for earthquake resistant design of buildings.

IS 13920 was first published in 1993, and revised in 2016. In 2022, the Committee decided to present the provisions for different types of structures in separate parts, to keep abreast with rapid developments and extensive research carried out in earthquake-resistant design of various structures. Thus, IS 13920 is split into 11 parts, namely:

Part 1: General Provisions,
Part 3: Liquid Retaining Tanks (*to be formulated*);
Part 4: Bridges and Retaining Walls (*to be formulated*);
Part 5: Industrial Structures (*to be formulated*);
Part 6: Base Isolated Buildings (*to be formulated*);
Part 7: Pipelines (*to be formulated*);
Part 8: Dams and Embankments (*to be formulated*);
Part 9: Coastal Structures (*to be formulated*);
Part 10: Steel Towers (*to be formulated*); and
Part 11: Tunnels (*to be formulated*).

CED 39 (25407) contains general provisions on earthquake resistant design and detailing applicable to all structures. This standard provides additional provisions for earthquake resistant design and detailing applicable to all buildings. Also, the provisions of building in this standard are presented in four sections, namely.

In this second revision, the following major changes have been included:

Section 1: Additional Criteria for All Structures

1) Provisions applicable for design of masonry, reinforced concrete and steel building have been separated.

Section 2: Additional Criteria for Masonry Buildings

1) Provisions on buildings have been harmonized from IS 4326 and IS 15988; and
2) The admissibility of different structural systems in the earthquake zones is clarified.
3) Guidelines (as given in IS 13827 and IS 13828) for earthquake resistance related to construction using low strength masonry and earthen buildings are included.

Section 3: Additional Criteria for Concrete Buildings

1) The admissibility of different structural systems in different earthquake zones is clarified; and
2) The structural plan density of the structural walls is varied with the earthquake zone and category of the building.

Section 4: Additional Criteria for Steel Buildings

1) New provisions have been prepared for steel buildings including admissibility of different structural systems in different earthquake zones.

In the formulation of this standard, effort has been made to coordinate with standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country. Assistance has particularly been derived from the following publications:

1) IBC, (2021), International Building Code, International Code Council, USA, 2021;
2) ACI 318-19(22), (2022), *Building Code Requirements for Structural Concrete and Commentary*, American Concrete Institute, Chicago, IL, USA;
3) NZS 3101 (Part 1), (2006), *Concrete Structures Standard*, Standards New Zealand, Ministry of Business, Innovation & Employment, Wellington, NZ;
4) EN 1998, (2005), *Eurocode 8: Design of structures for earthquake resistance*, European Committee for Standardization, Brussels

The composition of the Committee including the Drafting Group responsible for the formulation of this standard is given in Annex B.

This standard contributes to the following Sustainable Development Goal: Goal 9 ‘Industry, Innovation and Infrastructure’ towards building resilient infrastructure; promote inclusive and sustainable industrialization and faster innovation; and Goal 11 ‘Sustainable Cities and Communities’ towards making cities and human settlements inclusive, safe, resilient and sustainable.

For 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: 2022 'Rules for rounding off numerical values (second revision)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

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Draft Indian Standard
EARTHQUAKE RESISTANT DESIGN AND DETAILING OF STRUCTURES — CODE OF PRACTICE
PART 2 BUILDINGS
(*Second Revision* of IS 13920)

SECTION 1

PROVISIONS FOR ALL BUILDINGS

1 SCOPE

1.1 This standard provides requirements for designing and detailing of members and joints of buildings to resist lateral effects of earthquake shaking, to impart in them adequate stiffness, strength, and in particular, ductility capacity to resist severe earthquake shaking without collapse.

1.2 This standard addresses designing and detailing of structural systems of the following types of buildings:
a) Masonry buildings,
b) Reinforced concrete buildings, and
c) Steel buildings;

1.3 This standard provides additional specifications for designing and detailing of structural members, which are made of different materials, like masonry, concrete and structural steel, to make them capable of resisting the effects of earthquake shaking. These provisions shall be applied over and above those specified already in the respective standards for their design, namely IS 1905 for masonry structures, IS 456 for concrete structures and IS 800 for steel structures.

1.4 Cold-formed light gauge steel structural members (as per IS 801) shall not be used as part of the earthquake load resisting system.

1.5 The provisions of this standard are applicable for all buildings listed in **1.2**. Further, for Critical & Lifeline Structures, Special Structures, and Nuclear Power Plant Structures, additional requirements may be imposed by the associated statutory authorities in India. In such cases, the requirements specified by this standard shall be taken as at least the minimum that should be met with.

1.6 The Architectural Elements and Utilities (AEUs) of buildings, which are supported on the lateral force resisting system, shall be designed to resist earthquake effects.

1.7 Masonry Building

1.7.1 Section 2 of this standard addresses earthquake resistant design and detailing masonry buildings having the following structural systems:

a) Masonry buildings with bands and prescriptive horizontal and vertical reinforcement, following guidelines on the size and position of opening specified in **8.2**.
b) Confined masonry buildings with the load-bearing walls having reinforced concrete horizontal (that is, tie-beams) and vertical (that is, tie-columns); and
c) Reinforced masonry buildings with load-bearing masonry walls provided with vertical and horizontal reinforcement.

1.7.2 Provisions of section 2 shall be adopted in structural systems of masonry buildings (identified in **1.7.1**) as admissible in different earthquake zones as per Table 8 of CED 39 (22343) and adopting design return periods as specified in Table 5 of CED 39 (22343) for Building Sets 1, 2 and 3.

1.7.3 The structural design of reinforced masonry buildings with vertical and horizontal reinforcement and the design of confined masonry buildings are based on the Working Stress Method.

1.8 Reinforced Concrete Buildings

1.8.1 Section 3 of this standard provides the requirements for designing and detailing members of concrete buildings to resist lateral effects of earthquake shaking, so as to give them adequate stiffness, strength and ductility to resist severe earthquake shaking without collapse.

1.8.2 This standard addresses earthquake resistant design and detailing of RC buildings having the following structural systems:
a) RC Special Moment Resisting Frames,
b) RC Special Moment Resisting Frames with unreinforced masonry infill walls,
c) RC Special Moment Resisting Frames with RC Special Structural Walls, and
d) RC Special Structural Walls.

1.8.3 Provisions of this standard shall be adopted in structural systems of RC buildings as admissible in different earthquake zones as per Table 9 of CED 39 (22343)

1.8.4 The provisions for RC buildings given herein shall apply to monolithic RC construction, which have ductility to resist the effects of strong earthquake shaking. When a concrete structural system comprising of precast concrete components are used, it shall be demonstrated through full-scale testing that the structural system possesses the lateral stiffness, strength, and deformability capacities as that of equivalent monolithic RC structural system.

1.8.5 Monolithically cast concrete structures shall be designed based on results of structural analysis using a single 3-dimensional mathematical model. And in the model, moment continuity b/w beams and columns shall not be released.

1.8.6 Along one plan direction of a building having RC MRFs plus SWs along one direction and only MRFs along the other plan direction, or only RC MRFs along each of the two plan directions, in different parallel vertical planes,
a) The planar MRFs plus SWs or only MRFs can be selected judiciously (based on their relative stiffness and location in the building) and designed to resist together 100 percent of the effects of design vertical loads appearing on each of them and 100 percent of the effects of total design earthquake lateral load of the building, and

b) The remaining planar MRFs shall be designed to resist the effects imposed on them due to vertical loads and the deformation compatibility induced effects of earthquake shaking.

Also:

i) All MRFs plus SWs along each plan direction shall be selected [in (1) above] to carry the design lateral loads; only when MRFs appear without SWs in one plane, they can be exempted from being designed for the lateral loads; and

ii) When the design horizontal acceleration coefficient A_h of the building is less than 0.05, the above provision does not apply.

1.9 Steel Building

1.9.1 Section 4 of this standard covers the requirements for designing and detailing of structural components and members of the following steel buildings:

a) Residential buildings,
b) Office and commercial buildings, and
c) Community, utility and lifeline buildings required for disaster management activities,

which are designed to resist lateral effects of earthquake shaking, so as to provide them with adequate stiffness, strength and ductility to resist severe earthquake shaking without collapse.

The general concepts adopted in this standard for buildings are applicable also for other types of structures; in particular, provisions of this standard may be taken as a guide for design of components of industrial structures.

1.9.2 This standard addresses earthquake resistant design and detailing steel buildings having the following structural systems:

a) Special moment resisting frame (SMRF);
b) Special concentrically braced frame (SCBF); and
c) Eccentrically braced frame (EBF).

1.9.3 Provisions of this standard shall be adopted in structural systems of steel buildings (identified in **1.2**) as admissible in different in earthquake zones as per Table 11 of CED 39 (22345).

1.9.4 All SMRFs (and their elements) in a building need not be designed to resist effects of earthquake shaking. The designer can identify judiciously select planar frames (based on relative stiffness and location in the building), to resist together the vertical loads and at least 80 percent of the effects of design earthquake lateral load, and designate them as part of the lateral load resisting system and design them for resisting full effects of earthquake shaking. The other frames and structural members not part of the lateral force resisting system (that is, which are assumed not to participate in resisting effects of earthquake shaking) shall be designed to resist the effects imposed on them due to vertical loads and the deformation compatibility induced effects of earthquake shaking.

1.9.5 Moments shall not be transferred to the designated lateral load resisting systems from frames spanning in perpendicular direction.

2 REFERENCES

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

3 TERMINOLOGY

For this standard, the following definitions shall apply.

3.1 All Buildings

For buildings referred to in this standard, the following definitions shall apply.

3.1.1 *Beam* — A member (generally horizontal) that resists load through bending and shearing actions.

3.1.2 *Boundary Elements* — Strengthened portions of the ends of a structural wall.

3.1.3 *Column* — A member (generally vertical) that resists load through axial, bending and shearing actions.

3.1.4 *Capacity Design* — The design process beyond those in the basic design of non-yielding structural elements (members and connections), which consider the effects of inelasticity incurred in yielding members resulting in overstrength-based demands on the non-yielding structural elements.

3.1.5 *Capacity Protected Element* — A member designed to remain elastic using capacity design principles when an adjacent member undergoes inelastic straining during design earthquake shaking.

3.1.6 *Gravity Columns* — A column, which is not a part of the designated lateral load resisting system, and designed to resist:

a) Force actions (that is, axial force, shear force and bending moments) due to gravity loads, and
b) Effects arising from displacement compatibility induced during earthquakes, through axial, flexural and shearing actions.

3.1.7 *Lateral Force Resisting System* — An arrangement of structural members, which resists lateral forces imposed on the structure.

3.1.8 *Moment Resisting Frame (MRF)* — An arrangement of interconnected beams and columns, without structural walls or and inclined members (braces), 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 axial, shearing and flexural actions. This is applicable to concrete and steel buildings.

3.1.8.1 *Ordinary moment resisting frame (OMRF)* — A moment-resisting frame designed and detailed as per requirements specified in IS 456 or IS 800.

3.1.8.2 *Special moment resisting frame (SMRF)* — A moment-resisting frame designed and detailed as per requirements specified in IS 456 or IS 800, and additional requirements to provide ductile behaviour specified in this standard.

3.1.9 *Structural Wall* — A planar element (generally in the vertical plane) provided along the full height of the building, which is designed primarily to resist lateral force effects (axial force, shear force and bending moment) in its own plane.

3.1.9.1 *Ordinary structural wall* — A structural wall designed and detailed as per requirements specified in IS 1905, IS 456 or IS 800, respectively.

3.1.9.2 *Special structural wall* — A structural wall designed and detailed as per requirements specified in IS 1905, IS 456 or IS 800, respectively, and additional requirements to provide ductile behaviour specified in this standard.

3.1.10 *Curvature Ductility* **—** The ratio of curvature at the ultimate deformability of the section and that at first flexural yield in the section.

3.1.11 *Space Frame* — A three-dimensional structural system composed of interconnected members, without structural walls, so as to function as a complete self-contained unit.

3.2 Masonry Buildings

For masonry buildings referred to in this standard, the following additional definitions shall apply.

3.2.1. *Separation Section* — The gap of specified width between adjacent buildings or parts of the same building either left uncovered or covered suitably to permit movement to avoid pounding due to earthquake.

3.2.1.1 *Crumple section* — *T*he separation gap filled with appropriate material that crumples or fractures in the event of an earthquake.

3.2.2 *Centre of Rigidity* — The point in a structure where application of lateral force produces equal deflections of its components at any level in a particular direction.

3.2.3 *Load Bearing Wall* — A wall designed to resist axial force, shear force in its own plane and bending moment about its major axis.

3.2.4 *Box System* — A building made of masonry load bearing walls and horizontal floors.

3.2.5 *Band* — A wooden (in low strength masonry buildings), reinforced concrete or reinforced brick runner provided in the walls to tie them together and to impart horizontal bending strength in them.

3.2.6 *Earthquake Zone* — The earthquake zones as classified in **6.2.1** of CED 39 (22343).

3.2.7 *Design Horizontal Acceleration Coefficient* — *T*he Horizontal Acceleration Coefficient A_h computed considering the *Importance Factor* and *Soil-Foundation System* as specified in **5.2.2.1(a)** of CED 39 (22343).

3.2.8 *Concrete Strength* — The 28-day compressive strength (in MPa) of concrete cubes of 150 mm size.

3.2.9 *Cross-Sectional Area of Masonry Unit* — The net cross-sectional area of a masonry unit shall be taken as the gross cross-sectional area minus the area of cellular space (if any). Gross cross-sectional area of cored units shall be determined to the outside of the coring, but cross-sectional area of grooves (if any) shall not be deducted from the gross cross-sectional area to obtain the net cross-sectional area.

3.2.10 *Grout* — A mixture of cement, sand, and water of pourable consistency for filling small voids.

3.2.11 *Grouted Masonry* — Masonry in which the masonry units with holes are filled with a mixture of cement and sand.

3.2.11.1 *Grouted hollow-unit masonry* — A form of grouted masonry construction in which certain designated cells of hollow units are continuously filled with grout.

3.2.11.2 *Grouted multi-wythe masonry* — A form of grouted masonry construction in which the space between the wythes is solidly or periodically filled with grout.

3.2.12 *Joint Reinforcement* — A prefabricated reinforcement in the form of lattice truss which has been hot dip galvanized after fabrication and is to be laid in the mortar bed joint.

3.2.13 Pier — An isolated vertical member whose horizontal dimension measured at right angles to its thickness is not less than 4 times its thickness and whose height is less than 5 times its length.

3.2.14 *Prism* — An assemblage of masonry units bonded by mortar with or without grout used as a test specimen for determining properties of masonry.

3.2.15 *Reinforced Masonry* — Masonry which is reinforced (as specified in Section 2) and grouted (if necessary) so that the two materials act together in resisting forces.

3.2.16 *Grouted Cavity Reinforced Masonry* — Masonry with two parallel single-leaf walls spaced at least 50 mm apart, and effectively tied together with wall ties. The intervening cavity contains steel reinforcement and is filled with infill concrete so as to result in common action with masonry under load.

3.2.17 *Pocket type Reinforced Masonry* — Masonry reinforced primarily to resist lateral loading where the main reinforcement is concentrated in vertical pockets formed in the tension face of the masonry and is surrounded by in situ concrete.

3.2.18 *Quetta Bond Reinforced Masonry* — Masonry that is at least one and half units thick, in which vertical pockets containing reinforcement and mortar or concrete infill occur at intervals along its length.

3.2.19 *Specified Compressive Strength of Masonry* — Minimum Compressive strength (in MPa) is force per unit of net cross-section area, which is required of the masonry used in construction and upon which the design is based.

3.2.20 *Wall Tie* — A metal fastener, which connects wythes of masonry to each other or to other materials.

3.2.21 *Wythe* — A continuous vertical layer of masonry wall of one unit in thickness.

3.3 Reinforced Concrete Buildings

For concrete buildings referred to in this standard, the following additional definitions shall apply.

3.3.1 *Boundary Elements* — Portions along the edges of a structural wall that are strengthened by longitudinal and transverse reinforcement. They can be concealed within the thickness of the wall. But, it is advantageous to provide boundary elements with width greater than thickness of the wall web.

3.3.2 *Cover Concrete* — Concrete which is not confined by transverse reinforcement.

3.3.3 *Crosstie* — 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.

3.3.4 *Hoop* — A closed stirrup having a 135° hook with a 10-diameter extension (but not less than 75 mm) at each end that is embedded in the confined core of the section. Also, it may be made of two pieces of reinforcement; a V-stirrup with a 135° hook and a 10-diameter extension (but not less than 75 mm) at each end, embedded in the confined core and a crosstie.

3.3.5 *Link* — 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.

3.3.6 *Structural Wall* — A plate-like vertical member of a building that resists axial load, bending moment and shear force arising out of effects of the gravity loads and earthquake shaking effects. The structural wall shall have the bending moment diagram about its major axis along the height like that of a vertical cantilever, with small jumps at the floor levels; the bending moment at the roof level need not be zero, owing to the presence of a beams at that level.

When the bending moment diagram of the vertical member about its major axis or minor along the height changes sign within each storey, then it shall be treated like a column.

3.3.7 *Transverse Reinforcement* — A continuous bar having a 135° hook with an extension of 8 times diameter (but not < 65 mm) at one end and a hook not less than 90° with an extension of 8 times diameter (but not < 65 mm) at the other end. The hooks shall engage peripheral longitudinal bars. In general, the 90° hooks of two successive crossties engaging the same longitudinal bars shall be alternated end for end. Transverse reinforcement (also called hoops) in columns is typically called stirrups and that in beams is called cross-ties.

3.4 Steel Building

For the purpose of steel buildings referred to in this standard, the following additional definitions shall apply.

3.4.1 *Brace* — Member (generally inclined) resisting loads through axial actions.

3.4.2 *Collector* — Member that serves to transfer loads between diaphragms and the members of the vertical force-resisting elements of the lateral load resisting system.

3.4.3 *Column Base* — An assembly of columns, plates, connector weld or bolts, and anchor rods at the base columns designed to transmit forces from the steel superstructure to reinforced concrete pedestal or foundation.

3.4.5 *Concentrically Braced Frame (CBF)* — A lateral load resisting system composed of interconnected beams and columns with inclined members as braces, which function as a complete self-contained unit with or without the aid of horizontal diaphragms of floor bracing systems, in which the system resists gravity and lateral force effects primarily by axial actions

3.4.5.1 *Special concentrically braced frame (SCBF)* — A CBF specially designed and detailed to provide ductile behaviour as per the requirements specified in this standard.

3.4.6 *Continuity Plate (CP)* — A plate provided within the flanges of the column at the levels of the flanges of the beams framing in the direction of the web of the column.

3.4.7 *Demand Critical Weld* — A weld connecting two structural components which are part of lateral load resisting system and at least one of which is expected to undergo inelastic straining following yielding.

3.4.8 *Doubler Plate (DP)* — A plate provided parallel to the web of the column, and connected along its own perimeter to the web of the column and at some intermediate locations within itself when necessary.

3.4.9 *Eccentrically Braced Frame (EBF)* — A lateral load resisting system composed of interconnected beams and columns with inclined members as braces that has at least one end connected to a beam through a link with a defined eccentricity from another beam-to-brace connection, which functions as a complete self-contained unit with or without the aid of horizontal diaphragms of floor bracing systems, in which the system resist gravity and lateral force effects primarily by axial action in the braces and shearing and flexural actions in the links. It is specially designed and detailed to provide ductile behaviour as per the requirements specified in this standard.

3.4.10 *Joint Panel Zone (JPZ)* — A finite-sized area of the column (from top flange level to bottom flange level of the deepest beam) at the junction where the beams frame into the column.

3.4.11 *Link* — A segment of a beam that is located between the ends of the connections of two inclined braces in EBFs. The length of the link is defined as the clear distance between the ends of two diagonal braces.

3.4.12 *Material Strength Uncertainty Factor* — The ratio of expected stress (yield or ultimate) to the characteristic stress (yield or ultimate) of a material; greater than unity.

3.4.13 *Protected Zone* — Area of a member designed to undergo inelastic deformations under design earthquake effects, and is required to be devoid of additional attachments or discontinuities resulting from fabrication and erection procedures.

4 SYMBOLS

4.1 Masonry Building

For reinforced concrete buildings, the letters and symbols indicated below shall be referred.

A_c	Cross-sectional area of concrete excluding reinforcing steel
A_m	Net area of masonry
A_{op}	Area of opening
A_s	Area of longitudinal reinforcement
A_{sc}	Area of transverse reinforcement
A_T	Area of cross-section of confined masonry wall including tie-columns
D	Effective depth of the wall section
d_b	Diameter of ties that is, both tie column and tie beam
E_c	Modulus of elasticity of concrete
E_m	Modulus of elasticity of masonry
F	Correction factor depending upon aspect ratio
fa	Allowable axial compressive stress
f_{cc}	Permissible stress in concrete in direct compression
f_{ck}	Characteristic compressive strength of concrete cubes
F_d	Design loads
f_d	Design strength of material
f_m	Compressive strength of masonry
f_s	Permissible steel tensile stress
f_t	Flexural tensile strength of masonry normal to bed joints
f_y	Characteristic yield strength of the reinforcing steel
G_m	Shear modulus of masonry
H	Clear wall height between floors
H'	Unsupported wall height between horizontal bands
h_c	Dimension of tie-column or tie-beam
h_o	Clear floor height of tie-column
k_s	Stress reduction factor
L	Actual length, unsupported length, length centre-to-centre distance of the intersecting member
L_w	Length of web
M	Factored bending moment
M	Modular ratio
M_u	Moment of resistance of confined masonry wall
M_{uf}	Bending moment corresponding to pure bending load condition
P	Factored gravity load
P_o	Permissible axial compressive force in confined masonry
P_c	Factored gravity load contributed by column
P_d	Design compressive axial load
P_m	Factored gravity load contributed by masonry wall
P_u	Ultimate axial compressive force in confined masonry
s	Spacing of ties
S_m	Section modulus of wall section
t	Thickness of wall
V	Factored applied shear force

v_a	Allowable masonry shear strength
v_m	Masonry shear strength
V_u	Shear resistance of confined masonry wall
WI_{floor}	Wall index per floor
ε_c	Maximum compressive strain in concrete
ε_m	Maximum compressive strain in masonry
γ_f	Partial safety factor for load
γ_m	Partial safety factor for material

4.2 Reinforced Concrete Buildings

For reinforced concrete buildings, the letters and symbols indicated below shall be referred.

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 link
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 link
d	Effective depth of member
$\mathrm{d_b}$	Diameter of longitudinal bar
d_w	Effective depth of wall section
E_s	Elastic modulus of steel
f_{ck}	Characteristic compressive strength (in MPa) of concrete cube
f_y	Yield stress (in MPa) of steel reinforcing bars, OR 0.2 percent proof strength (in MPa) of reinforcing steel
h	Longer dimension of rectangular confining link 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 / longer cross-section dimension 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 links 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 links
x_u x_u^*	Depth of neutral axis from extreme compression fiber
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

4.3 Steel Building

For steel buildings, the letters and symbols indicated below shall be referred.

A_f	Area of flange
A_g	Gross cross-sectional area
A_{gL}	Gross cross-sectional area of link
A_n	Net cross-sectional area
A_{st}	Area of stiffener
A_{wL}	Area of web of link
DL	Dead load as per IS 875 (Part 1)
E	Modulus of elasticity of Steel = 200 GPa
EL	Earthquake load as per IS 1893 (Part 1)
EL_m	Estimated maximum equivalent earthquake load induced in the structure
F_i	Lateral load acting on storey *i*
L_{br}	Unbraced length along the centerline of a member
LL	Live load as per IS 875 (Part 2)
H_i	Height of storey *i*
I_x	Second moment of area about major axis of bending
I_y	Second moment of area about minor axis of bending
K_{br}	Required shear stiffness of panel bracing system
M	Bending moment
M_{bo}	Estimated overstrength moment capacity of beam
M_{pb}	Plastic moment capacity of beam section
M_{pL}	Plastic bending moment capacity of link
M_{pc}	Plastic moment capacity of column section
P	Axial load
P_{br}	Required strength of connection of panel bracing system
P_d	Design compressive strength determined using IS 800
P_u	Maximum factored axial load demand
P_y	Yield axial load capacity
S_h	Factor to account for strain hardening and strain rate in links: 1.25 for I-shaped links and 1.4 for box shaped links
R_u	Ratio of the expected tensile stress to the characteristic tensile stress
R_y	Ratio of the expected yield stress to the characteristic yield stress
V_{br}	Required shear strength of panel bracing system
V_{pL}	Plastic shear capacity of link
V_{pzc}	Nominal shear force capacity of panel zone
V_{pzd}	Shear force demand on panel zone at the face of the column
W_i	Weight acting on storey *i*
Z_p	Plastic section modulus
Z_{pb}	Plastic section modulus of beam
Z_{pc}	Plastic section modulus of column
Z_{pL}	Plastic section modulus of link
b	Width
b_{bf}	Overall breadth of beam flange

b_{cf}	Overall breadth of column flange
d	Depth
d_f	Distance between centroids of flanges of a section under bending
d_b	Overall depth of beam
d_c	Depth of column
d_{pz}	Depth of panel zone
e	Length of link
h	Inter-storey height
f_y	Characteristic yield tensile stress of structural steel
f_{yb}	Characteristic yield tensile stress of structural steel used in beam
f_{yc}	Characteristic yield tensile stress of structural steel used in column
f_u	Characteristic ultimate tensile stress of structural steel
r_y	Radius of gyration about the weaker axis of bending
t	Thickness
t_{bf}	Thickness of beam flange
t_{cf}	Thickness of column flange
t_{pz}	Thickness of joint panel zone including the thickness of doubler plates if provided
w_{pz}	Width of panel zone
Ω	Overstrength factor of the building
α_i	Ratio of the secondary overturning moment to primary overturning moment at storey *i*
Δ_i	Lateral displacement of storey *i*, from linear elastic analysis
γ_{LL}	Partial safety factor for live load

Unless otherwise specified, all dimensions are in millimeters (*mm)*, loads in Newton (*N*), stresses in Mega-Pascal (*MPa*).

SECTION 2

ADDITIONAL SPECIFICATIONS FOR MASONRY BUILDINGS

5 GENERAL SPECIFICATIONS

5.1 The design and construction of masonry buildings shall be governed by provisions of IS 1905, except as modified by the provisions of this standard for those elements participating in lateral force resistance.

5.2 The Working Stress Method at serviceability loads as prescribed by IS 1905 shall be adopted for design of the four masonry building systems described below.

5.2.1 *Masonry Building with Prescriptive Bands* (*MWB*) and *Masonry Building with Prescriptive Bands and Vertical Reinforcements* (*MWBR*)

This category comprises of simple masonry buildings conforming to geometrical requirements as per IS 1905 not structurally designed, but provided with horizontal seismic bands and minimum prescriptive horizontal and vertical reinforcement. These buildings must in addition conform to the guidelines of size and position of openings in the masonry walls as specified in **8.2**.

5.2.2 *Confined Masonry Building* (*CMB*)

This category comprises of simple masonry buildings with load-bearing walls having reinforced concrete horizontal (tie-beams) and vertical (tie-columns) confining members built on all four edges of the masonry wall panel, conforming to the geometrical and design requirements specified in **9**.

5.2.3 *Reinforced Masonry Building* (*RMB*)

This category comprises of masonry buildings with load-bearing masonry walls provided with vertical and horizontal reinforcement as per structural design requirements specified in **10**.

5.2 Masonry bearing walls unless designed as reinforced masonry shall not be built of height greater than 15 m subject to a maximum of four storeys when measured from the base to the roof slab or ridge level. Such masonry bearing walls shall be reinforced in accordance with design requirements specified in **10**.

5.3 The minimum compressive strength of clay masonry units used for buildings up to 2-storey shall be 3.5 MPa, whereas 7.0 MPa for buildings more than 2-storey. A minimum compressive strength of 7.0 MPa shall be guaranteed for concrete blocks, and for masonry units considered for reinforced masonry construction. Steel reinforcement of grades Fe 415, Fe 500 and Fe 550 shall be used for construction of reinforced concrete tie-columns and tie-beams, but of grade Fe 415 or less for horizontal or vertical reinforcement in masonry walls. A minimum strength of concrete is 20 MPa as per IS 456.

6 MATERIALS

The materials used as part of lateral force resisting systems in masonry buildings shall conform to the provisions given hereunder.

6.1 Masonry Units

6.1.1 *General*

Masonry units shall meet the requirements of relevant standards. Masonry units that have been previously used shall not be re-used in brickwork or block-work construction unless they have been thoroughly cleaned and conform to the standard for similar new masonry units. Further, masonry units shall be strong enough so that they do not undergo local brittle failure.

The shape and dimension of masonry units, construction practices, including methods of positioning of reinforcement, placing, and compacting of grout, as well as design and detailing should be such as to promote homogeneity of structural members, development of the bond of the grout to both reinforcement and masonry units and avoidance of corrosion of reinforcement.

6.1.2 *Types of Units*

Masonry units complying with the following standards are acceptable for masonry construction:

a) Burnt clay building bricks : IS 1077, IS 2180 or IS 2222
b) Concrete blocks (solid and hollow) : IS 2185 (Part 1)
c) Burnt clay hollow bricks : IS 3952
d) Other solid bricks : IS 16720

6.1.3 *Compressive Strength*

The minimum compressive strength of masonry units used for the said four masonry building systems shall be:

a) Clay bricks:
  i) MWB and MWBR: 3.5 MPa for buildings up to 2 storeys, and
  7.0 MPa for buildings taller than 2 storeys.
  ii) CMB: 3.5 MPa for buildings up to 2 storeys, and
  7.0 MPa for buildings taller than 2 storeys.
  iii) RMB: 7.0 MPa for all buildings, and
b) Concrete blocks 7.0 MPa for the above four masonry building systems.

In the above, the compressive strength shall be determined based on the net area.

6.2 Mortar

Recommended mortar mixes given in Table 1 and complying with IS 2250 and IS 1905 are acceptable for masonry construction. The following grades of mortars shall be permissible for use in the said four earthquake resistant masonry buildings:

a) MWBs and MWBR : Types M3, M2, M1 and H2
b) CMBs : Types M1, M2, H1 and H2
c) RMBs : Types H2

In relation to Table 1, the following shall be ensured:

i) Sand for making mortar should be well graded. In case sand is not well graded, its proportion shall be reduced to achieve the minimum specified strength.
ii) For mixes given in 2(a) and 2(b) of Table 1, use of lime is not essential from consideration of strength as it does not result in increase in strength, but its use is recommended to improve workability.
iii) For mixes given in 3(a), 4(a) and 5(a) of Table 1, either lime C or B to the extent of 1/4 part of cement (by volume) or some plasticizer should be added to improve workability.
iv) For mixes given in 4(b) and 5(b) of Table 1, first lime and sand should be ground in mortar mill and then cement added to coarse material.
v) The lime mortar mixes B and C denote eminently semi-hydraulic lime and fat lime, respectively, as specified in IS 712.
vi) Mortar mix 2(c) for Grade H2 is without the use of lime.

Table 1 Recommended Mortar Mixes
(*Clause* 6.2)

S.No. as given in Table 1 of IS 1905	Grade of Mortar	Mix Proportions (by Volume)			Minimum Compressive Strength (MPa) at 28 Days
		Cement	Lime	Sand	
(1)	(2)	(3)	(4)	(5)	(6)
2(c)	H2	1	0	3	10.0
2(a)		1	¼ C or B	4	7.5
2(b)		1	½ C or B	4½	6.0
3(a)	M1	1		5	5.0
3(b)		1	1 C or B	6	3.0
4(a)	M2	1		6	3.0
4(b)		1	2 B	9	2.0
5(a)	M3	1		7	1.5
5(b)		1	3 B	12	1.5

6.3 Masonry

6.3.1 *Specified Compressive Strength*

The specified compressive strength f_m shall be determined by:

a) Testing prism specimens as per Annex B of IS 1905; or
b) Using the expression $f_m = 0.433 f_b^{0.64} f_{m0}^{0.36}$, where f_b is the compressive strength of clay brick (MPa) and f_{m0} is the compressive strength of mortar; or
c) Multiplying permissible compressive stress (as per **5.4.1** of IS 1905), for the combination of masonry unit strength and mortar type, by a factor of 4.0 (as per B-2.1 of IS 1905).

6.3.2 *Elastic Modulus*

Elastic modulus E_m of masonry shall be determined by:

a) Testing Masonry Prisms as per **6.3.1(1)**, subjected to compression, as the chord modulus of elasticity taken between 0.05 and 0.33 of the maximum compressive strength of each prism (Fig 1); or
b) Using the following empirical expressions:
   i) Clay brick masonry : $E_m = 550 f_m$, and
   ii) Concrete block masonry : $E_m = 850 f_m$.

6.3.3 *Shear Modulus*

The shear modulus G_m shall be determined by using the empirical expression:

$$G_m = 0.2 E_m .$$

6.4 Concrete

6.4.1 A minimum grade of concrete shall be M20 as per IS 456.

6.4.2 The chosen concrete mix shall have medium workability of 75-100 mm slump as specified in IS 1199 (Part 2) for the typical small cross-sections of reinforced concrete bands, tie-columns, and tie-beams. Also, the size of the coarse aggregate (as per IS 383) used in construction should not be more than 12.5 mm.

6.5 Grout

The grout to be used in vertical voids within the wall for the vertical steel reinforcement shall be of strength not lower than the masonry units. Also, the size of the coarse aggregate (as per IS 383) used in construction should not be more than 10 mm.

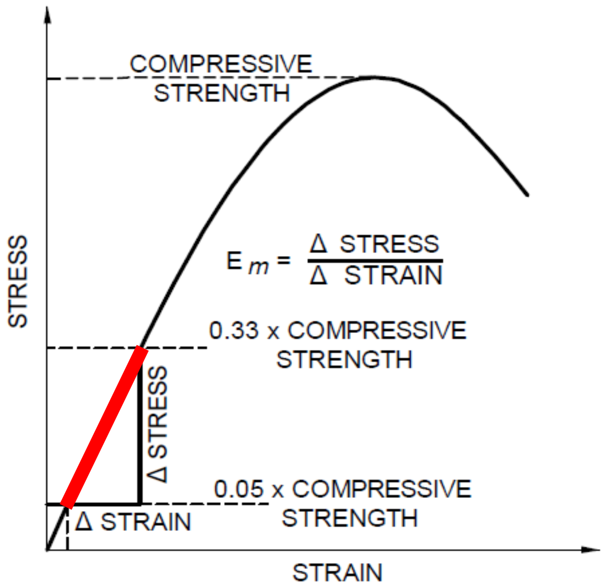


FIG. 1 CHORD MODULUS OF ELASTICITY OF MASONRY

6.5 Reinforcement

The grades of steel that can be used in reinforced concrete bands, tie-columns and tie-beams in masonry buildings shall be Fe415, Fe500 or Fe550 (IS 1786). Steel reinforcement of grade Fe 415 or less only shall be used as horizontal or vertical reinforcement in masonry walls. Only deformed bar shall be used.

7 DESIGN CONSIDERATIONS

Individual masonry members of the said four masonry building systems shall comply with provisions of **6.1** of IS 1893 (Part 2), and the additional provisions given hereunder.

7.1 Design Philosophy

The design approach adopted for masonry buildings is the Working Stress Method at serviceability loads, with structural materials assumed to behave in linear elastic manner. Safety is ensured by restricting stresses in material induced by expected “working loads” or “service loads” on structure. But the loads are not differentiated.

7.1.1 *Working Stress Method*

In the *Working Stress Method* of design of members, permissible stresses given in Table 2 shall govern the earthquake resistant design of members of said 3 masonry building systems.

7.2 Bands and Vertical Elements

Masonry buildings, other than Reinforced Masonry and Confined Masonry Buildings, shall be provided with horizontal bands and vertical reinforcing elements:

a) The horizontal bands shall run along the full perimeter of the external and internal walls, within the full thickness of the walls.
b) The vertical reinforcing elements shall pass through vertically at all corners and adjoining all openings (door, window, and ventilator).

7.2.1 *Types*

Five types of bands are admissible, namely:

a) Gable Band (when masonry buildings have gable walls),
b) Roof Band,
c) Lintel Band,
d) Sill Band, and
e) Plinth Band.

These bands shall be provided in masonry building in the above order of priority.

Table 2 Permissible stresses in materials to be used design of members of masonry buildings
(*Clause* 7.1.1)

Sl No.	Compression		Tension	Shear
	Axial	Axial and Flexural	Axial and Flexural	
(1)	(2)	(3)	(4)	(5)
1. Masonry Building with Prescriptive Bands (MWB) *2. Masonry Building with Prescriptive Bands and Vertical Reinforcements (MWBR)*				
i)	5.4.1 and Table 8 of IS 1905 apply	Increase in permissible compressive stress permitted as per 5.4.1.4 of IS 1905 applies	5.4.2 of IS 1905 applies. Masonry walls subjected to in-plane bending and vertical loads shall be designed based on no-tension, permissible shear, and compressive stresses	5.4.3 of IS 1905 applies
3. Reinforced Masonry Buildings (RMB)				
ii)	Compressive force due to axial load shall not exceed: P_o = $(0.25f_mA_m + 0.65A_tF_s)k_s$	Compressive stress in masonry due to combined action of axial load and bending shall not exceed 1.25 F_a.	5.4.2 of IS 1905 applies.	For Flexural members (i) without web reinforcement: $F_v = 0.083\sqrt{f_m}$ $\leq 0.75 MPa$ (ii) with web reinforcement: $F_v = 0.25\sqrt{f_m}$ $\leq 0.75 MPa$ For walls, shall be as per Table 1
4. Confined Masonry Buildings (CMB)				
iii)	Compressive force in confined masonry due to axial load shall not exceed: P_o = $(0.25f_mA_m + f_{cc}\ A_c + 0.65A_sf_s)\ k_s$	Compressive stress in masonry due to combined action of axial load and bending shall not exceed 1.25fa	5.4.2 of IS 1905 applies.	Allowable shear stress shall be according to Table 1

7.2.2 *Geometry*

The width of RC band shall be same as the thickness of the wall. Wall thickness shall be at least 200 mm. A clear cover of 20 mm from face of wall shall be maintained.

The vertical thickness of RC Band be kept 75 mm minimum, where:

a) Two longitudinal bars are specified, one on each face; and
b) 150 mm, where four bars are specified.

7.2.3 *Ductile Detailing*

The bands shall be made of reinforced concrete of grade not leaner than M20 or reinforced brickwork in cement mortar not leaner than 1:3. In coastal areas, the concrete grade shall be M20 concrete and the filling mortar of 1:3 (cement-sand with water proofing admixture).

7.3 Floor Slabs

The roof and floor slabs in MWBs, MWBR, CMBs and RMBs shall be designed as beam supported slabs as per the provisions of IS 456.

7.4 Footings

Strip footings under load bearing walls shall be designed as per the provisions of IS 1904 in conjunction with IS 1893 (Part 1).

8 MASONRY BUILDINGS WITH PRESCRIPTIVE BANDS (MWB), AND MASONRY BUILDINGS WITH PRESCRIPTIVE BANDS AND VERTICAL REINFORCEMENT (MWBR)

Masonry buildings shall be strengthened by the methods specified hereunder. For specifying the earthquake resisting features in MWBs and MWBRs, the buildings shall be categorized into four types (Table 3), namely Categories B, C, D and E, based on the earthquake zone and the category of the building [mentioned in Table 5 of CED 39(22345)]. Type E buildings shall not be more than 3 storeys tall.

MWB and MWBR are not admissible structural systems for *Critical and Lifeline Buildings* and *Special Buildings* as per CED 39(22345) in all zones, and for Important Building Category in Earthquake Zones IV, V and VI.

Table 3 Building Types depending on Category of Building and Earthquake Zone
(*Clause* 8)

Sl No.	Category of Building	Earthquake Zone				
		II	III	IVs	V	VI
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	Normal Building	B	C	D	E	E
ii)	Important Building	C	D	Not Admissible as per CED 39(22345)		
iii)	Critical and Lifeline Building	MWB and MWBR are not admissible as per CED 39(22345)				
iv)	Special Building					

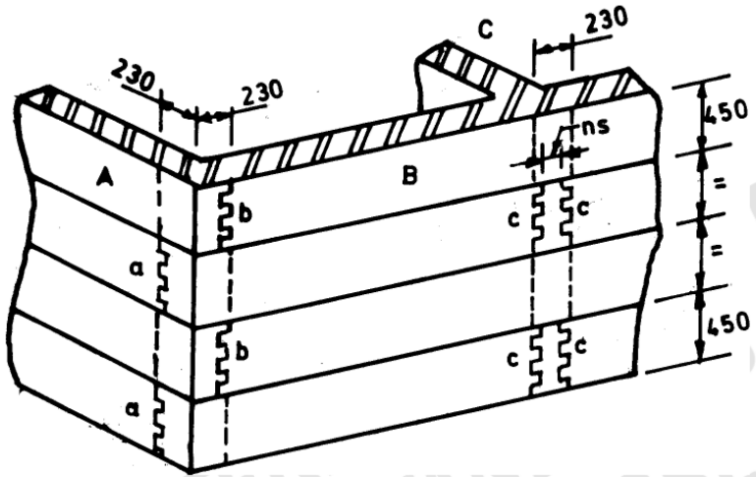
8.1 Masonry Walls

8.1.1 The bearing walls in both directions shall be straight and symmetrical in plan as far as possible.

8.1.2 The wall panels formed between cross walls and floors, or roof shall be checked for their strength in bending as a plate or as a vertical strip subjected to the earthquake force acting on its own mass. This check need not be complied with in walls of 200 m or larger thickness and of storey height not more than 3.5 m.

8.1.3 *Masonry Bond*

For achieving full strength of masonry, the usual bonds specified for masonry should be followed so that the vertical joints are broken properly from course to course. To obtain full bond between perpendicular walls, it is necessary to make a slopping (stepped) joint by making the corners first to a height of 600 mm and then building the wall in between them. Otherwise, the toothed joint should be made in both the walls alternatively in lifts of about 450 mm (see Fig. 2).



a, b, c — toothed joints wall and A, B, C

FIG. 2 ALTERNATING TOOTHED JOINTS IN WALLS AT CORNER AND T-JUNCTION
(ALL DIMENSIONS ARE IN MM)

8.2 Openings in Bearing Walls

8.2.1 Door and window openings reduce the lateral load resistance of walls, and hence, should preferably be small and more centrally located. The guidelines on the size and position of opening are given in Table 4 and Fig. 3.

Table 4 Size and Position of Openings in Bearing Walls
(*Clause* 8.2.1)

Sl No.	Position of Opening	Details of Opening		
		B	C	D and E
(1)	(2)	(3)	(4)	(5)
i)	Minimum Distance b_5 (mm) from inside corner of	0	230	450

	outside wall			
ii)	Total length of openings shall be such that the ratio $(b_1 + b_2 + b_3)/l_1$ or $(b_6 + b_7)/l_2$ does not exceed:			
	(a) 1-storeyed building	0.60	0.55	0.50
	(b) 2-storeyed building	0.50	0.46	0.42
	(c) 3 or 4 storeyed building	0.42	0.37	0.33
iii)	Minimum width (mm) of pier between consecutive openings b_4	340	450	560
iv)	Minimum vertical distance (mm) between two openings (one above the other) h_3	600	600	600
v)	Maximum Width (mm) of opening of ventilator b_8	900	900	900

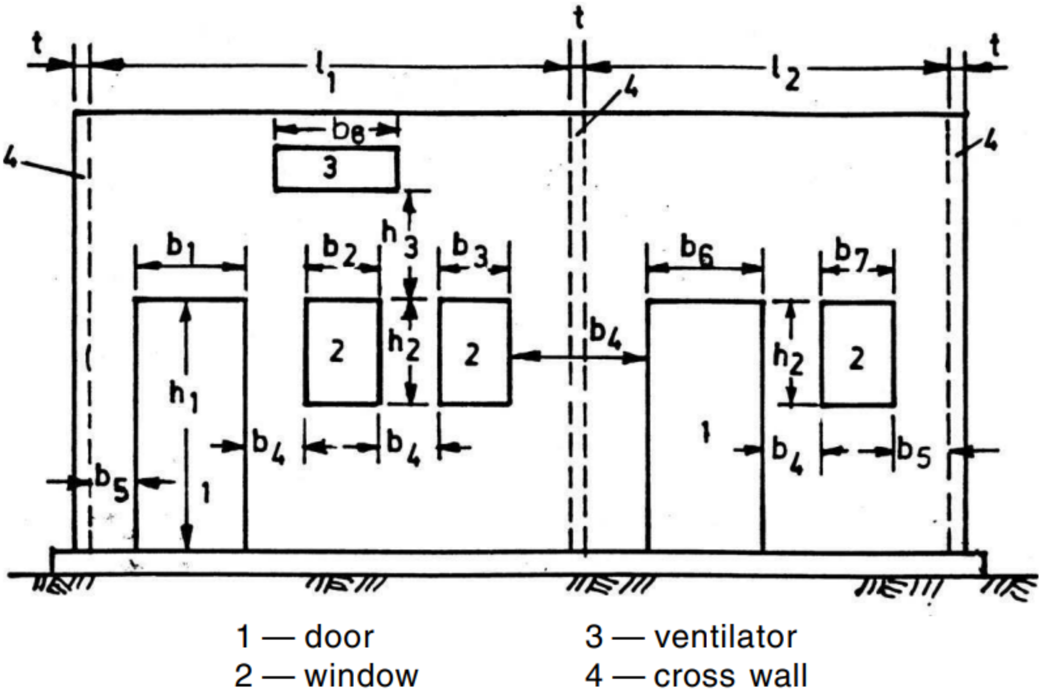


FIG. 3 DIMENSIONS OF OPENINGS AND PIERS FOR RECOMMENDATIONS IN TABLE 4

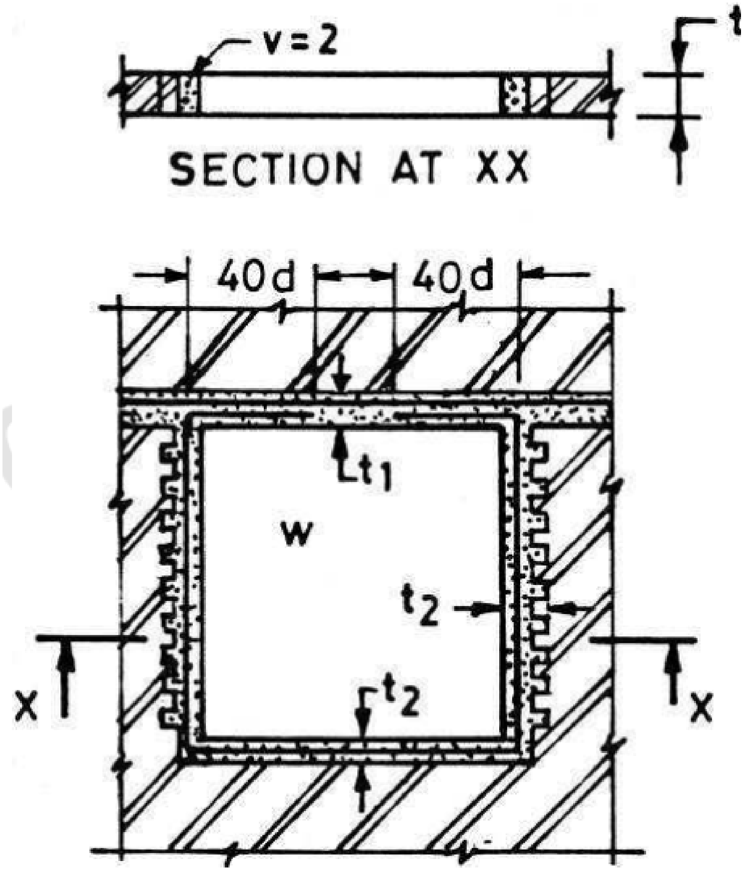
8.2.2 Openings in any storey shall preferably have their top at the same level so that a continuous band could be provided over them, including the lintels throughout the building. In addition, openings in different storeys in a masonry wall must be vertically aligned, and not staggered.

8.2.3 Where openings do not comply with the guidelines of Table 4, they should be strengthened by providing reinforced concrete or reinforcing the brickwork, as shown in Fig. 8 with high strength deformed (H.S.D.) bars of 8 mm diameter but the quantity of steel shall be increased at the jambs to comply with **8.3.9,** if so required.

8.2.4 If a window or ventilator is to be projected out, the projection shall be in reinforced masonry or concrete and well anchored.

8.2.5 If an opening is tall (from bottom to almost top of a storey, thus dividing the wall into two portions), the adjoining portions shall be reinforced with horizontal reinforcement of 6 mm diameter bars at not more than 450 mm intervals, one on inner and one on outer face, properly tied to vertical steel at jambs, corners or junction of walls, where used.

8.2.6 The use of arches to span over the openings is a source of weakness and shall be avoided. When compelled to use, steel ties shall be provided.



w — window *t* — wall thickness
t_1 — lintel thickness t_2 — thickness of concrete in jamb
v — vertical bar *d* — diameter of reinforcing bar

FIG. 4 STRENGTHENING MASONRY AROUND OPENING

8.3 Seismic Strengthening Arrangements

8.3.1 All masonry buildings shall be strengthened by the methods specified for different types of buildings, as listed in Table 5, and detailed in subsequent clauses. Fig. 5 and Fig. 6 show, schematically, the overall strengthening arrangements to be adopted for Types D and E buildings, which consist of horizontal bands of reinforcement at critical levels, vertical reinforcing bars at corners, junctions of walls and jambs of openings.

In four-storey buildings of Type B, the requirements of vertical steel may be checked through an earthquake analysis using a design horizontal acceleration coefficient equal to 4 times that given in CED 39 (22343). If this analysis shows that vertical steel is not required, the designer may take the decision accordingly.

Table 5 Strengthening Arrangements Recommended for Masonry Buildings made with Rectangular Masonry Units
(*Clause* 8.3.1)

Sl No.	Type of Building	Number of Storeys	Strengthening to be provided in All Storeys
(1)	(2)	(3)	(4)
i)	B	1 to 3	a, b, c, f, g
		4	a, b, c, d, f, g
ii)	C	1 and 2	a, b, c, f, g
		3 and 4	a to g
iii)	D	1 and 2	a to g
		3 and 4	a to h
iv)	E	1 to 3 (4 storeys are not permitted)	a to h

where
a : Masonry mortar (*as per* **6.2**);
b : Lintel band (*as per* **8.3.2**);
c : Roof band and gable band where necessary (*as per* **8.3.3** and **8.3.4**);
d : Vertical steel at corners and junctions of walls (*as per* **8.3.8**);
e : Vertical steel at jambs of openings (*as per* **8.3.9**);
f : Bracing in plan at tie level of roofs;
g : Plinth band where necessary (*as per* **8.3.6**); and
h : Dowel bars (*as per* **8.3.7**).

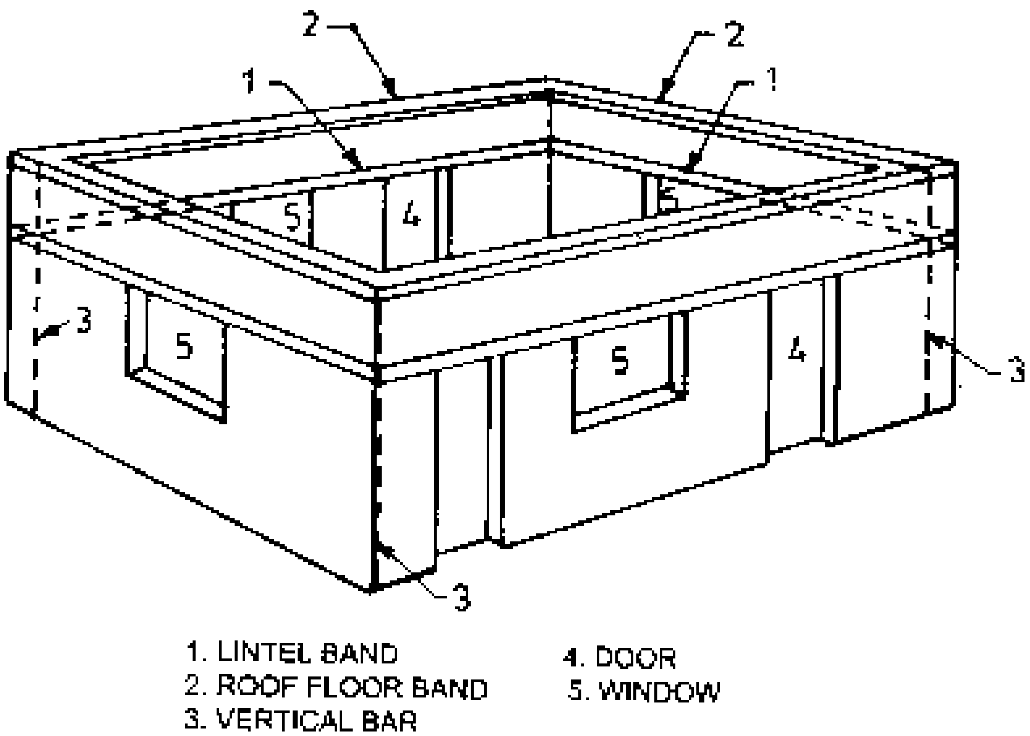
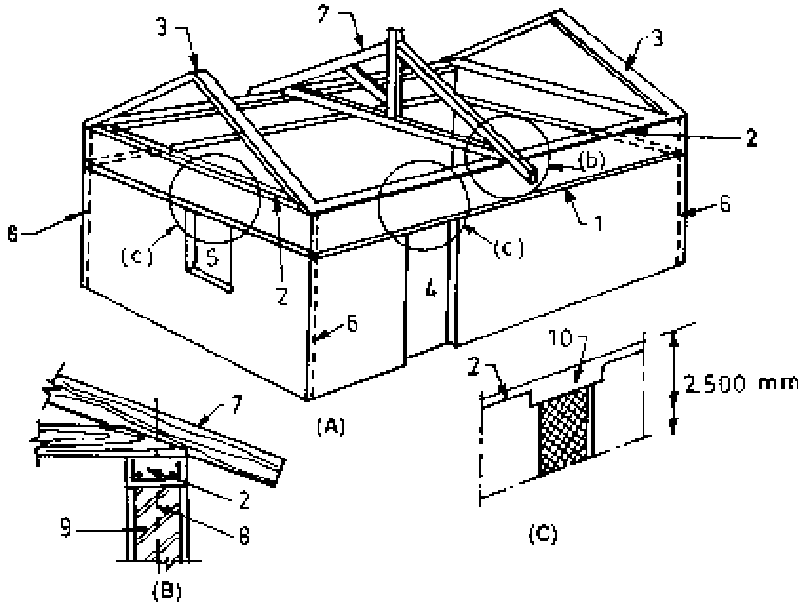


FIG. 5 OVERALL ARRANGEMENT OF REINFORCING MASONRY BUILDINGS



1. LINTEL BAND
2. EAVE LEVEL (ROOF) BAND
3. GABLE BAND
4. DOOR
5. WINDOW
6. VERTICAL STEEL BAR
7. RAFTER
8. HOLDING DOWN BOLT
9. BRICK/STONE WALL
10. DOOR LINTEL INTEGRATE WITH ROOF BAND

18 A PERSPECTIVE VIEW
18 B DETAILS OF TRUSS CONNECTION WITH WALL
18 C DETAILS OF INTEGRATING DOOR LINTEL WITH ROOF BAND

FIG. 6 OVERALL ARRANGEMENT OF REINFORCING MASONRY BUILDING HAVING PITCHED ROOF

8.3.2 Lintel band shall be provided at the lintel level on all load bearing internal, external longitudinal and cross walls, as per **8.3.5**. This band provided in panel or partition walls improves their stability during strong earthquake shaking.

8.3.3 Roof band shall be provided immediately below the roof or floors. The specifications of the band are given in **8.3.5**. Such a band need not be provided underneath reinforced concrete or brick-work slabs resting on bearing walls, provided that the slabs are continuous over the intermediate wall up to the crumple sections, if any, and cover the width of end walls, fully or at least three-fourths of the wall thickness.

8.3.4 Gable band is provided at the top of gable masonry below the purlins. The specifications of the band are given in **8.3.5**. This band shall be made continuous with the roof band at the eaves level.

8.3.5 *Section and Reinforcement of Band*

8.3.5.1 The band shall be made of reinforced concrete of grade not leaner than M15 or reinforced brickwork in cement mortar not leaner than 1:3. In coastal areas, the concrete grade shall be M20 concrete and the filling mortar of 1:3 (cement-sand with water proofing admixture).

8.3.5.2 The bands shall be of the full width of the wall not less than 75 mm in depth and reinforced with longitudinal steel (Table 6). The longitudinal steel bars shall be held in position by steel links or stirrups 6 mm diameter spaced 150 mm apart. With respect to Table 6,

a) Span of wall shall be the distance between centerlines of its cross walls or buttresses. For spans greater than 8 m it shall be desirable to insert pilasters or buttresses to reduce the span or special calculation shall be made to determine the strength of wall and section of band.
b) The numbers and diameter of bars given above pertain to high strength deformed bars.
c) Width of R.C. band is assumed same as the thickness of the wall. Wall thickness shall be 200 mm minimum. A clear cover of 20 mm from the face of wall shall be maintained.
d) The vertical thickness of R.C. Band be kept 75 mm minimum, where two longitudinal bars are specified, one on each face; and 150 mm, where four bars are specified.
e) Concrete mix shall be of grade M 20 of IS 456 of 1:1½ :3 by volume.
f) The longitudinal steel bars shall be held in position by steel links or stirrups 6 mm diameter spaced at 150 mm apart.

Table 6 Recommended Longitudinal Steel in Reinforced Concrete Bands
(*Clauses* 8.3.5.2)

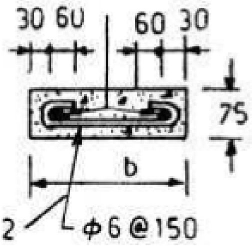
Sl No.	**Span** (m)	**Type of Building**							
		B		**C**		**D**		**E**	
		Number of Bars	*Diameter* (mm)	*Number of Bars*	*Diameter* (mm)	*Number of Bars*	*Diameter* (mm)	*Number of Bars*	*Diameter* (mm)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
i)	5 or less	2	8	2	8	2	8	2	10
ii)	6	2	8	2	8	2	10	2	12
iii)	7	2	8	2	10	2	12	4	10
iv)	8	2	10	2	12	4	10	4	12

8.3.5.3 In reinforced brickwork, the thickness of joints containing steel bars shall be increased to have at least a minimum mortar cover of 10 mm around the bar. In bands of reinforced brickwork, the area of steel provided should be equal to that specified above for reinforced concrete bands.

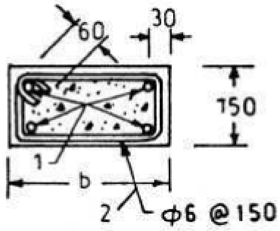
8.3.5.4 For full integrity of walls at corners and junctions of walls and effective horizontal bending resistance of bands continuity of reinforcement is essential. The details as shown in Fig. 7 are recommended.

8.3.6 Plinths band is provided at plinth level of walls on top of the foundation wall. This is to be provided where strip footings of masonry (other than reinforced concrete or reinforced masonry) are used and the soil is either soft or uneven in its properties, as frequently happens in hill tracts. Where used, its section may be kept the same as in **8.3.5.** This band shall serve as damp proof course as well.

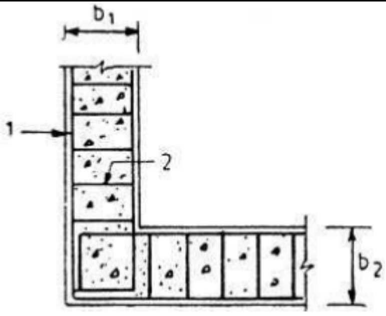
8.3.7 In Type D and E buildings, to further enhance the box action of walls, steel dowel bars may be used at corners and T-junctions of walls at the sill level of windows to length of 900 mm from the inside corner in each wall. Such dowel may be in the form of U stirrups 8 mm diameter. Where used, such bars must be laid in 1:3 cement-sand-mortar with a minimum clear cover of 10 mm on all sides to minimize corrosion.



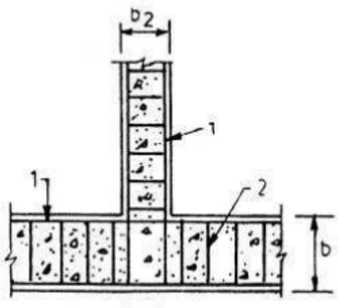
7A SECTION OF BAND WITH TWO BARS



7B SECTION OF BAND WITH FOUR BARS



7C STRUCTURAL PLAN AT CORNER JUNCTION



7D SECTION PLAN AT T-JUNCTION OF WALLS

1 — longitudinal bars
2 — lateral ties
b1, b2 — wall thickness

FIG. 7 REINFORCEMENT AND BENDING DETAIL IN RC BAND (ALL DIMENSIONS ARE IN MM)

8.3.8 *Vertical Reinforcement*

Vertical steel at corners and junctions of walls, which are up to 340 mm (1½ brick) thick, shall be provided as specified in Table 7. For walls thicker than 340 mm, the area of the bars shall be proportionately increased.

With respect to Table 7,

a) The diameters given above are for reinforcing steels of grade Fe415 or higher.
b) Typical details of providing vertical steel in brickwork masonry with rectangular solid units at corners and T-junctions are shown in Fig. 8. The vertical bars shall be covered with concrete M 20 or mortar 1:3 grade in suitably created pockets around the bars (Fig. 8). This shall ensure their safety from corrosion and good bond with masonry.
c) In floors or roofs with small precast components, see **9.2.3** for floor or roof band details.

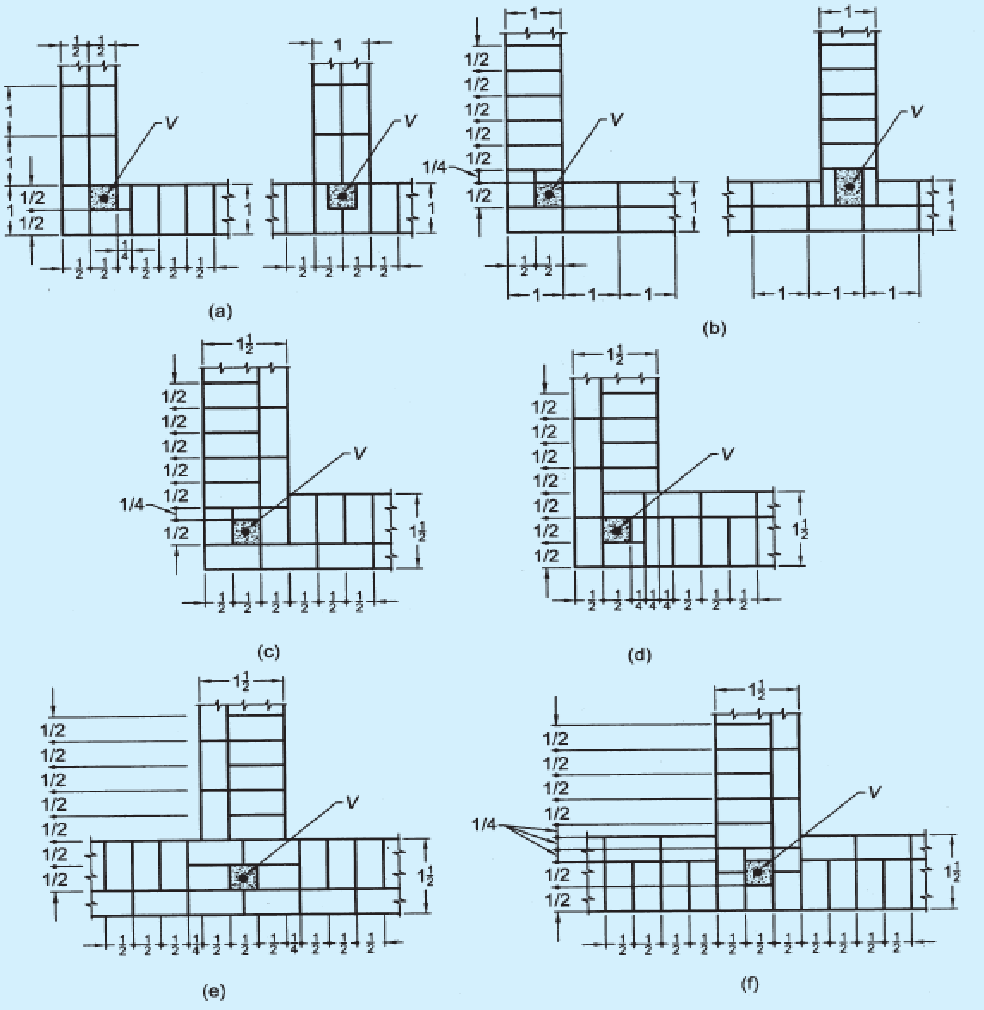
Table 7 Vertical Steel Reinforcement in Masonry Walls with Rectangular Masonry Units
(*Clause* 8.3.8)

Sl No.	Number of Storeys	Storey	Diameter (in mm) of HSD Single Bar at each critical section			
			Type B	Type C	Type D	Type E
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	1	-	Nil	Nil	10	12
ii)	2	Top	Nil	Nil	10	12
		Bottom	Nil	Nil	12	16
iii)	3	Top	Nil	10	10	12
		Middle	Nil	10	12	16
		Bottom	Nil	12	12	16
iv)	4	Top	10	10	10	Buildings with 4 storeys not permitted.
		Third	10	10	12	
		Second	10	12	16	
		Bottom	12	12	20	

8.3.8.1 The vertical reinforcement shall be properly embedded in the plinth masonry of foundations and roof slab or roof band so as to develop its tensile strength in bond. It shall pass through the lintel bands and floor slabs or floor level bands in all storeys.

8.3.8.2 Bars in different storeys may be suitably lapped.

8.3.9 Vertical reinforcement of jambs of window and door openings shall be provided as per Table 7. It may start from the foundation of floor and terminate in lintel band (Fig. 8).



1 — One-brick length, ½ — Half brick length, *V* — Vertical steel bar with mortar/concrete filling in pocket
(a) and (b) — Alternate courses in one brick
(c) and (d) — Alternate courses at corner junction of 1½-brick wall
(e) and (f) — Alternate courses at T-junction of 1½-brick wall

FIG. 8 TYPICAL DETAILS OF PROVIDING VERTICAL STEEL BARS IN BRICK MASONRY

9 CONFINED MASONRY BUILDINGS

9.1 Design Considerations

9.1.1 *General*

The provisions given in **9.1.2** to **9.1.6** apply in addition to those specified in **4** of IS 1905.

9.1.2 *Structural Integrity*

Intersecting walls shall be joined together to resist the effect of gravity and lateral loads. The walls shall be adequately bonded to elements which provide lateral support, such as floors and roofs.

9.1.3 *Building Configuration*

A regular building configuration is one of the key requirements for satisfactory earthquake performance. The following recommendations related to building plan shape shall be followed:

a) The building plan should be of a regular shape.
b) The building's length-to-width ratio in plan shall not exceed 4 and storey height shall be less than 4 m.
c) The walls should be built in a symmetrical manner with regard to the horizontal axes through the centre of the building plan. The walls should be placed as far apart as possible, preferably at the façade, to avoid twisting (torsion) of the building in an earthquake.
d) There are at least two lines of walls in each orthogonal direction of the building plan, and the walls along each line extend over at least 50 percent of the building dimension in the direction of analysis at each storey level (Fig. 9).
e) The walls should always be continuous up to the building height – vertical offsets are not permitted.
f) Openings (doors and windows) should be placed in the same position on each floor.
g) The total cross-sectional area of all walls at two adjacent floors should not be different by more than 30 percent.
h) In a building, it is not permitted to have a moment resisting frame system in the ground storey and confined masonry system in the upper storeys.

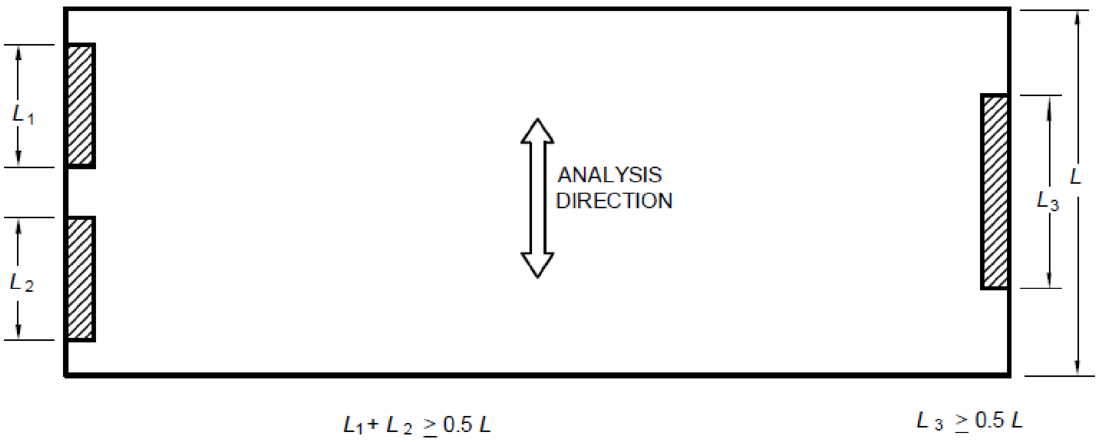


FIG. 9 AT LEAST TWO PARALLEL LINES OF WALLS ARE REQUIRED IN EACH PLAN DIRECTION

9.1.4 *Minimum Design Dimensions and Placements of Confining Elements*

Requirements regarding spacing of tie-columns and tie-beams are presented in Fig. 10. Minimum tie-column and tie-beam dimensions shall be 150 mm along the wall direction and as much as the thickness of the wall in the direction perpendicular to it.

9.1.4.1 *Placement of reinforced concrete tie-columns and tie-beams*

Tie-columns should be provided at:

a) Intersections of walls,
b) Intermediate locations in longer walls, where spacing should not exceed lesser than 1.5*h* (where *h* is the clear wall height between floors) or 4 m; and
c) Free ends of wall panels that provide lateral load resistance to the building (Fig. 11).

Also, reinforced concrete tie-beams must be provided at the top of each wall and spacing between tie-beams should be preferably less than 4 m.

9.1.4.2 *Horizontal Lintel Bands*

When the unsupported height of the wall *h* is greater than 2.5 m, a continuous horizontal lintel band shall be provided over the openings. However, continuous horizontal band is not essential for $h < 2.5$ m, but a lintel beam shall be provided above the openings.

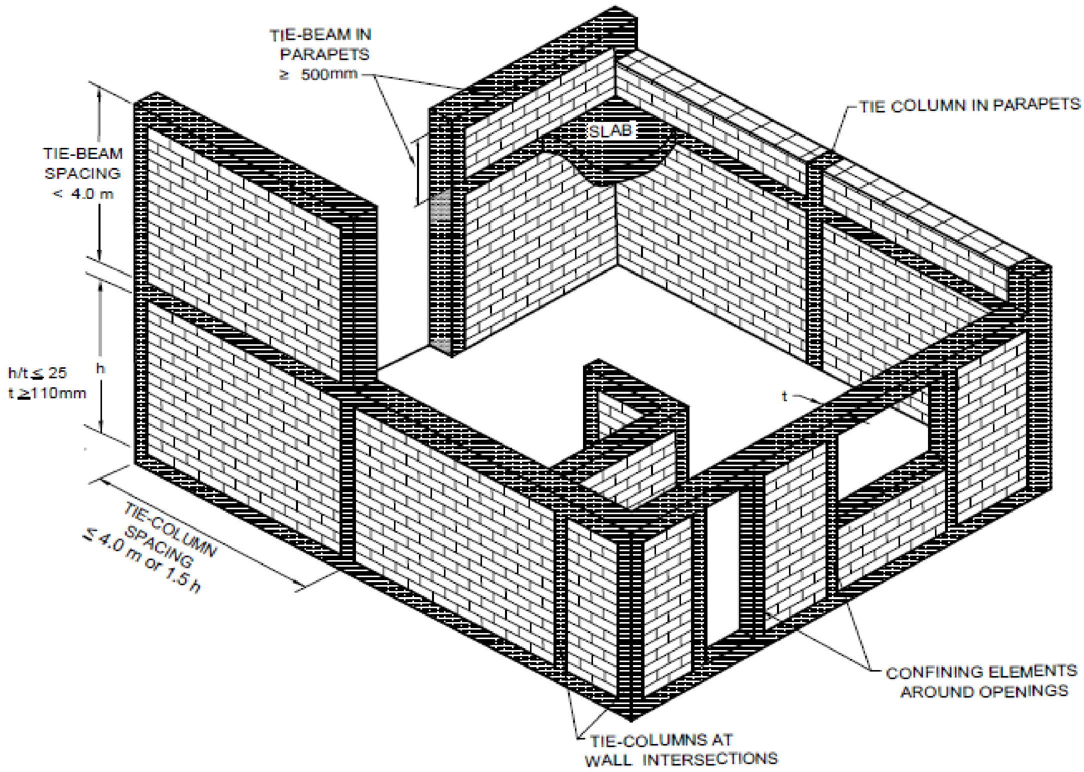


FIG. 10 REQUIREMENTS REGARDING CONFINING ELEMENTS

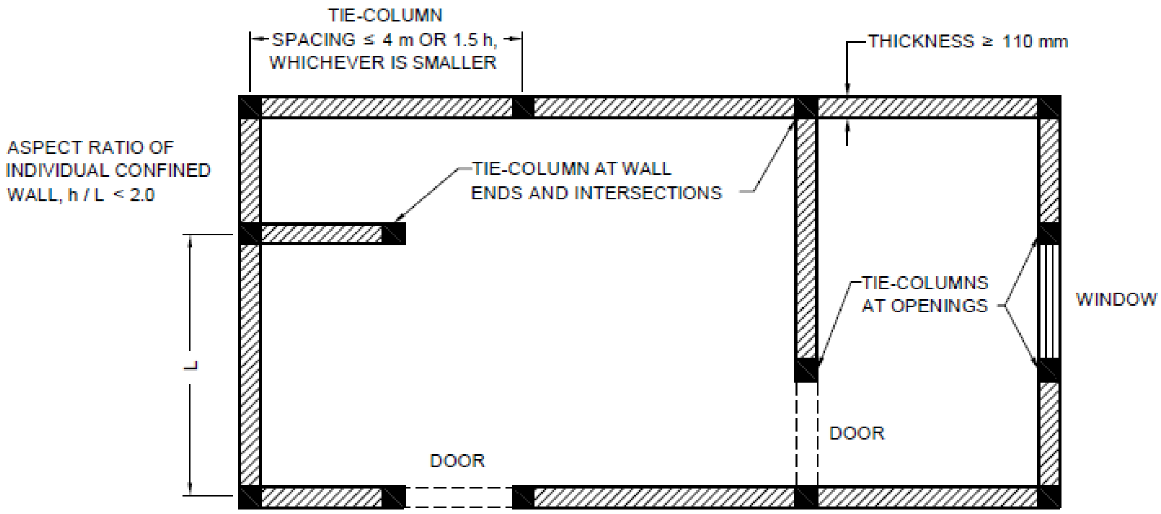


FIG. 11 TYPICAL PLAN ILLUSTRATING THE PLACEMENT OF RC TIE COLUMNS

9.1.5 *Minimum Dimensions of Masonry Walls*

The following dimensions shall be applicable:

a) Wall thickness (t) should not be less than 110 mm; and
b) The maximum wall height/thickness (h/t) ratio shall not exceed 25, where h is the wall height between floors.

9.1.5.1 *Aspect Ratio of Confined Masonry Walls*

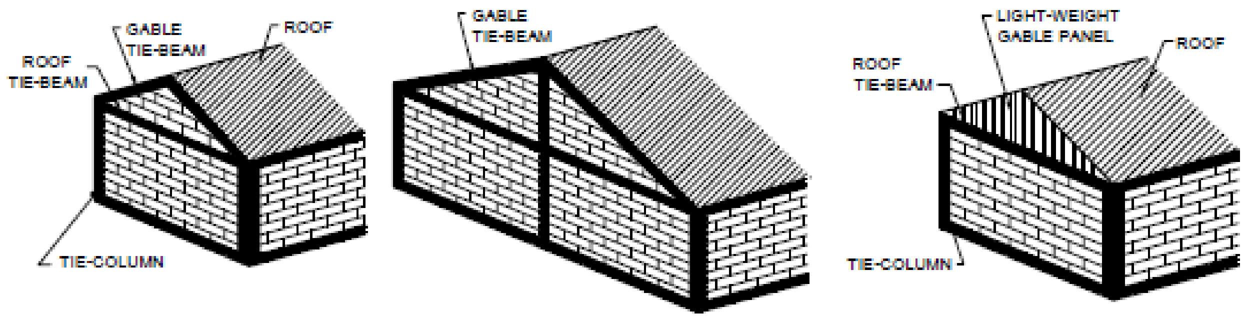
The height-to-width ratio of a wall should be kept less than 2 for better lateral load transfer, otherwise a wall should not be considered in seismic design.

9.1.5.2 *Parapets*

RC tie-columns and tie-beams should extend to the top of the parapet (Fig. 10). When a parapet is not confined by tie-beams, height should not exceed 3 times the thickness, otherwise the height limit shall be 1.2 m.

9.1.5.3 *Gable Walls*

The top of gable should be confined with reinforced concrete tie-beams, and tie-columns located at the middle of the gable wall should be extended from the lower floor to the top of gable wall (whenever applicable), as shown in Fig. 12A. Alternatively, a gable portion of the wall can be made of timber or other light-weight material (Fig. 12B).



5A RC Confining Elements 5B Light-Weight Gable Panels

FIG. 12 GABLE WALLS

9.1.6 *Walls with Openings*

9.1.6.1 *Size of opening*

The presence of large openings may have a negative effect on seismic performance of confined masonry buildings, especially if openings are not confined. A large opening has a total area greater than 10 percent of the wall panel area, while a small opening has a total area less than or equal to 10 percent of the wall panel area.

9.1.6.2 *Walls with large openings*

The following three approaches shall be followed in walls with large openings:

a) When reinforced concrete tie-columns are not provided at the ends of an opening, the panel is not considered as confined and its contribution to seismic resistance of the building should be disregarded but should be strengthened (Fig. 13A).
b) When reinforced concrete tie-columns are provided at the opening, the confined masonry panels are considered to contribute to seismic resistance of the building, but the aspect ratio *H*/*L* of these panels should be less than 2.0 (Fig. 13B). Better

performance can be achieved by providing both sill and lintel bands below and above the openings, respectively.

c) If total area of openings is greater than 25 percent, both openings and masonry piers must be confined with horizontal and vertical confining elements (Fig. 13C).

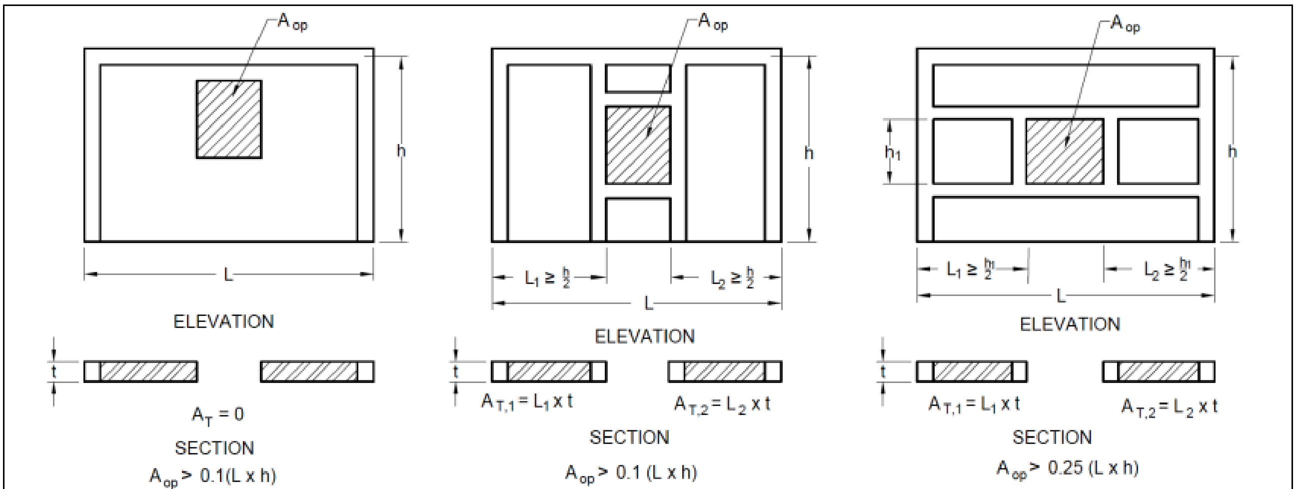
9.1.6.3 *Walls with small and single openings*

The following three approaches shall be followed in walls with small openings:

a) When the opening is located outside the diagonals (Fig. 14A), it can be ignored, and the entire wall cross-sectional area considered for earthquake resistance.

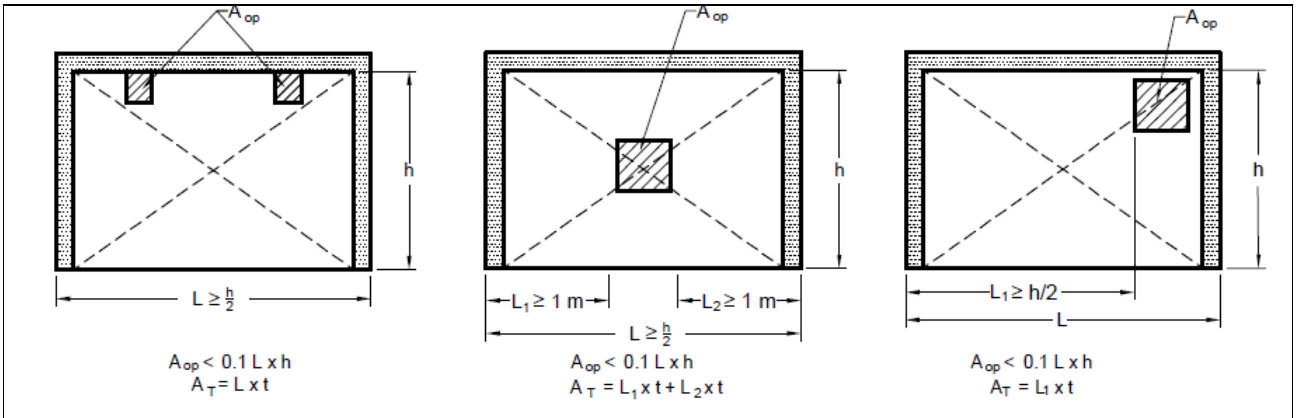
b) When an opening is located at the intersection of the panel diagonals (Fig. 14B), the panel cross-sectional area (A_T) considered for earthquake resistance should exclude the opening length.

c) When an opening is located close to one end of the panel, the panel cross-sectional area (A_T) considered in earthquake resistance should use a larger pier length (Fig. 14C).



13A Unconfined panel to be disregarded in the simplified method of calculations	13B RC tie-columns are provided at the opening and wall panels can be considered in the simplified method of calculations	13C Provision of continuous horizontal bands for confining both openings and masonry piers

FIG. 13 MASONRY WALLS WITH LARGE OPENINGS



14A Openings outside the diagonals can be neglected	14B Openings at the intersection of diagonals should considered	14C Openings along a diagonal should be considered

FIG. 14 CONFINED MASONRY WALL PANEL WITH A SMALL OPENING

9.2 Structural Design

9.2.1 *Design Criteria*

Design of structural elements should be performed according to either the working stress design method or the limit states design method.

9.2.2 *Structural Design as per Working Stress Method*

9.2.2.1 *Load combinations*

When the Working Stress Design method is followed for the structural design of confined masonry structures, adequacy of the structure and member shall be investigated for the following load combinations:

a) DL + IL
b) DL + IL + (WL or EL)
c) DL + WL
d) 0.9 DL + EL

Permissible stresses and loads for load cases (b), (c), and (d) may be increased by one-third when wind or earthquake loads are considered in load combinations.

9.2.2.2 *Transformed section properties*

Confined masonry wall panel is a composite structural element which consists of a masonry wall and reinforced concrete tie-columns, and it shall be designed as a transformed section. Modular ratio (*m*) represents the ratio of elastic modulus for concrete and masonry. The modulus of elasticity of concrete can taken as per **IS 456** and modulus of elasticity of masonry as per **6.3.2**. Transformed section properties of a confined masonry panel used in design are illustrated in Fig. 15.

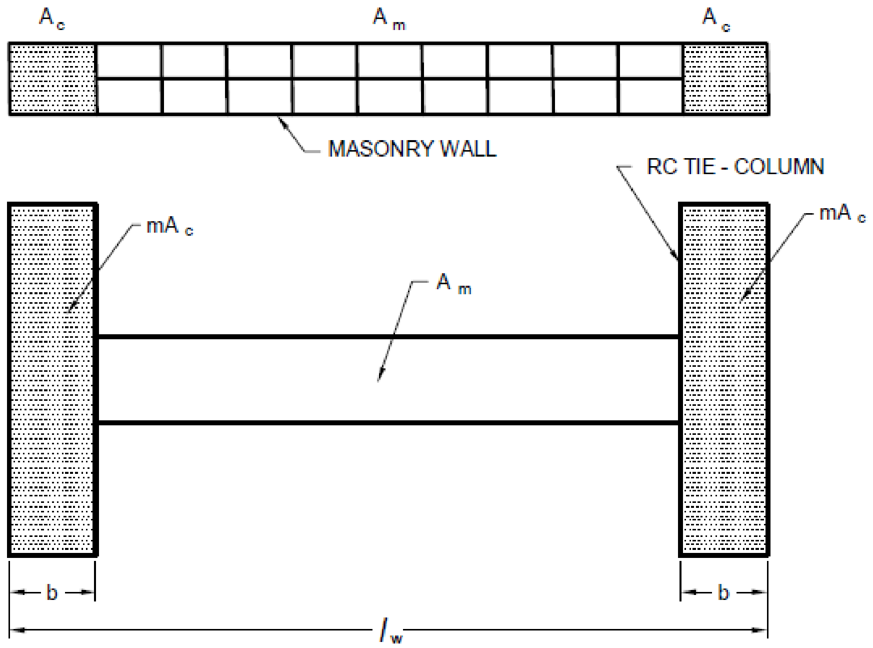


FIG. 15 TRANSFORMED SECTION PROPERTIES OF A CONFINED MASONRY PANEL

9.2.2.3 *Permissible compressive force*

Compressive force in confined masonry due to axial load shall not exceed that given by:

$$P_o = (0.25\, f_m\, A_m + f_{cc}\, A_c + 0.65\, A_s\, f_s)\, k_s$$

where

A_m = Net area of masonry;
A_c = Cross-sectional area of concrete excluding reinforcing steel;
A_s = Area of steel;
f_{cc} = Permissible stress in concrete in direct compression (Table 21 of IS 456);
f_s = Permissible steel tensile stress (Table 22 of IS 456); and
k_s = Stress reduction factor as in Table 9 of IS 1905.

9.2.2.4 *Combined Permissible Axial and Flexural Compressive Stress*

For walls subjected to combined axial load and flexure, the compressive stress in masonry due to combined action of axial load and bending shall not exceed 1.25 f_a and compressive stress in masonry due to axial load only shall not exceed f_a.

9.2.2.5 *Permissible Tensile Stress*

Provisions of **5.4.2** of IS 1905 shall apply.

9.2.2.6 *Permissible Shear Stress*

The allowable shear stress for confined masonry walls shall be as per Table 8. If there is tension in any part of a section of masonry, the area under tension shall be ignored while working out shear stress on the section.

When designing buildings as per Working Stress Method, the Load Combinations given in Table 9 shall be used. These combinations are provided considering short-term effects. When assessing the long-term effects due to creep, the Dead Load, and that part of Imposed Load likely to be permanent alone shall be used.

Table 8 Shear Stress τ_{cm} for Confined Masonry
(*Clause* 9.2.2.6)

Sl No.	M/V_d	f_t (MPa)	Maximum Allowable Shear Strength (MPa)
(1)	(2)	(3)	(4)
i)	< 1	$\frac{1}{36}\left(4-\frac{M}{Vl_w}\right)$	$\left(0.4-0.2\frac{M}{Vl_w}\right)$
ii)	> 1	$0.083\sqrt{f_m}$	0.4

Table 9 Load Combinations to be considered in Design of Buildings as per Working Stress Method
(*Clause* 9.2.2.6)

Combination Case	Serviceability Load Combination
(1)	(2)
1	DL + IL
2	DL + EL
3	DL + 0.8 IL + 0.8 EL

9.2.3.5 *Design assumptions*

The following assumptions shall be taken in design of confined masonry wall panels subjected to axial and/or flexural loads:

a) Masonry behaves like a homogeneous material.
b) The strain distribution along the wall length assumes that the wall section remains plane.
c) Tensile stresses are resisted only by reinforcing steel (masonry and concrete tensile resistance is ignored).
d) There is a perfect bond between steel reinforcement and adjacent concrete.
e) The maximum compressive strains in masonry (ε_m) and concrete (ε_c) shall be taken equal to 0.0040 and 0.0030, respectively.
f) Linear stress-strain relationship for masonry shall be considered for the design unless a more accurate relationship is determined from experimental testing of masonry prisms.

9.2.3.6 *Stress-strain curves for concrete, masonry and steel*

The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be rectangle, trapezoid, parabola, or any other shape which results in prediction of strength in substantial agreement with the results of test. An acceptable stress-strain curve is given in IS 456. But the ultimate strain in concrete should be taken as 0.0030 in place of 0.0035. For design purposes, the compressive strength of concrete in the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor γ_m of 1.5 shall be applied in addition to this.

The stresses in the reinforcement are derived from representative stress-strain curve for the type of steel used. Typical curves are given in IS 456 (Fig. 23 of IS 456). For design purposes, the partial safety factor γ_m of 1.15 shall be applied.

The relation between the compressive stress-strain in masonry may be assumed to be rectangle, trapezoid, parabola, or any other shape which results in prediction of strength in substantial agreement with the results of test. An acceptable stress-strain curve is given in Fig. 16. For design purposes the partial safety factor γ_m of 2.0 shall be applied.

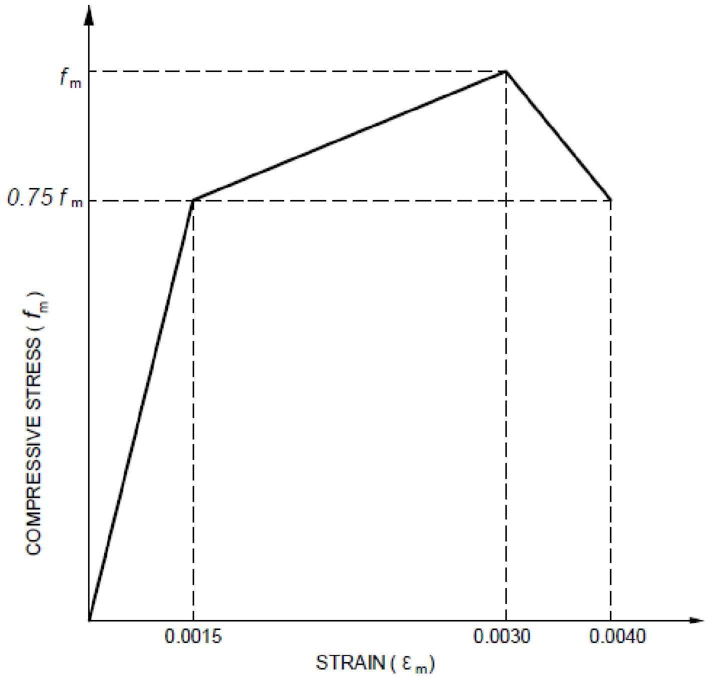


FIG. 16 ACCEPTABLE STRESS-STRAIN CURVE FOR MASONRY

9.2.3.7 *Axial load resistance*

Axial load resistance of a confined masonry wall (P_u) shall be determined considering the contribution of masonry and longitudinal steel reinforcement in tie-columns assuming that the steel has yielded in compression, that is:

$$P_u = k_s\left(0.4 f_m A_m + 0.45 f_{ck} A_c + 0.75 f_y A_s\right),$$

where
A_m = Net area of masonry;
A_c = Cross-sectional area of concrete excluding reinforcing steel;
A_s = Area of steel;
f_{ck} = Characteristic cube compressive strength of concrete;
f_y = Characteristic yield strength of the reinforcing steel;
f_m = Compressive strength of masonry; and
k_s = Stress reduction factor as in Table 9 of IS 1905.

9.2.3.8 *Design of confined masonry walls for combined axial load and out-of-plane bending*

Confined masonry walls need to be designed for combined effects of axial load and out-of-plane bending (bending perpendicular to the wall surface). Bending moments may be due to eccentric gravity loads or lateral loads acting perpendicular to wall plane (wind and earthquake). Generally, confined masonry walls have sufficient resistance for earthquake force acting in out-of-plane direction if slenderness limits in **6.5** are satisfied. For out-of-plane forces, only one-way bending may be considered, that is, bending in the vertical direction as shown in Fig. 17.

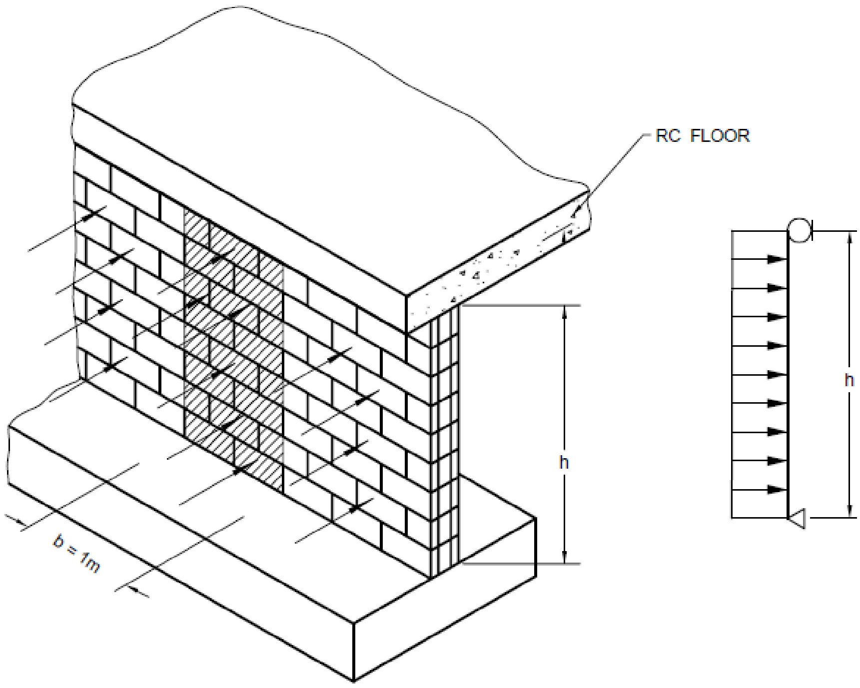


FIG. 17 UNIT LENGTH B OF MASONRY WALL CONSIDERED FOR OUT-OF-PLANE BENDING IN THE VERTICAL DIRECTION

The interaction of compression due to axial stresses and bending stresses should be within the following limits:

$$\frac{P}{P_u} + \frac{M}{M_u} \le 1.0 .$$

Factored bending moment *M* due to design factored wind pressure p_d [as per IS 875 (Part 3)] acting out-of-plane over unit length *b* can be calculated as follows:

$$M = \frac{p_d h^2}{8},$$

where *h* is floor-to-floor height. The axial resistance P_u shall be calculated as in **9.2.3.7** by ignoring the contribution of concrete and reinforcement in tie-column and considering $A_m^{'}$. The moment of resistance M_u per unit length shall be estimated for bending in vertical direction as follows:

$$M_u = \left(\frac{f_t}{\gamma_m} + \frac{P}{A_m^{'}}\right) S_m^{'},$$

where axial load *P* should not exceed self-weight of the unit length of panel plus $0.15A_m^{'}$ MPa, wherein,

$A_m^{'}$ = Net area of masonry per unit length (mm^2);

$S_m^{'}$ = Section modulus per unit length (mm^3); and

f_t = Flexural tensile strength normal to bed joints
= 0.30 MPa for Grade M1 or better mortars, and 0.20 MPa for Grade M2 mortar.

9.2.3.9 *Moment resistance of confined masonry walls due to combined axial load and in-plane bending*

a) *General Approach*

Moment resistance M_u for a section of a confined masonry wall subjected to the design axial load P shall be determined from equilibrium of internal forces acting on the section, by following the assumptions stated in **9.2.3.5**.

b) *Simplified axial load and bending moment interaction diagram*

For walls with longitudinal bars placed symmetrically in tie-columns, the following equations define a simplified axial load-bending moment interaction diagram (Fig. 18):

i) Part A-B: when $0 \le P < (P_u/3)$

$$M_u = \lfloor 0.30Pd + M_{uf} \rfloor$$

where $M_u = 0.87 f_y A_s (l_w - b)$ is the moment resistance corresponding to pure bending load condition, and d the effective depth of the wall section.

ii) When $P = (P_u/3)$

Point B: $M_u = \lfloor 0.10Pd + M_{uf} \rfloor$, and Point C:

$$M_u = \left[0.05Pd + 1.5M_{uf}\right]\left(\frac{2}{3}\right)$$

iii) Part C-D: when $P > (P_u/3)$

$$M_u = \left[0.15Pd + 1.5M_{uf}\right]\left(1 - \frac{P}{P_u}\right)$$

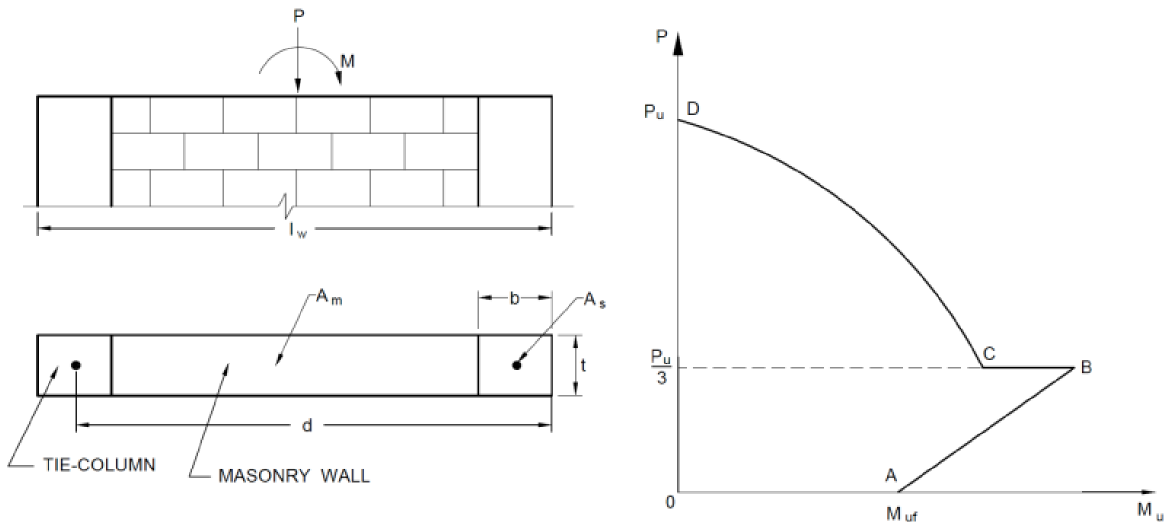


FIG.18 SIMPLIFIED INTERACTION DIAGRAM FOR DESIGN OF CONFINED MASONRY WALLS

c) *Alternate Method*

The P-M interaction curve for the confined masonry section can also be determined by considering the incremental values of neutral axis across the section. But this will involve lengthy calculation by trial and error. The P-M interaction curve can be obtained by using sectional analysis programs available for reinforced concrete members.

9.2.3.10 *Design for shear*

Shear resistance of a confined masonry wall is due to combined contribution of masonry and horizontal wall reinforcement. Although reinforced concrete tie-columns contribute to shear resistance of a confined masonry wall, their contribution is not considered in the design to increase a safety margin in a confined masonry structure.

a) *Masonry Shear Resistance*

Shear resistance V_u provided by masonry shall be determined as:

$$V_u = 0.8(0.5 v_m A_T + 0.4 P_d) f \leq 1.5 v_m A_T$$

where,

P_d = Design compressive axial load which shall include permanent loads only and with the partial safety factor of 1.0,

A_T = Area of cross-section of confined masonry wall including tie-columns and

v_m = Masonry shear strength $= 0.16\sqrt{f_m} \leq 0.6$ (MPa)

f = Corrected factor for the aspect ratio *H*/*L* of the wall

$$= \begin{cases} 1.55 & (H/L) \leq 0.2 \\ 1.7 - 0.7\left(\dfrac{H}{L}\right) & 0.2 < (H/L) \leq 1.0 \\ 1.0 & (H/L) > 1.0 \end{cases}$$

Alternatively, v_m can be obtained from the standard test method for diagonal tension (shear) in masonry assemblages as given hereunder:

i) When shear strength v_m of masonry is to be established by tests, it shall be done in advance of the construction, using masonry specimens built of similar materials under the same conditions with the same bonding arrangement as for the structure. In making the specimens, moisture content of the units at the time of laying, the consistency of the mortar, the thickness of mortar joints and workmanship shall be the same as will be used in the structure.

   The walls specimens (Fig. 19) shall have a length at least two and a half times the length of the masonry unit in one direction and adequate number of courses in perpendicular direction so that the specimen has approximately square shape. The sides of the specimen may not be less than 600 mm. The walls specimens shall be tested by loading them in compression along one diagonal. Tests shall be conducted on at least three specimens constructed with the same size and type of masonry units, mortar, and workmanship.

   Specimens shall be tested after 28 days and two steel loading shoes (Fig. 19) shall be used to apply the machine load to the specimen. For the distribution of the applied load *P*s*,* the ratio of the length of bearing, *l*b and the width or height of specimen must be equal to or greater than 0.2. The load shall be applied at the rate of 50 kN/min to 100 kN/min and the load at failure should be recorded. Load should be applied at a uniform rate so that the maximum load is reached in not less than 1 min and not more than 2 min. Minimum thickness of steel plate used for loading shoe shall be 12 mm.

ii) Estimate the shear stress v_m of the specimen as:

$$v_m = \frac{0.707 P_s}{A_{sn}},$$

   where

   P_s = Applied load (N), and

   A_{sn} = Net area (mm^2) of the specimen

   $$= \left(\frac{w_s + h_s}{2} \right) t_s,$$

   wherein w_s is the width (mm), h_s the height (mm), and t_s the total thickness (mm) of specimen.

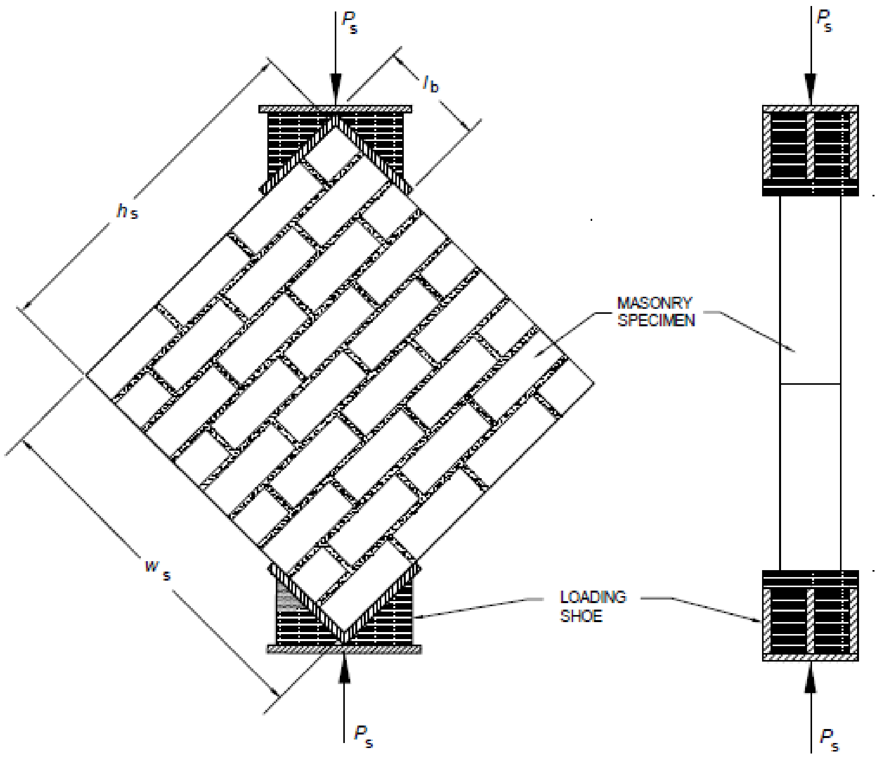


Fig. 19 Wall specimen for diagonal compression test

(b) *Alternate Method (Strut-and-Tie method)*

The Strut-and-Tie method can be used for determining the shear capacity of confined masonry walls. This method provides a rational and consistent design approach by idealizing complex structural members with an appropriate simplified truss model. According to this procedure, based on the knowledge of direction of principal stresses, load paths are drawn through the wall in form of a truss which is analyzed for the design loads. A possible strut-and-tie model for the confined masonry wall based on its load resisting mechanism is illustrated in Fig. 20; the broken and solid lines represent the *strut* and *tie*, respectively. If the opening distribution is irregular or complex in elevation, strut and tie method can be used to estimate the in-plane resistance of the confined masonry wall.

The basic prerequisites for the strut-and-tie model are as follows:

a) The equilibrium of forces at nodes should be maintained for a given set of loads.
b) Tension in concrete and masonry is neglected and adequate detailing of anchorage should be provided for tie reinforcement.
c) The member forces in the struts and ties are uniaxial which should not exceed the corresponding member strength at every section.
d) Sufficient ductility should be available to make the transition from elastic to plastic behaviour which will enable redistribution of internal forces in the members.

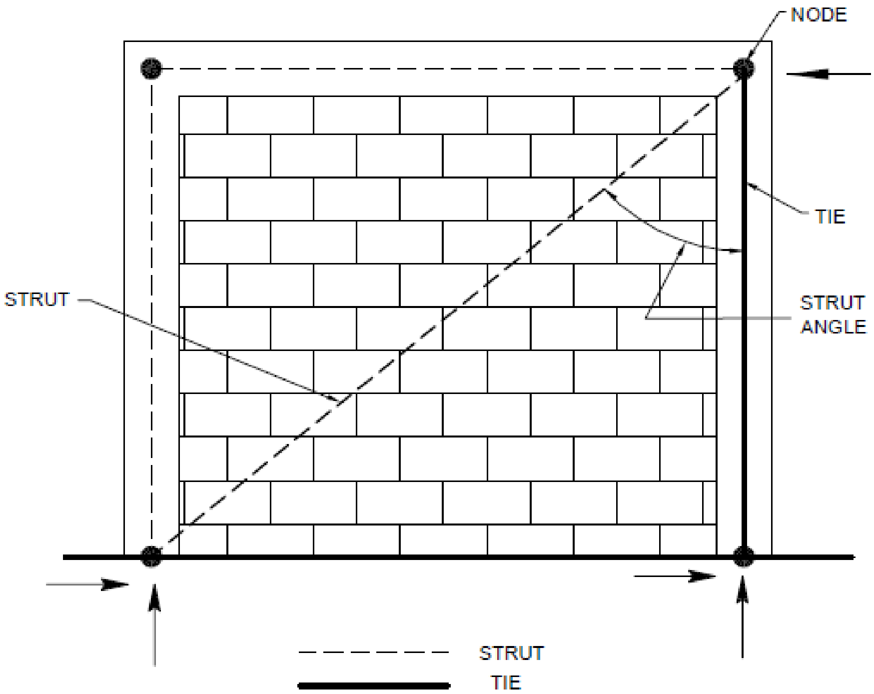


FIG. 20 STRUT-AND-TIE MODEL OF CONFINED MASONRY WALLS

9.2.3.11 *Design of tie-columns and tie-beams*

a) *Minimum amount of longitudinal reinforcement*

Longitudinal reinforcement in tie-columns and tie-beams shall be proportioned to resist the corresponding vertical and horizontal components of the compression strut that develops in masonry when resisting combined gravity and lateral loads. The total area of reinforcement should be not less than 0.8 percent of the gross cross-section area of the column. For a 2-storey height, the minimum area of reinforcement can be 0.6 percent.

b) *Minimum amount of transverse reinforcement (Ties)*

Tie-columns and tie-beams shall have transverse reinforcement in the form of closed stirrups (ties) with the minimum total cross-sectional area (mm^2) of stirrup legs effective in shear equal to:

$$A_{sc} = 0.0012 s h_c ,$$

where, h_c is the dimension (in mm) of tie-column or tie-beam in the wall plane, and s the tie spacing (in mm).

c) *Spacing of transverse reinforcement*

Tie spacing shall not exceed the lesser of 200 mm and 1.5*t*.

9.2.4 *Foundations and Plinth Construction*

The foundation should be constructed in a similar manner as for traditional masonry construction. Either a brick masonry footing or an RC strip footing can be used. An RC plinth band should be constructed on top of the foundation. A plinth band is essential to fully confine wall panels along their bases and prevent excessive wall damage due to building settlement in soft soil areas. Note that the longitudinal reinforcement should be extended from an RC tie-column into the plinth band, and whenever possible, into the foundation.

9.3 Earthquake Resistant Design

9.3.1 *Basis*

Structure should be designed for seismic forces determined as per CED 39 (22343).

9.3.2 *Methods of Analysis*

9.3.2.1 *General considerations*

9.3.2.1.1 For determining internal forces and moments acting on confined masonry walls, the structures may be analyzed either using the simplified method of seismic analysis, or advanced analysis methods (static and/or dynamic analysis). The simplified method can be used for buildings up to three storeys high (G + 2), provided that the requirements stipulated in **9.3.2.2** have been met. The effect of openings shall be considered in lateral stiffness and strength calculations. The structure shall be modelled to adequately simulate the behaviour of its critical structural elements. Modulus of elasticity E_m and shear modulus G_m of masonry shall be used to reflect the axial and shear stiffness expected in the structure.

9.3.2.1.2 Design forces and moments shall be obtained from the analysis using design loads and the corresponding load factors. A confined masonry wall shall be designed for the effects of gravity loads, and the effects of lateral loads, including shear force, in-plane bending, and out-of-plane bending moments. When the simplified method is used, design for lateral loads may be limited to the effects of shear force.

9.3.2.2 *Simplified method*

9.3.2.2.1 The simplified method is based on an idealized distribution of lateral seismic forces in regular bearing wall structures with rigid diaphragms. The method compares the shear capacity of all walls at the base of the building and the earthquake base shear force determined from CED 39 (22343). It is suitable for verifying adequacy of regular buildings with symmetrical wall layout and predominant shear behaviour. The method does not consider torsional effects. Due to its assumptions, application of this method is restricted to buildings with a height not exceeding three storeys (G + 2).

9.3.2.2.2 The design can be simplified further for small buildings by applying the concept of wall index, as per procedure in **9.3.2.3**, which can be used for the design of buildings with the total ground plan area not exceeding 200 m^2 and the height not exceeding two storeys or 7 m.

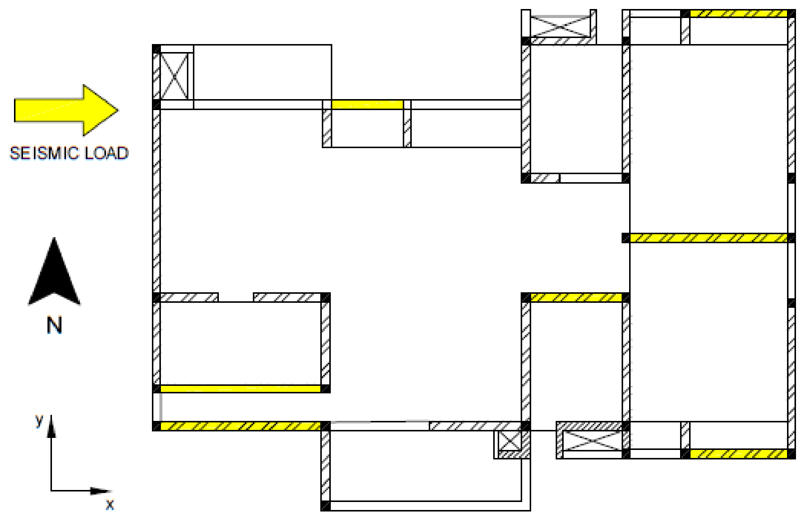
9.3.2.3 Wall Index (*WI*), also known as wall density, is an indicator of earthquake load-resisting capacity of a masonry building and can be used for seismic design of low-rise buildings with regular plan shape and elevation. The design according to the simplified method is deemed satisfactory provided that the actual wall index in each direction of the building plan is greater than or equal to the required wall index (WI_{req}) for specific building.

Wall index WI_{floor} per floor is ratio of the sum of cross-sectional areas of all confined masonry walls in the direction considered for seismic load relative to the ground floor plan area, that is:

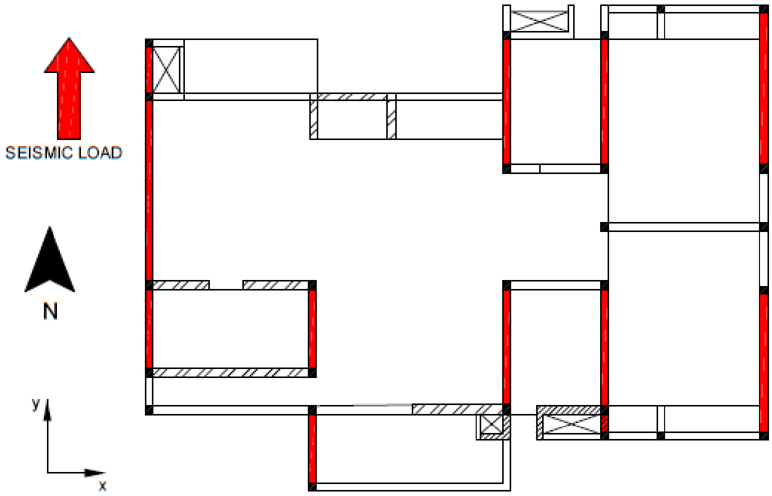
$$WI_{floor} = A_w/A_p$$

where A_w is the cross-sectional area of all confined walls in one direction at the ground floor level. The cross-sectional area of a confined masonry wall is the product of its length (including the reinforced concrete tie-columns and the masonry wall) and thickness. A_p is the plan area of the floor diaphragm (floor slab) at the ground floor level. The area of any cutouts (openings) in the floor slab should be deducted in the calculation of A_p.

Parameters for wall index calculations are illustrated in Fig. 21. The wall index shall be calculated for *each* principal direction of the building plan (*X* and *Y*). Only confined masonry walls aligned in the direction under consideration (shown shaded in Fig. 21) should be considered in the A_w calculations for the specific direction.



21 A



21 B

FIG. 21 PARAMETERS FOR THE WALL INDEX CALCULATIONS FOR THE PRINCIPAL DIRECTIONS OF A BUILDING PLAN (X AND Y)

9.3.2.3.1 The required wall index WI_{req} for a building shall account for the building height in terms of the number of floors, as follows:

$$WI_{req} \geq nWI_{floor},$$

where *n* is the number of floor levels in a building, for example, *n* = 3 for a G + 2 building. WI_{req} increases in direct proportion with the number of floor levels in a building, and its value for a specific building depends on the following parameters:

a) Earthquake Zones III, IV, V and VI and soil type – expressed through Design Horizontal Acceleration coefficient A_h determined from CED 39 (22343);
b) Number of floor levels in a building, *n*;
c) Compressive strength of bricks/blocks and mortar mix; and
d) Average floor weight, *w* in kN/m^2, which includes the self-weight of floor slab, flooring, and walls, plus 25 percent of the imposed load.

The required values of WI_{floor} for different earthquake zones of India are given in Table 10; these were obtained based on the Limit State Method of Design, and assuming masonry unit compressive strength of 5.0 MPa, mortar type M1 as per IS 1905, and average floor weight *w of* 12 kN/m^2.

Alternatively, the values of WI_{floor} can be calculated from as follows:

$$WI_{floor} \geq \frac{1.5A_h w}{(v_m/\gamma_m)},$$

where v_m is masonry shear strength as in **9.2.3.10**, w the average floor weight, A_h the design horizontal acceleration coefficient determined as per CED 39 (22343), γ_m the partial safety factor for masonry as given in **7.3.2**.

The Design Base Shear Force V_B as per CED 39 (22343) is given by:

$$V_B = A_h W_T,$$

where W_T is the seismic weight as per CED 39 (22345).

Different masonry properties (masonry unit compressive strength and mortar type) can be used, and the corresponding masonry shear strength v_m determined from **9.2.3.10**.

Table 10 WI_{floor} for different Earthquake Zones
(*Clause* 9.3.2.3.1)

Earthquake Zone	**III**	**IV**	**V**
WI_{floor} (percent)	*1.1*	*1.6*	*2.4*

9.3.2.3 *Advanced Analysis Methods*

Seismic analysis of confined masonry buildings of four-storey or higher and buildings up to three-storey high that do not meet requirements for the application of simplified method shall be performed using dynamic or static methods according to CED 39 (22343). Lateral load effects induced by earthquake shall be determined based on the relative stiffness of walls and wall segments, by considering the effect of both shear and flexure in stiffness calculations. The analysis model shall consider stiffness of floor and roof systems, and any other restraints that may influence wall rotations.

9.3.3 *Other Seismic Design Requirements*

9.3.3.1 *Wall spacing*

The maximum spacing of transverse walls in buildings with flexible diaphragms shall not exceed 4 m. For spacing more than 4 m, the tie-beam at roof level should have a width of L/20.

9.3.3.2 *Spacing of Transverse Reinforcement (Ties) in Tie-columns*

For seismic zones IV and V, reduced tie spacing (s/2) is required at the ends of tie-columns, as shown in Fig. 22. The length over which the reduced tie spacing is used shall not exceed the largest of the following values: twice the column dimension ($2h_c$ or $2t$), or $h_o/6$, where h_o is the tie-column clear floor height.

9.4 General Requirements

9.4.1 *Reinforcement Detailing*

9.4.1.1 *Concrete cover*

a) The minimum concrete cover to ties (transverse reinforcement) in tie-columns and tie-beams shall not be less than 20 mm.

b) When horizontal wall reinforcement is provided, the minimum clear distance between a horizontal bar and the exterior wall surface shall not be less than 20 mm.

9.4.1.2 *Bar Size and number in tie-columns and tie-beams*

Longitudinal reinforcement in tie-columns and tie-beams shall consist of a minimum of 4 reinforcing bars with the minimum 10 mm diameter.

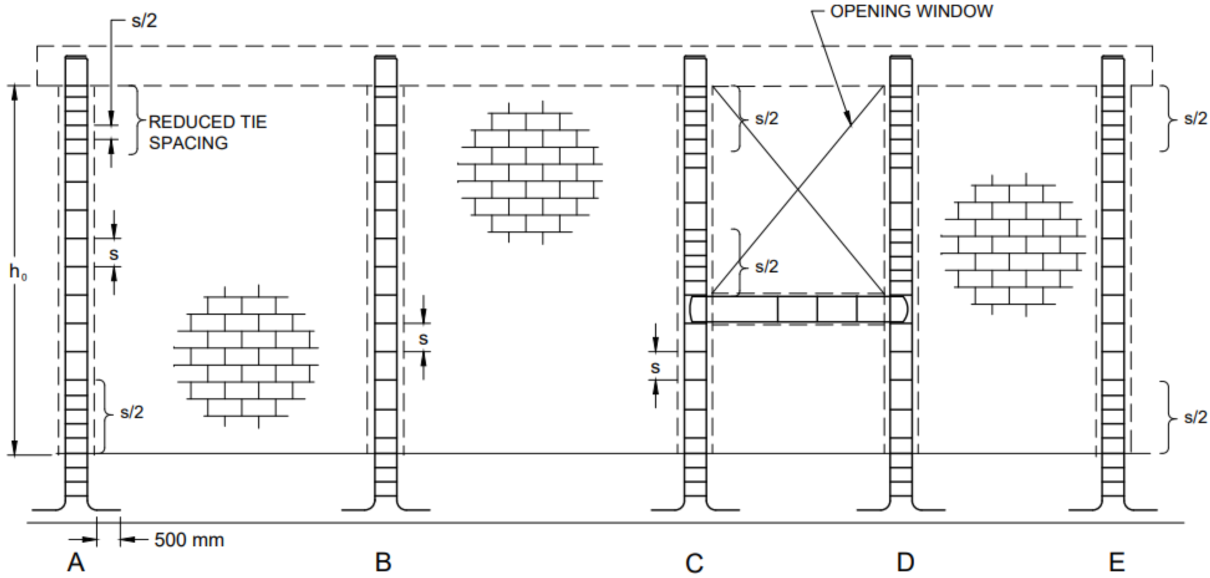


FIG. 22 TIE-COLUMN TRANSVERSE REINFORCEMENT REQUIREMENTS FOR EARTHQUAKE ZONES IV, V AND VI

9.4.1.3 *Ties in tie-columns and tie-beams*

Minimum 6 mm diameter bars shall be used for ties in tie-columns and tie-beams (Fig. 23). Ties shall have 135° hooked ends. Hooks shall be staggered as shown in Fig. 23.

9.4.1.4 *Anchorage of Longitudinal Bars*

a) Longitudinal reinforcement in tie-columns and tie-beams shall be anchored to develop the full specified steel yield stress. Longitudinal bars in tie-columns and tie-beams shall have a 90° hooked anchorage at intersections. The lap length of the hook tails shall be the largest of 20 times the bar diameter or 500 mm (Fig. 24).
b) Tie-column longitudinal bars at the roof level shall be bent by 90 degrees and lapped with the tie-beam longitudinal reinforcement (Fig. 25).

9.4.1.5 *Lap Splices for Longitudinal Reinforcement*

Lap splices for longitudinal reinforcement should be at least 40 times the bar diameter. Longitudinal reinforcing bars should be spliced within the middle third of the column height or beam span. The splices should be staggered so that not more than 2 bars are spliced at any one location.

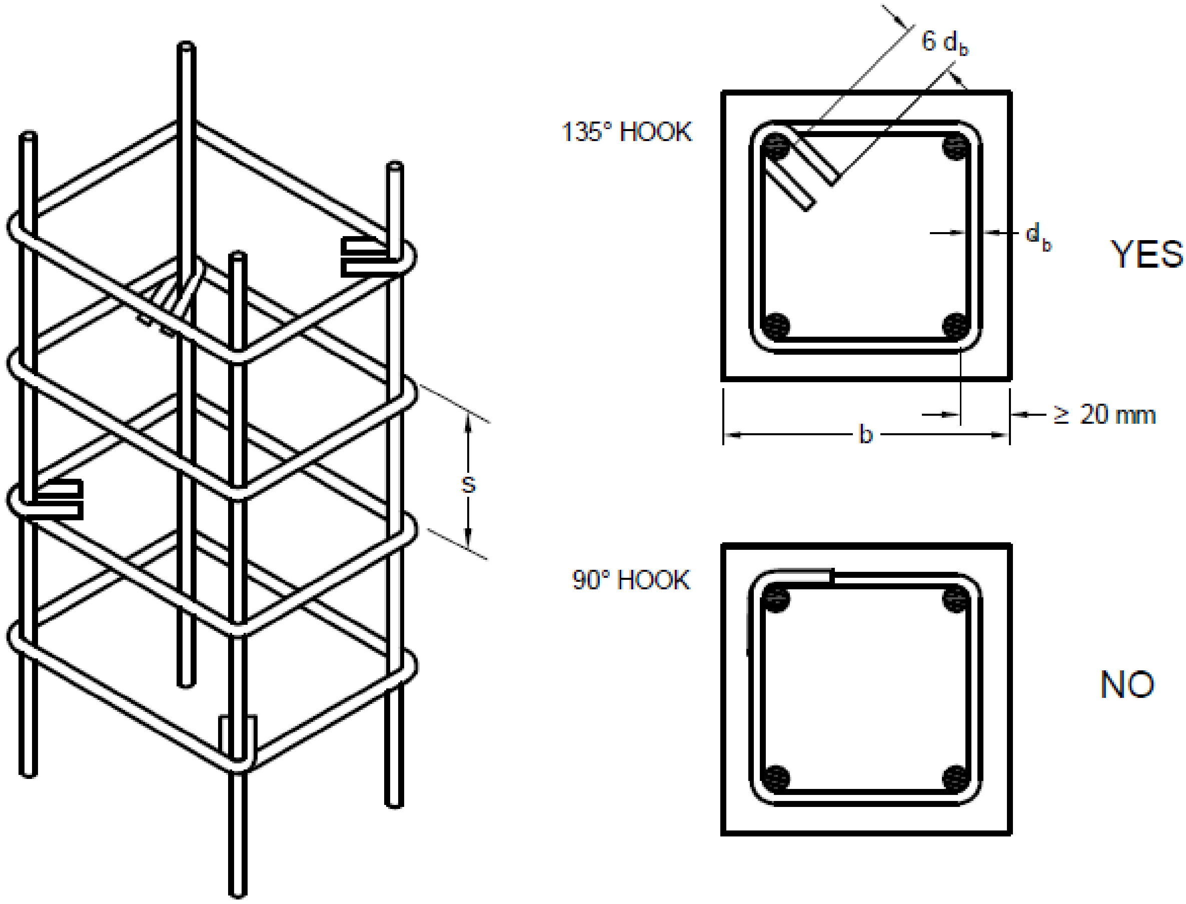
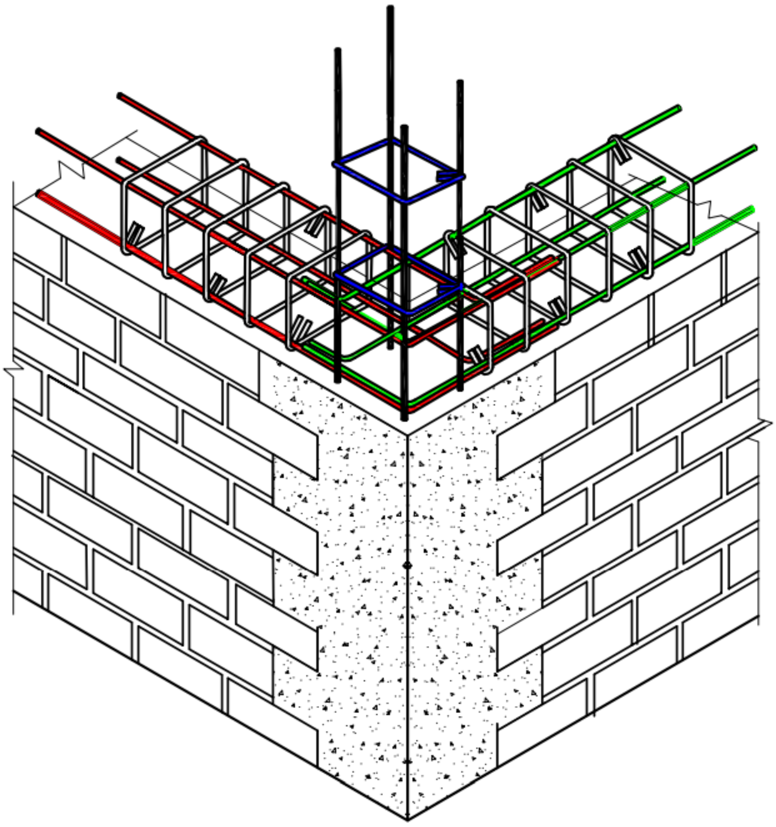
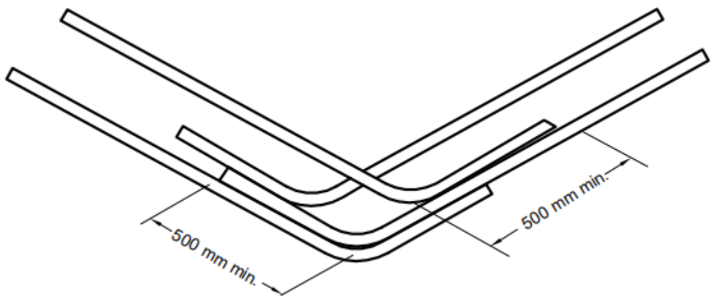


FIG. 23 LAYOUT AND DETAILING OF TIE-COLUMNS AND TIE-BEAMS



24A WALL INTERSECTION



24B HOOKED ANCHORAGE FOR LONGITUDINAL REINFORCEMENT AT WALL INTERSECTIONS

FIG. 24 TIE-BEAM CONSTRUCTION

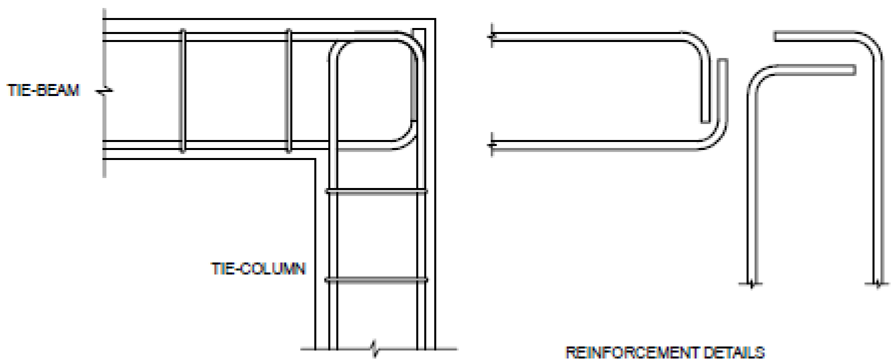


FIG. 25 ANCHORAGE OF TIE-COLUMN AND TIE-BEAM LONGITUDINAL REINFORCEMENT

9.4.1.6 *Tie-Column-to-Tie-Beam Joints*

Continuity of longitudinal tie-beam reinforcement through the joint must be ensured. An example of a continuous longitudinal reinforcement is shown in Fig. 26. First tie at the ends of tie-columns (top and bottom) shall be placed as close to the joint as possible. When tie-beam depth exceeds 300 mm, vertical reinforcement in an RC tie-column must be confined by the ties, below and above the joint. An additional U-shaped stirrup must be placed at the tie-beam mid-height (Fig. 27).

Reinforced concrete tie-columns shall be provided at the openings as shown in Fig. 14B and Fig. 14C. A nominal reinforcement around openings as detailed in Fig. 13 shall be provided. When reinforced concrete tie-columns are not provided at the ends of an opening then they should be strengthened as per IS 4326 or minimum two vertical bars of 10 mm diameter with 6 mm ties at 150 mm spacing should be provided.

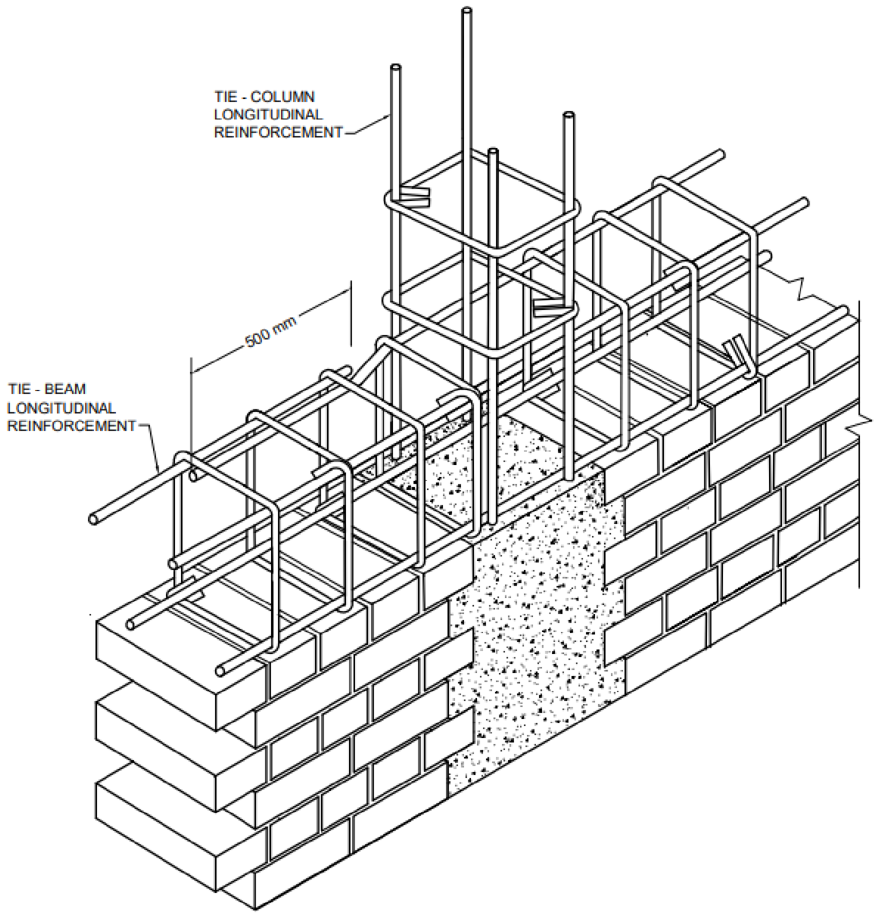


FIG. 26 CONTINUITY OF TIE-BEAM REINFORCEMENT THROUGH JOINT

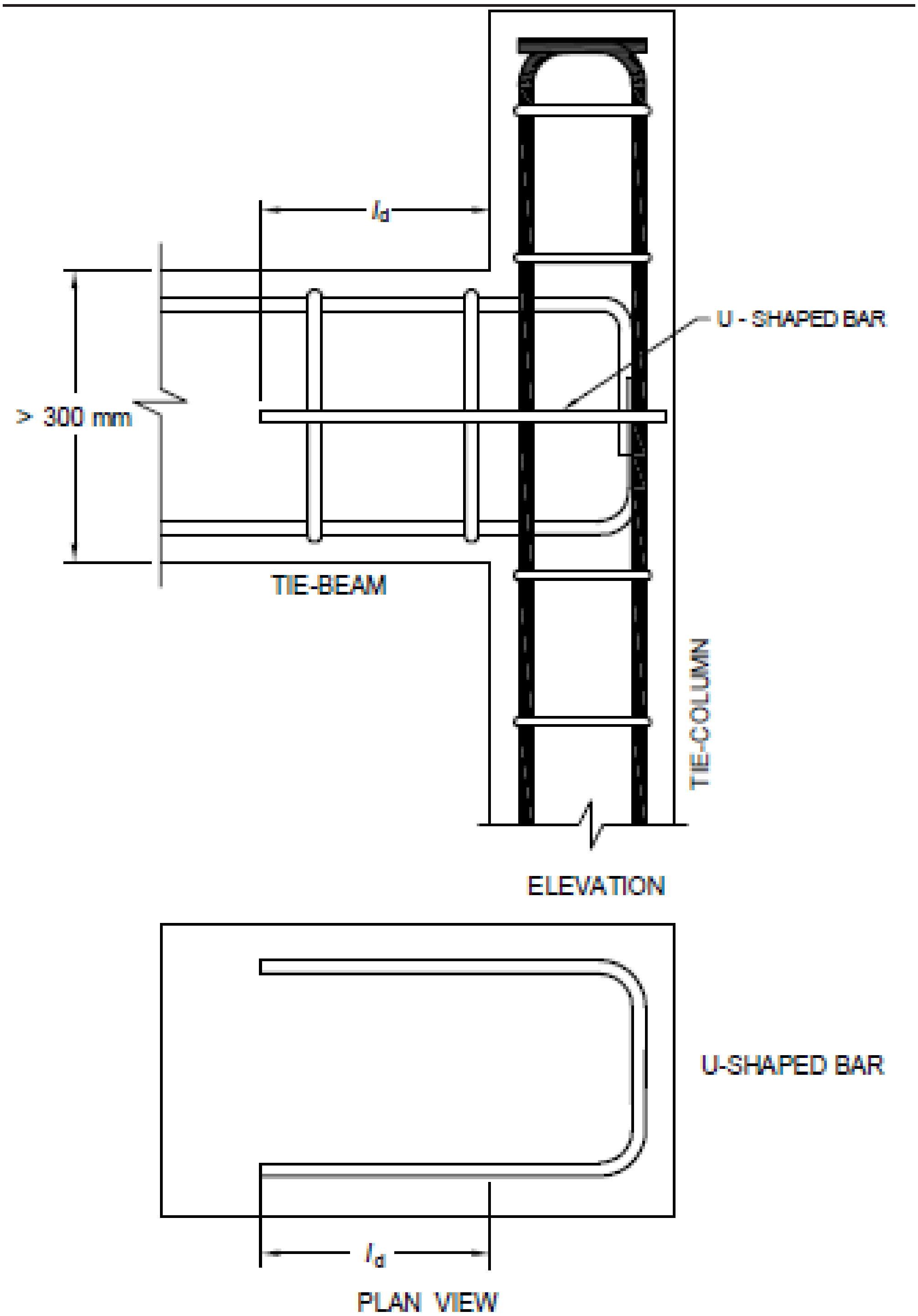


FIG. 27 ADDITIONAL CONFINEMENT FOR VERTICAL REINFORCEMENT IN THE TIE-BEAM AND TIE-COLUMN END JOINT REGION

9.4.2 *Construction*

Construction of confined masonry buildings shall satisfy the conditions specified in relevant Indian Standards.

9.4.2.1 *Masonry wall construction*

a) *General*

During construction, in addition to the requirements of the sections above, the following shall be complied:

a) All intersecting walls shall be connected, unless measures to assure stability and good performance are taken.
b) Surfaces of construction joints shall be clean and rough. The joint shall be moistened before construction when clay units are used.
c) To fulfill the requirements of excellent masonry work, Flemish bond or English bond in brickwork shall be adopted.
d) During construction, necessary caution to assure wall stability at the job, shall be taken, possible horizontal pressure and loads, including wind and earthquake shall be considered.

b) *Masonry units*

The shape and dimension of masonry units, construction practices, including methods of positioning of reinforcement, placing and compacting of grout, as well as design and detailing should be, such as to promote homogeneity of structural members, development of the bond between the grout to both reinforcement and masonry units and avoidance of corrosion of reinforcement.

c) *Mortar joint thickness*

Joint mortar shall totally cover the horizontal and vertical sides of the masonry unit. Joint thickness shall be the minimum to get a uniform layer of mortar and good alignment of the units. If industrialized units are used, the thickness of the horizontal joints shall not exceed 12 mm if horizontal reinforcement is placed in the joints, and 10 mm without horizontal reinforcement. Minimum thickness shall be 6 mm.

d) *Toothing*

The vertical interface of masonry with adjacent tie-columns shall be detailed to transfer shear forces. It shall be accepted to indent the masonry (referred to as toothing) or else, to place steel dowels or horizontal reinforcement as shown in Fig. 28.

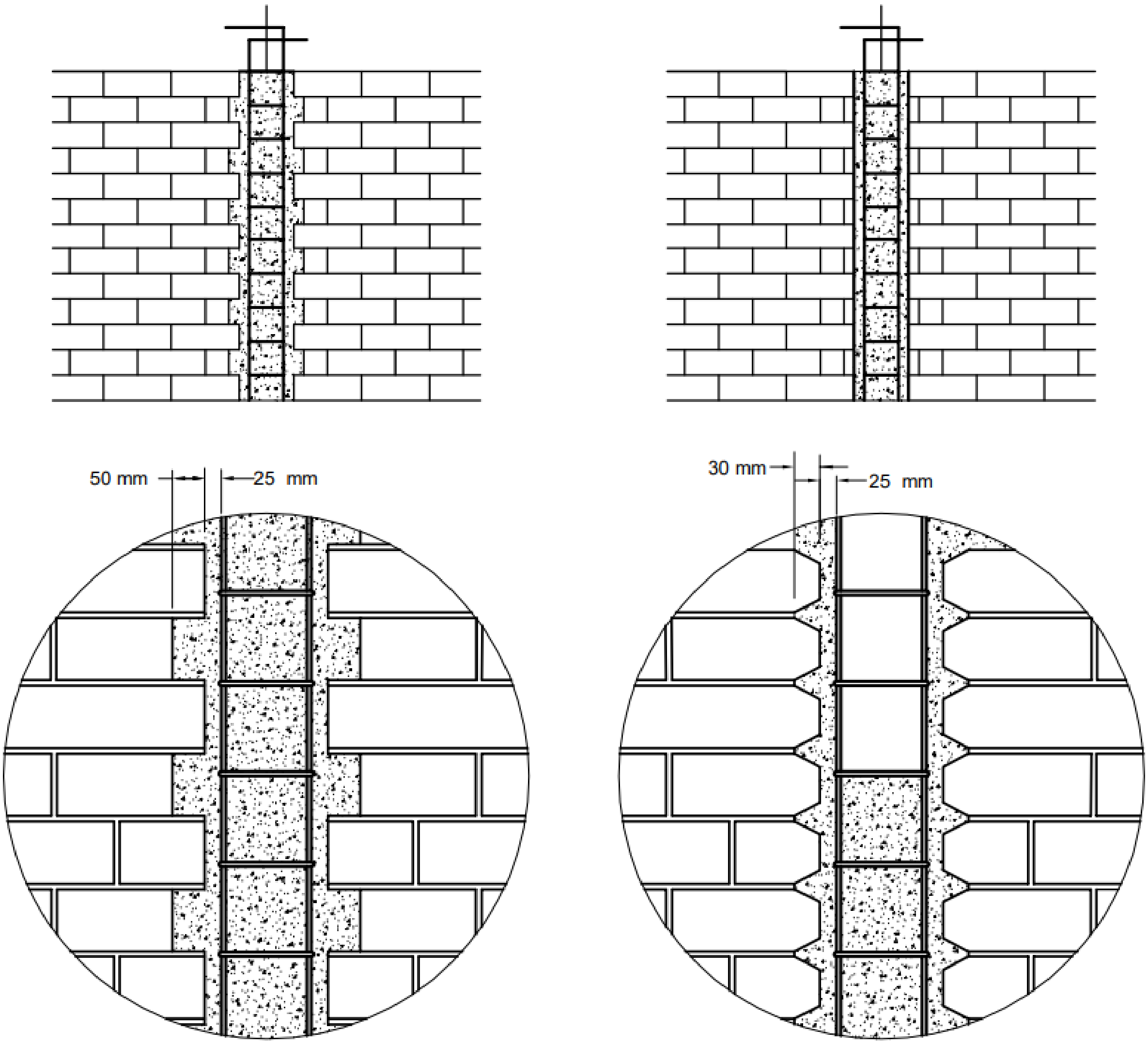
Toothed edges shall be left on each side of the wall at the interface with the tie-columns. Toothing length shall be equal to one-quarter of the masonry unit length, but not less than 50 mm, as shown in Fig. 28A. When hand-made bricks are used, it is desirable to cut the brick edges, as shown in Fig. 28B. It is important to clean the surfaces of “toothed” masonry units before the concrete has been poured. Horizontal reinforcement anchored into RC tie-columns, also known as dowels, can be used as an alternative to toothing, as shown in Fig. 28C. The dowels are not necessary when toothed edges are used. 6 mm diameter bars shall be preferably used as dowels.

9.4.2.2 *Reinforcement*

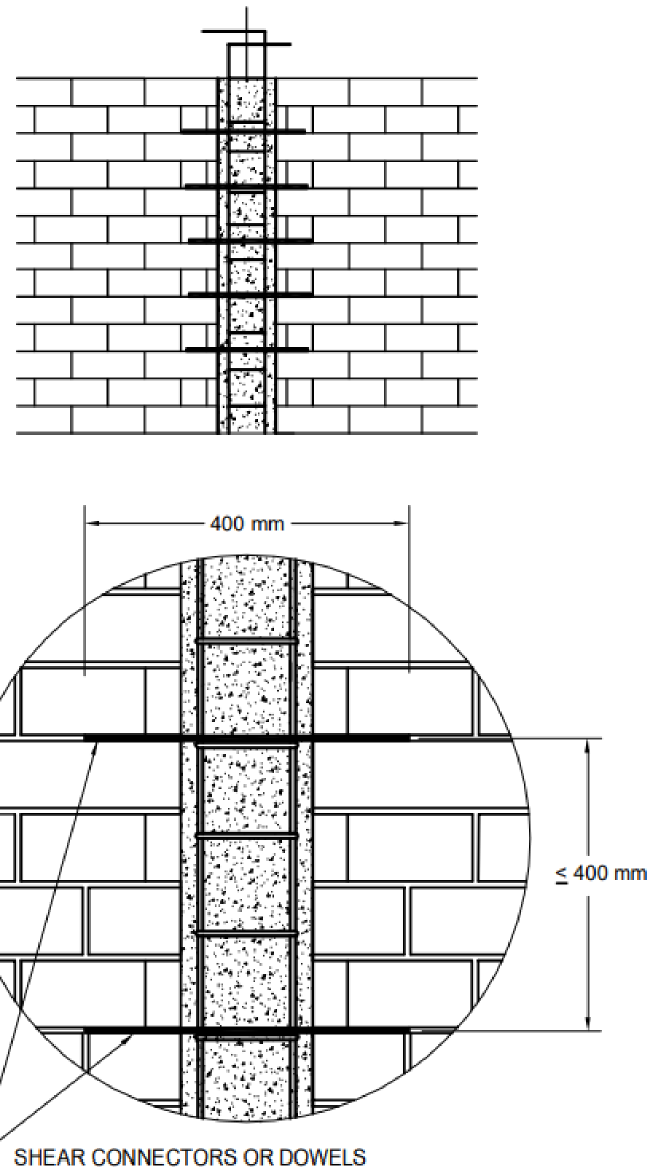
Reinforcement shall be tightly secured before casting.

9.4.2.3 *Concrete*

Casting of tie-columns shall be made once the masonry wall or the corresponding part has been constructed. The masonry around the tie-columns shall be saturated before the concrete has been poured.



28A MACHINE MADE HOLLOW UNITS　　28B HANDMADE HOLLOW UNITS



28C PROVISION OF HORIZONTAL REINFORCEMENT WHEN TOOTHING IS NOT POSSIBLE

FIG. 28 TOOTHING IN CONFINED MASONRY WALLS

9.4.2.4 *Piping and ducts*

Piping and ducts shall be installed without damaging the masonry. If solid or grouted hollow units are used, grooves in the wall shall be permitted to embed piping and ducts, but the following shall need to be satisfied:

a) Groove depth shall not exceed one fourth the thickness of the masonry wall (t/4);
b) Groove shall be vertical; and
c) Groove shall not be longer than one half the free height of the wall (H/2). If hollow units are used, pipes or ducts shall not be placed in cells with reinforcement. Cells with pipes and ducts shall be grouted.

It shall not be permitted to place piping and ducts in tie-columns with a structural function, whether external or internal.

10 REINFORCED MASONRY BUILDINGS

10.1 General

This section gives the recommendations for structural design aspect of reinforced load bearing/walls, constructed with solid or perforated burnt clay bricks, sand-lime bricks, stones, concrete blocks, lime based blocks or burnt clay hollow blocks with regard to the materials to be used, maximum permissible stresses and methods of design.

10.2 Design Considerations

10.2.1 *General*

Design considerations shall be as per geometrical requirements of IS 1905 along with the following provisions.

10.2.2 *Structural Continuity*

Intersecting structural elements intended to act as a unit shall be joined together to resist the design forces. Walls shall be joined together to all floors, roofs or other elements which provide lateral support for the wall. Where floors or roofs are designed to transmit horizontal forces to walls, the anchorages to the walls shall be designed to resist the horizontal forces.

10.2.3 *Effective Span*

10.2.3.1 The effective span of simply supported/continuous members may be taken as the smaller of the following:

a) Distance between centers of supports, and
b) Clear distance between supports plus an effective depth *d*.

10.2.4 *Slenderness Ratio*

10.2.3.2 Effective span of a cantilever shall be taken as greater of distance between the end of cantilever and:

a) the center of its support; and
b) the face of support plus half its effective depth.

10.2.4.1 *Wall*

The slenderness ratio (ratio of effective height h_{ef} to effective thickness t_{ef}) should not exceed 27 for vertically loaded reinforced masonry walls in their plane. For reinforced masonry members such as walls subjected to out-of-plane bending and for beams as a part of wall subjected to bending in the plane of the wall, the maximum effective span to effective depth ratio shall be as given in Table 11.

Table 11 Maximum Effective Span to Effective Depth Ratio for Walls subjected to Out-of-Plane Bending
(*Clause* 10.2.4.1)

Sl No.	Boundary Conditions	Maximum Effective Span to Effective Depth (l_{ef} /d) or Effective Thickness Ratios (l_{ef} /t_{ef})	
		Wall subjected to out-of-plane bending	Beam part of wall subjected to in-plane bending
(1)	(2)	(3)	(4)
i)	Simply supported	35	20
ii)	Continuous	45	26
iii)	Spanning in 2 directions	45	-
iv)	Cantilevered	18	7

10.2.4.2 *Columns*

For a column, slenderness ratio shall be taken to be the greater of the ratios of effective heights to the respective effective thickness in the two principal directions. Slenderness ratio for a load bearing unreinforced column shall not exceed 15 whereas for reinforced column the slenderness ratio should be limited to 20.

10.2.5 *Minimum Design Dimensions*

10.2.5.1 *Minimum thickness of load bearing walls columns*

The nominal thickness of masonry bearing walls in building shall not be less than 230 mm.

10.2.5.2 *Parapet wall*

Parapet walls shall be at least 200 mm thick and height shall not exceed 3 times the thickness. The parapet wall shall not be thinner than the wall below.

10.2.6 *Eccentricity in Columns*

Columns shall be designed for a minimum eccentricity of 10 percent of side dimension for each axis in addition to applied loads.

10.3 Requirements Governing Reinforcement and Detailing

10.3.1 *General*

This section provides requirements for the working (allowable) stress design of masonry structure neglecting the contribution of tensile strength of masonry.

10.3.1.1 Members are designed for composite action. Stresses shall be computed using transformed area concept of linear elastic analysis as follows:

$$A_t = A_b + m\,A_s$$

where

A_t = Total transformed cross-sectional area of the member,

A_b = Cross sectional area of brick,

A_s = Cross sectional area of reinforcement, and

m = Modular ratio of steel reinforcement and brick.

10.3.1.2 Stiffness calculation shall be based on un-cracked section properties.

10.3.2 *Steel Reinforcement-Allowable Stresses*

10.3.2.1 *Tension*

Tensile stress in reinforcement shall not exceed the following:

a) 140 MPa in case of Mild Steel (MS) bars of diameter ≤ 20 mm and 130 MPa in case of MS bars of diameter > 20 mm, and
b) $0.55f_y$ in case high strength bars, where f_y is the characteristic strength of steel.

10.3.2.2 *Compression*

Compressive stress in reinforcement shall not exceed the following:

a) 130 MPa in case of MS bars, and
b) 190 MPa in case high strength bars.

10.3.3 *Size of Reinforcement*

a) The maximum size of reinforcement used in masonry shall be 25 mm diameter bars and minimum size shall not be less than 8 mm.
b) The diameter of reinforcement shall not exceed one-half the least clear dimension of the cell, bond beam, or collar joint in which it is placed.

10.3.4 *Spacing of Reinforcement*

a) Clear distance between parallel bars shall not be less than the diameter of the bars, or less than 25 mm; and
b) In columns and pilasters, clear distance between vertical bars shall not be less than 1.5 times the bar diameter, nor less than 35 mm.

10.3.5 *Anchorage*

10.3.5.1 *Development length of bars*

The development length L_d for deformed bars conforming to accepted standard shall be given by the following equation but shall not be less than 300 mm:

$$L_d = 0.25\, d_b\, F_s$$

where,

d_b= nominal diameter of bar (mm), and

F_s= permissible tensile/compressive stress in steel (MPa).

10.3.5.2 For MS bars and epoxy coated bars, L_d shall be increased by 60 percent.

10.3.5.3 *Standard hooks*

a) Standard hooks shall be formed by one of the following methods (*see* Fig. 29):
   (1) 180° turn plus extension of at least 4 bar diameters but not less than 64 mm at free end of bar.
   (2) A 90° turn plus extension of at least 10 bar diameters at free end of bar.
   (3) For stirrup and tie anchorage only a 90° or a 135° turn plus an extension of at least 5 bar diameters at the free end of the bar.

b) The diameter of bend, measured to the inside of the bar other than stirrups and ties, shall not be less than 5 bar diameters for 6 mm through 20 mm diameter bars. For 25 mm bars through 40 mm, a minimum bend diameter of 6 times bar diameters shall be used.

c) Inside diameter of bend for 12 mm diameter or smaller stirrups and ties shall not be less than 4 bar diameters. Inside diameter of bend for 16 mm diameter or larger stirrups and ties shall not be less than that given in (b).

d) Hooks shall not be permitted in the tension portion of any beam, except at the ends of simple or cantilever beams or at the freely supported ends of continuous or restrained beams.

10.3.5.4 *Anchorage of reinforcing bars*

a) Deformed bars may be used without end anchorages provided development length requirement is satisfied. Hooks should normally be provided for plain bars in tension.
b) Bends and hooks shall conform to shall accepted standard.

   1) *Bends* — The anchorage value of bend shall be taken as 4 times the diameter of the bar for each 45° bend subject to a maximum of 16 times the diameter of the bar.
   2) *Hooks* — The anchorage value of a standard U-type hook shall be equal to 16 times the diameter of the bar.

c) For stirrups and transverse ties complete development length and anchorage shall be deemed to have been provided when the bar is provided with standard hook as described in **10.3.5.3.**

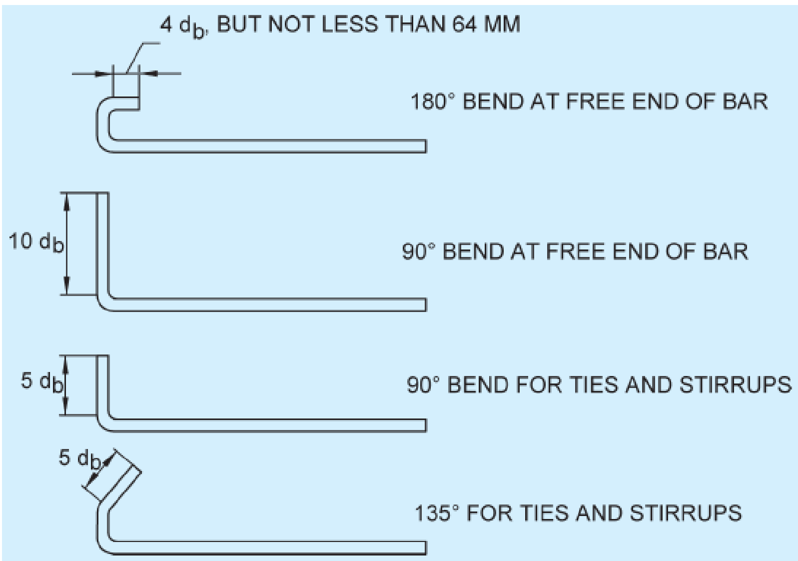


FIG. 29 STANDARD HOOK $4D_B$, BUT NOT LESS THAN 64 MM

10.3.6 *Lap Splices*

10.3.6.1 Where splices are provided in the reinforcing bars, they shall as far as possible be away from the sections of maximum stress and be staggered. It is recommended that splices in flexural members shall not be at sections where the bending moment is more than 50 percent of the moment of resistance; and not more than half the bars shall be spliced at a section. Where more than one-half of the bars are spliced at a section or where splices are made at points of maximum stress, special precautions shall be taken, such as increasing the length of lap and/or using spirals or closely spaced stirrups around the length of the splice.

10.3.6.2 Lap length including anchorage value of hooks for bars in flexural tension shall be L_d (see **10.3.5.1**) or $30d_b$ whichever is greater and for direct tension shall be $2L_d$ or $30d_b$ whichever is greater. The straight length of the lap shall not be less than $15d_b$ or 200 mm.

10.3.6.3 The following provisions shall also apply:

a) Where lap occurs for a tension bar located at:
   1) Top of a section as cast and the minimum cover is less than twice the diameter of the lapped bar, the lap length shall be increased by a factor of 1.4; and
   2) Corner of a section and the minimum cover to either face is less than twice the diameter of the lapped bar or where the clear distance between adjacent laps is less than 75 mm or 6 times the diameter of lapped bar, whichever is greater, the lap length should be increased by a factor of 1.4.
b) Where both conditions (1) and (2) apply, the lap length should be increased by a factor of 2.0.
c) Splices in tension members shall been enclosed in spirals made of bars not less than 6mm diameter with pitch not more than 100 mm.

10.3.7 *Curtailment of Tension Reinforcement*

In any member subjected to bending, every reinforcing bar should extend, except at end supports, beyond the point at which it is no longer needed, for a distance equal to the effective depth of the member or 12 times the diameter of the bar, whichever is the greater. The point at which reinforcement is theoretically no longer needed is where the design resistance moment of the section, considering only the continuing bars, is equal to the applied design moment. However, reinforcement should not be curtailed in a tension zone unless at least one of the following conditions is satisfied for all arrangements of design load considered:

a) Reinforcing bars extend at least the anchorage length appropriate to their design strength from the point at which they are no longer required to resist bending.
b) Design shear capacity at the section where the reinforcement stops are greater than twice the shear force due to design loads, at that section; and
c) Continuing reinforcing bars at the section where the reinforcement stops provides double the area required to resist the bending moment at that section.

Where there is little or no end fixity for a member in bending, at least 25 percent of the area of the tension reinforcement required at mid-span should be carried through to the support. This reinforcement may be anchored in accordance with **10.3.5.4** or by providing:

1) an effective anchorage length equivalent to 12 times the bar diameter beyond the centre line of the support, where no bend or hook begins before the centre of the support, or
2) an effective anchorage length equivalent to 12 times the bar diameter plus $d/2$ from the face of the support, where d is the effective depth of the member, and no bend begins before $d/2$ inside the face of the support.

Where the distance from the face of a support to the nearer edges of a principal load is less than twice the effective depth, all the main reinforcement in a member subjected to bending should continue to the support and be provided with an anchorage equivalent to 20 times the bar diameter.

10.3.8 *Members Subjected to Flexure and Axial Forces*

10.3.8.1 A member which is subjected to axial stress less than $0.1f_m$, may be designed for bending only.

10.3.8.2 *Beams*

Reinforcement in masonry designed as beams shall provided over a support where the masonry is continuous, whether the beam has been designed as continuous or not. Where this occurs, an area of steel not less than 50 percent of the area of the tension reinforcement required at mid-span shall be provided in the top of the masonry over the support and anchored in accordance with **10.3.5.4**. In all cases, at least one quarter of the reinforcement required at mid-span shall be carried through to the support and similarly anchored.

10.3.8.3 *Columns*

The design of reinforced column shall meet the requirements given hereunder.

10.3.8.3.1 Vertical reinforcement shall not be less than 0.25 percent nor exceed 4 percent of the net area of column cross-section. The minimum number of bars shall be four.

10.3.8.3.2 *Lateral ties*

Lateral ties shall be provided in the column as per the following:

a) Longitudinal reinforcement shall be enclosed by lateral ties of at least 6 mm diameter. Vertical spacing of ties along the length of column shall be minimum of,
   1) 16 times diameter of longitudinal bar,
   2) 48 times diameter of lateral tie, and
   3) Least dimension of the column.
b) Arrangement of lateral ties is such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a lateral tie with an included angle of not more than 135˚.

10.3.9 *Members Subjected to Shear*

10.3.9.1 Reinforced masonry walls may be designed taking contribution of shear reinforcement.

10.3.9.2 Where contribution of shear reinforcement is considered in resisting shear force the minimum area of shear reinforcement in the direction of force shall be determined by the following:

$$A_{v,\min} = \frac{V_s}{F_s d}$$

where
V_s = Total applied shear force,
s = Spacing of the shear reinforcement,
d = Distance from extreme compression fiber to centroid of tension reinforcement, and
F_s = Permissible stress in steel reinforcement as defined in **10.3.2**.

10.3.9.3 The maximum spacing of shear reinforcement shall not be greater than 0.5*d* or 1.2 m, whichever is lesser.

10.3.9.4 In cantilever beams, maximum shear shall be used whereas for members subjected to uniformly distributed load it may be assumed that maximum shear load occurs at a distance of 0.5*d* from the face of support when the following conditions are met:

a) Support reaction causes compression in the end region of the member, and
b) No concentrated load between face of support and a distance of 0.5*d* from it.

10.3.10 *Reinforcement Detailing*

10.3.10.1 *General*

Reinforcement shall be located such that it acts compositely with the masonry and various ways in which it can be used in reinforced masonry are shown in Fig. 30.

10.3.10.2 *Protection of reinforcement*

Where steel reinforcing bars are embedded in filled cavity (or pockets) or special bond construction, the bars shall have the minimum clear cover of 10 mm in mortar or a minimum clear cover of 15 mm or bar diameter, whichever is more in cement concrete (grout) so as to achieve good bond and corrosion resistance.

For the reinforcement steel placed in mortar bed joint, the minimum depth of mortar cover from the reinforcing steel to the face of masonry shall be 15 mm. Also mortar cover above and below reinforcement placed in bed joints shall not be less than 2 mm.

Reinforcing steel shall be corrosion resistant or protected adequately against corrosion. Reinforcement shall be stainless steel or hot-dipped galvanized or epoxy coated steel reinforcement for protection against corrosion. As an alternative to solid stainless steel, normal steel reinforcing bar can be coated with at least 1mm thickness of austenitic stainless steel.

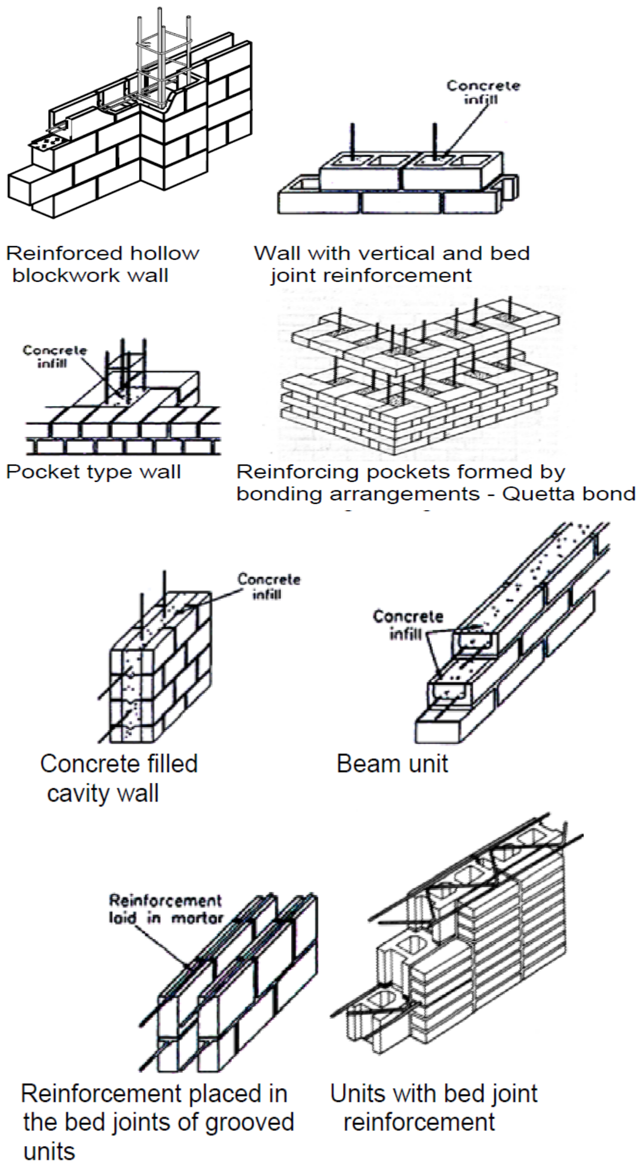


FIG. 30 REINFORCEMENT DETAILS

10.4 Structural Design

10.4.1 *Permissible Compressive Force*

Compressive force in reinforced masonry due to axial load shall not exceed that given by following equation:

$$P_o = (0.25 f_m A_n + 0.65 A_{st} F_s) k_s$$

where

A_n = Net area,

A_{st} = Area of steel,

F_s = Permissible steel tensile stress, and

k_s = Stress reduction factor as in Table 9 of IS 1905.

10.4.1.1 *Effective compressive width for locally concentrated reinforcement*

When the reinforcement in masonry is concentrated locally such that it cannot be treated as a flanged member (Fig. 31), the reinforced section shall be considered as having a width of not more than:

1) centre-to-centre bar spacing, and
2) 6 times the wall thickness.

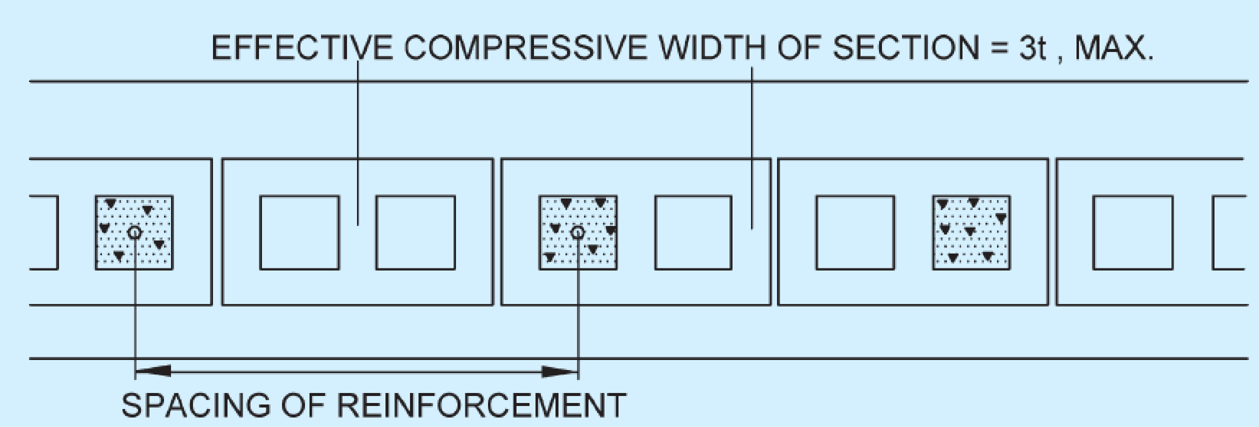


FIG. 31 EFFECTIVE COMPRESSIVE WIDTH

10.4.1.2 *Combined permissible axial and flexural compressive stress*

For reinforced members subjected to combined axial load and flexure, the compressive stress in masonry due to combined action of axial load and bending shall not exceed *1.25* F_a and compressive stress in masonry due to axial load only shall not exceed F_a.

10.4.1.3 *Permissible tensile stress*

Provisions of **5.4.2** of IS 1905 shall apply.

10.4.1.4 *Permissible shear stress*

For members in flexure, the *Permissible Shear Stress* shall be taken as:

$$F_v = \begin{cases} 0.083\sqrt{f_m} \leq 0.25MPa & \text{Walls without web reinforcement} \\ 0.250\sqrt{f_m} \leq 0.75MPa & \text{Walls with web reinforcement} \end{cases}$$

For walls in compression and bending, the *Permissible Shear Stress* for reinforced masonry walls shall be according to Table 12.

If there is tension in any part of a section of masonry, the area under tension shall be ignored while estimating the shear stress on the section.

Table 12 Permissible shear stress in reinforced masonry walls under compression and bending
(*Clause* 10.4.1.4)

Sl No.	Type of Wall	M/Vd	Permissible Shear Stress, F_v (MPa)	Maximum Allowable shear stress (MPa)
(1)	(2)	(3)	(4)	(5)
i)	Without Web Reinforcement	< 1.0	$\frac{1}{36}\left(4-\frac{M}{Vd}\right)\sqrt{f_m}$	$\left(0.4-0.2\frac{M}{Vd}\right)$
		> 1.0	$0.083\sqrt{f_m}$	0.2
ii)	With Web Reinforcement	< 1.0	$\frac{1}{24}\left(4-\frac{M}{Vd}\right)\sqrt{f_m}$	$\left(0.6-0.2\frac{M}{Vd}\right)$
		> 1.0	$0.125\sqrt{f_m}$	0.4

10.5 Seismic Design Requirements

10.5.1 The requirements of this section shall apply to the design and construction of reinforced masonry to improve its performance when subjected to earthquake loads. These provisions are in addition to the general requirements of CED 39 (22343).

10.5.2 *Different Performance Levels of Masonry Shear Walls*

Masonry buildings rely on masonry shear walls for the lateral load resistance and can be detailed for the following three levels of seismic performance, which can be appropriately chosen for a building considering its importance, location, and acceptable degree of damage. Table 13 summarizes the requirement for these structural walls and recommended *R* values for use with CED 39 (22343).

Table 13 Reinforcement and 'R' values for Reinforced Masonry Buildings with Different Wall Types and Earthquake Zones
(*Clause* 10.5.2)

Sl No.	Type of Wall	Description	Reinforcement	Earthquake Zone	R Value
(1)	(2)	(3)	(4)	(5)	(6)
i)	RMB 1	Reinforced Masonry Walls with minimum reinforcement	As per **10.5.2.1**	II and III	2.5
ii)	RMB 2	Reinforced Masonry Walls with design reinforcement	As per **10.5.2.2**	II and III IV and V	3.0

iii)	RMB 3	Reinforced Masonry Walls with special design reinforcement	As per **10.5.2.2** and **10.5.2.3sss**	II and III IV, V and VI	3.0

10.5.2.1 *Masonry Walls with Minimum Reinforcement (RMB 1)*

The design of masonry walls shall comply with requirements of unreinforced masonry wall IS 1905 and those given hereunder.

The requirements of this section apply to the design and construction of reinforced masonry to improve its performance when subjected to earthquake loads. These provisions are in addition to the general requirements of IS 1905.

a) Elastic Force Reduction Factor R shall be taken as 3 for *Reinforced Masonry Buildings*.

b) *Minimum Reinforcement Requirements* (Fig. 33)
The vertical reinforcement of at least 100 mm^2 in cross-sectional area shall be provided at a maximum spacing of 3 m on centre at the following critical sections:
1) Corners,
2) Within 400 mm of each side of openings,
3) Within 200 mm of the end of the walls.

c) Horizontal reinforcement shall consist of at least two bars of 6 mm spaced not more than 400 mm; or bond beam reinforcement shall be provided of at least 100 mm2 in cross-sectional area spaced not more than 3 m. Horizontal reinforcement shall be provided at the bottom and top of wall openings and shall extend at least 500 mm or 40 bar diameters past the openings; continuously at structurally connected roof and floor levels and within 400 mm of the top of the walls.

10.5.2.2 *Reinforced masonry wall with design reinforcement (RMB 2)*

The design of reinforced masonry walls shall comply with the requirements of reinforced masonry wall as outlined in this standard and shall comply with the requirements of **10.5.2.1 (a) to (c).**

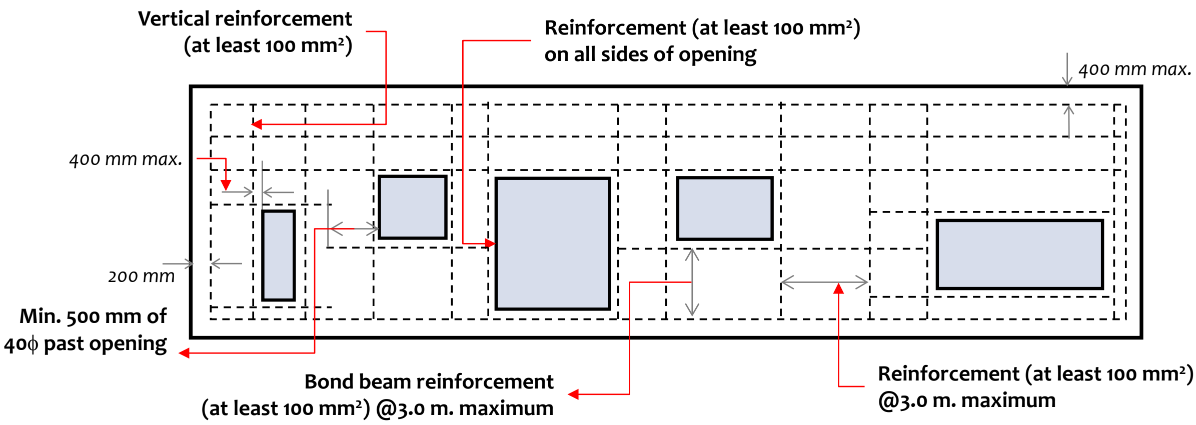


FIG. 33 REQUIREMENTS OF MINIMUM REINFORCEMENT IN MASONRY WALLS (RMB 1)

10.5.2.3 *Reinforced masonry walls with special design reinforcement (RMB 3)*

Design of special reinforced masonry wall shall comply with the requirement of reinforced masonry as outlined in **10.5.2.2** and the following:

a) The masonry shall be uniformly reinforced in both horizontal and vertical direction such that the sum of reinforcement area in both directions shall be at least 0.2 percent of the gross cross-sectional area of the wall and minimum reinforcement area in each direction shall be not less than 0.07 percent of the gross cross-sectional area of the wall.

b) Maximum spacing of horizontal and vertical reinforcement shall be lesser of,
   1) one-third length of the wall,
   2) one-third height of the wall, and
   3) 1.2 m.

c) Minimum cross-sectional area of reinforcement in vertical direction shall be one-third of the required shear reinforcement.

d) Shear reinforcement shall be anchored around vertical reinforcing bars with a 135˚ or 180˚ standard hook.

SECTION 3

ADDITIONAL CRITERIA FOR REINFORCED CONCRETE BUILDINGS

11 GENERAL SPECIFICATIONS

11.1 The design and construction of reinforced concrete (RC) 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.

11.2 Design Requirement

The general requirements for design and detailing of ductile RC buildings address the following broad aspects:

a) Stability,
b) Stiffness,
c) Strength,
d) Deformability, and
e) Ductility.

Thus, RC buildings shall be designed and detailed as per this standard to resist design earthquake hazard defined in CED 39 (22343).

11.2.1 RC buildings shall have planar frames oriented along the two principal plan directions of buildings. Irregularities listed in CED 39 (22343) 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 irregularities appear in a building as listed in **5.1** of CED 39 (22345) the guidance given therein for the respective irregularity shall be followed.

12 MATERIALS

The materials and members used as part of lateral force resisting systems in RC buildings shall conform to the following.

12.1 The minimum grade of structural concrete shall be M20. Further:

a) It shall be M25 for buildings that are:
    i) more than 15 m in height, or
    ii) located in Earthquake Zones III, IV, V and VI; and
b) It shall not be less than that required by IS 456 based on exposure conditions.

12.2 Steel reinforcement resisting earthquake-induced forces in RC frame members and in boundary elements of RC structural walls shall comply with **12.2.1** and **12.2.2**.

12.2.1 Steel reinforcements used in earthquake zones III, IV, V and VI shall comply with all of the following:

a) Elongation shall be at least 14.5 percent,
b) Ratio of ultimate stress to 0.2 percent proof stress shall lie in the range 1.15–1.25, and
c) Steel shall be only of strength grades with minimum 0.2 percent proof stress of 415 MPa, 500 MPa or 550 MPa, in addition to other requirements of IS 1786.

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

12.2.3 In the estimation of area of transverse reinforcement (including special confining reinforcement), the grade of transverse steel shall be taken as 415 MPa, even though the grade steel used therein is 500 MPa or 550 MPa.

13 DESIGN OF MEMBERS

Individual concrete members shall be designed as per the provisions of IS 456. Further, the additional provisions given in **13** to **18** shall be complied with.

13.1 Beams

Requirements of this section shall apply to beams resisting earthquake-induced effects, in which the factored axial compressive stress 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 **13.2**.

13.1.1 *Geometry*

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

13.1.1.2 Beams shall not have width less than 200 mm.

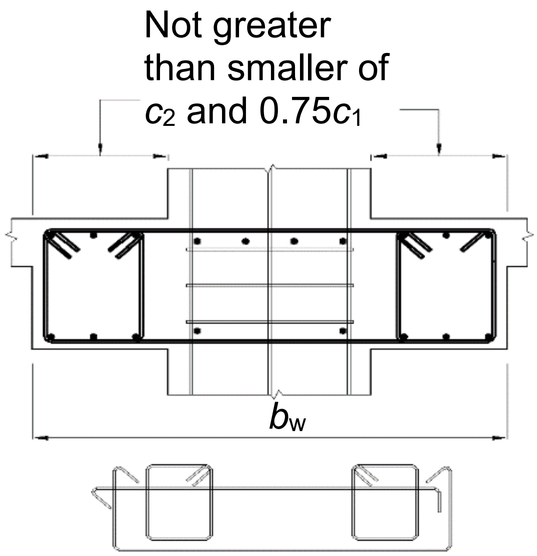
13.1.1.3 Beams meant to undergo flexural plastic hinges during strong earthquake shaking shall have slenderness ratio (L_c/D) in the range 12-14, where L_c is the *clear span* of the beam and D the *depth* of the beam.

13.1.1.4 Beams with small L_c/D ratio (like transfer girders and deep beams) shall be designed as per provisions of **12.2.3**.

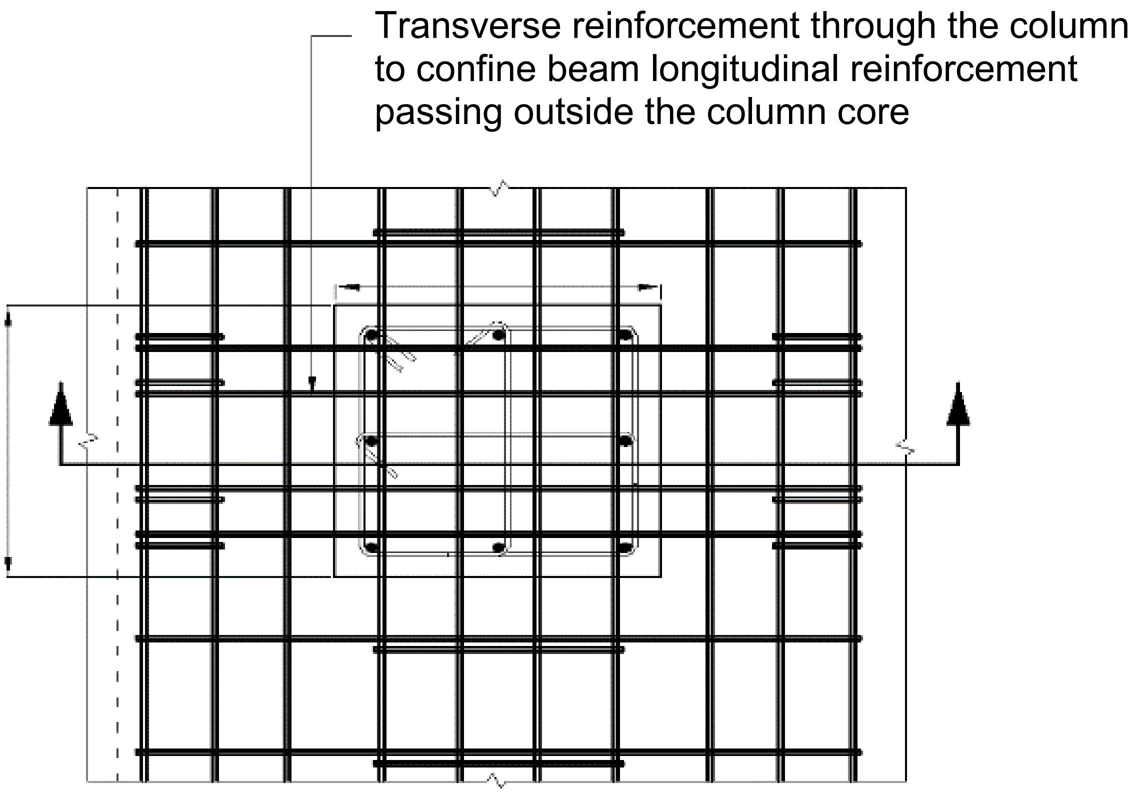
13.1.1.5 Width of beam b_w shall not exceed the width of supporting member c_2 plus distance on either side of supporting member equal to the smaller of:

a) Width c_2 of supporting member c_2; and
b) 0.75 times breadth of supporting member c_1 (*see* Fig. 34A and Fig. 34B).

Transverse reinforcement for the width of a beam that exceeds width of the column c_2 shall be provided throughout the beam span including within the beam column joint (*see* Fig. 34B).



Section A-A
34A



Plan
34B

FIG. 34 MAXIMUM EFFECTIVE WIDTH OF WIDE BEAM AND REQUIRED TRANSVERSE REINFORCEMENT

13.1.2 *Ductile Design*

13.1.2.1 *Design bending moment capacity*

The *Bending Moment Capacity* of a beam at the critical sections shall be determined considering Design Bending Moment Demands determined from linear structural analysis of the structure under the action of loads combined as per the load combinations corresponding to *Strength Design* as per *Limit State Method* of design for concrete structures specified in IS 456.

13.1.2.2 *Design shear force capacity*

The *Bending Moment Capacity* of a beam at the critical sections shall be determined consistent with the Principles of Capacity Design, as enumerated below:

a) The *Design Shear Force Capacity* of a beam at each end shall be at least larger of,

1) Design Shear Force Demands determined from linear structural analysis of the structure under the action of loads combined as per the load combinations corresponding to *Strength Design* as per Limit State Method of design for concrete structures specified in IS 456; and
2) Design Shear Force Demands considering factored gravity shear force demand and equilibrium shear force when plastic hinges are formed at both ends of the beam (*see* Fig. 35), given by:

i) For sway of the structure to the right:

$$V_{uA} = 1.2\left(V_A^{DL} + V_A^{LL}\right) - 1.4\left(\frac{M_{\mathrm{u}}^{\mathrm{As}} + M_{\mathrm{u}}^{\mathrm{Bh}}}{L_{\mathrm{AB}}}\right) \quad \text{and}$$

$$V_{uB} = 1.2\left(V_A^{DL} + V_A^{LL}\right) + 1.4\left(\frac{M_{\mathrm{u}}^{\mathrm{As}} + M_{\mathrm{u}}^{\mathrm{Bh}}}{L_{\mathrm{AB}}}\right).$$

ii) For sway of the structure to the left:

$$V_{uA} = 1.2\left(V_A^{DL} + V_A^{LL}\right) + 1.4\left(\frac{M_{\mathrm{u}}^{\mathrm{Ah}} + M_{\mathrm{u}}^{\mathrm{Bs}}}{L_{\mathrm{AB}}}\right) \quad \text{and}$$

$$V_{uB} = 1.2\left(V_A^{DL} + V_A^{LL}\right) - 1.4\left(\frac{M_{\mathrm{u}}^{\mathrm{Ah}} + M_{\mathrm{u}}^{\mathrm{Bs}}}{L_{\mathrm{AB}}}\right),$$

where $M_{\mathrm{u}}^{\mathrm{As}}, M_{\mathrm{u}}^{\mathrm{Ah}}$, $M_{\mathrm{u}}^{\mathrm{Bs}}$ *and* $M_{\mathrm{u}}^{\mathrm{Bh}}$ are sagging and hogging moments of resistance of the beam section at ends *A* and *B*, respectively. These shall be estimated as per the *Limit State Method* of design given in IS 456. L_{AB} is clear span of the beam. V_A^{DL} and V_B^{DL} are factored shear forces at ends *A* and *B*, respectively, due to dead loads acting on the span, and V_A^{LL} and V_B^{LL} due to imposed loads.

(b) In the estimation of *Design Shear Force Capacity* of a RC beam, contribution of the following alone shall be considered over the plastic hinge length of 2d from the face of the support or of 2d from the exterior faces of the intermediate beam (if any) meeting the main beam (Fig 36):

1) Closed-loop (with 135° hook ends) transverse steel, and
2) Straight transverse steel anchored to the longitudinal steel bars.

The contributions of bent-up bars, inclined links and uncracked concrete in the cross-section shall not be considered.

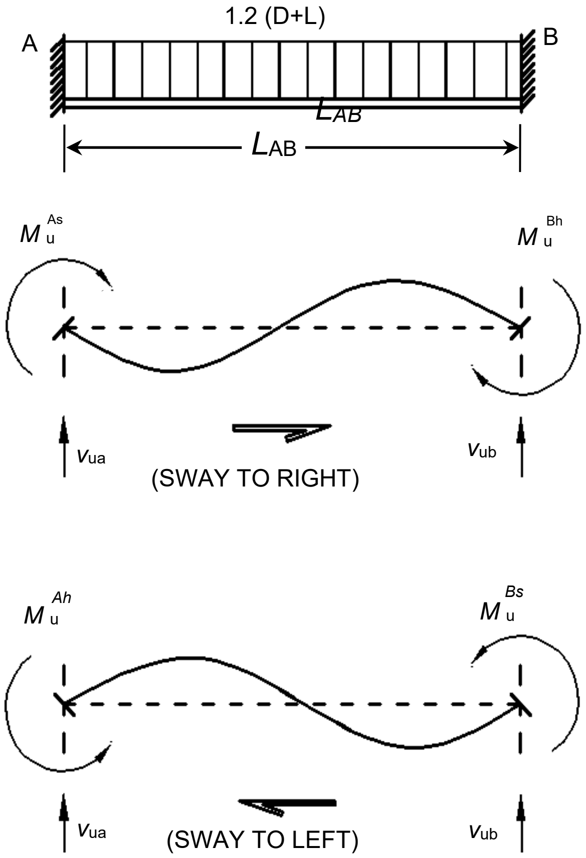


FIG. 35 FREE BODY DIAGRAMS OF A BEAM SHOWING EQUILIBRIUM SHEAR FORCES INDUCED WHEN PLASTIC HINGES ARE FORMED AT THEIR ENDS, WHEN THE STRUCTURE SWAYS TO THE RIGHT AND TO THE LEFT

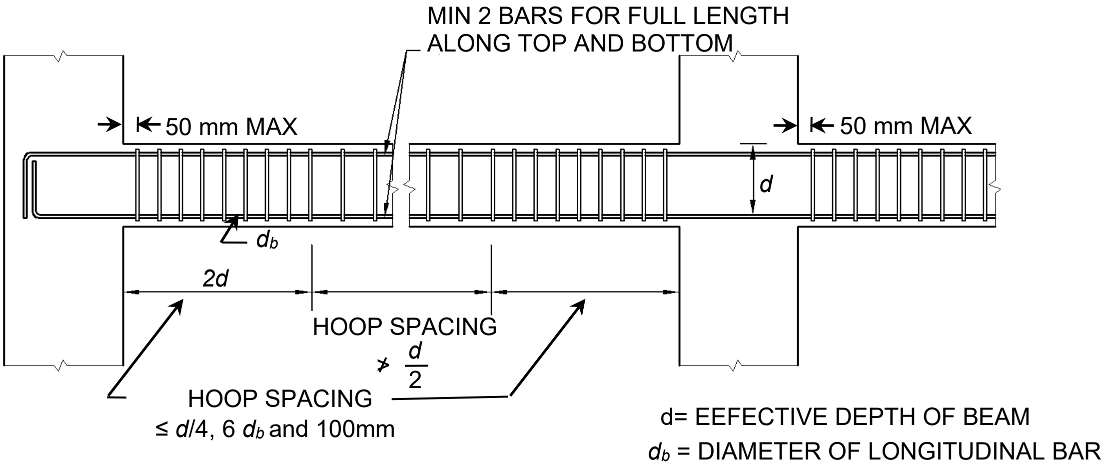


FIG. 36 DETAILS OF TRANSVERSE REINFORCEMENT IN BEAMS

13.1.3 *Ductile Detailing*

13.1.3.1 *Longitudinal reinforcement*

The longitudinal reinforcement in beams shall be as below:

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:

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

13.1.3.2 Maximum longitudinal steel ratio ρ_{max} provided on any face at any section is 0.025.

13.1.3.3 Beam cross-sections shall be provided with longitudinal steel reinforcement:

a) at any section on top or bottom face equal to at least 25 percent 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;
b) at any section in compression equal to at least 50 percent of the longitudinal reinforcement provided in tension.
c) at bottom face of a beam framing into a column (at the face of the column) equal to at least:

   i) 75 percent of the steel on its top face at the same section, in beams that are part of the lateral load resisting system, and
   ii) 50 percent of the steel on its top face at the same section, in beams that are not part of the lateral load resisting system.

   At exterior joints, the anchorage length calculation shall consider the bottom r/f to be tension reinforcement.

13.1.3.4 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. 37).

13.1.3.5 *Splicing of longitudinal bars*

a) *Lap splices*

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

1) spacing of these links shall not exceed 150 mm (*see* Fig. 38).
2) lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.
3) lap splices shall not be provided within:
   i) A joint;
   ii) 2d from face of the column; and
   ii) A quarter length of the beam adjoining the location where flexural yielding may occur under earthquake effects.

4) not more than 50 percent of area of steel bars on either top or bottom face shall be spliced at any one section.

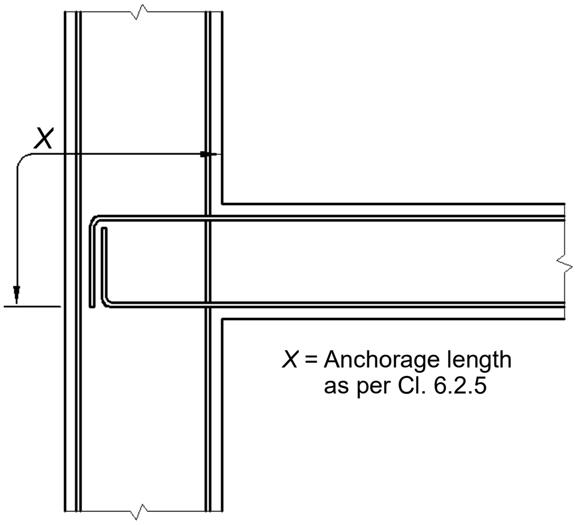


FIG. 37 ANCHORAGE OF LONGITUDINAL BEAM BARS AT EXTERIOR BEAM-COLUMN JOINT

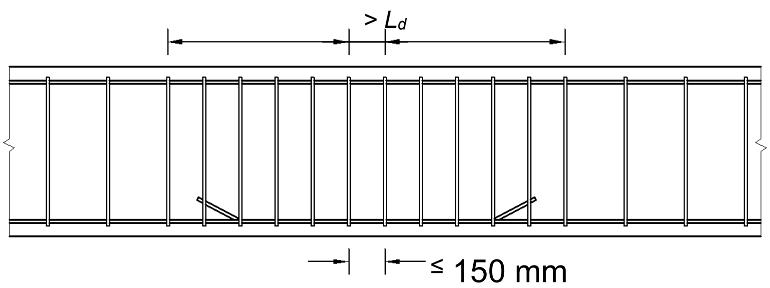


FIG. 38 LAP LENGTH AT LOCATION OF SPLICING OF LONGITUDINAL BARS IN BEAM

b) *Mechanical Couplers*

Mechanical couplers (conforming to IS 16172) shall be used when longitudinal steel bars must be continued for beam spans larger than their manufacture lengths. Further,

1) only those mechanical couplers which are conforming to IS 16172 and capable of developing the specified tensile strength in spliced bar shall be permitted. At any sections, not more than 50 percent of bars shall be coupled, and the next location of coupler shall be at least 300 mm away; and
2) the spacing between adjacent longitudinal bars shall be based also on the outer size of the coupler to allow easy flow of concrete.

c) *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 percent of area of steel bars shall be spliced at any one section. Welding of links, ties, inserts or other similar elements to vertical reinforcement bars required as per design is not permitted, in any earthquake zone.

13.1.3.6 *Transverse reinforcement*

13.1.3.6.1 Only vertical links shall be used in beams (*see* Fig. 39A); inclined links shall not be used. And,

a) in normal practice, a link is made of a single bent bar. But, it may be made of two bars also, namely a U-link with a 135^{o} hook with an extension of 8 times diameter (but not less than 75 mm) at each end, embedded in the core concrete, and a cross-tie (*see* Fig. 39B).

b) the hooks of the links 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.

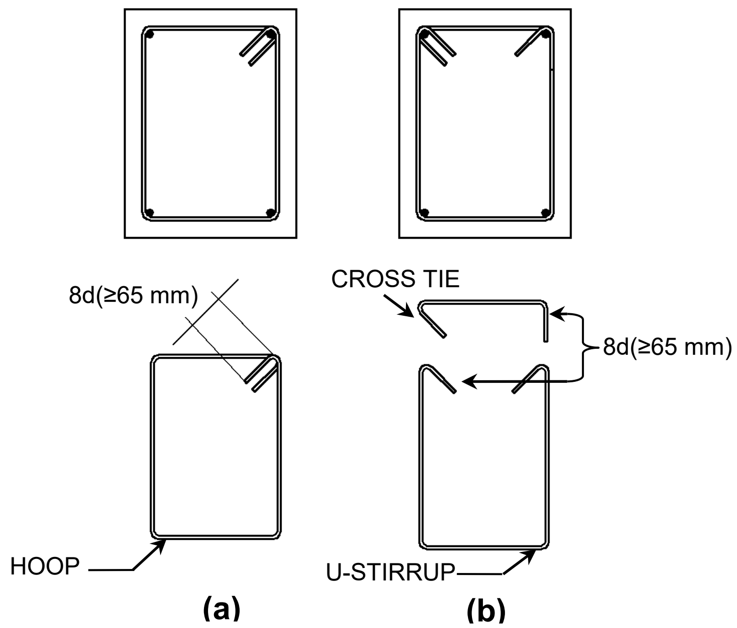


FIG. 39 DETAILS OF HOOKS OF TRANSVERSE REINFORCEMENT IN BEAMS

13.3.6.2 The minimum diameter of a link shall be 8 mm.

13.3.6.3 *Close spacing of links*

a) Spacing of links over a length of 2*d* at either end of a beam (*see* Fig. 39) shall not exceed,
   i) *d*/4;
   ii) 6 times the diameter of the smallest longitudinal bar; and
   iii) 100 mm.

b) The first link shall be at a distance not exceeding 50 mm from the joint face.

c) Closely spaced links shall be provided over a length equal to 2*d* on either side of a section where flexural yielding may occur under earthquake effects. Over the remaining length of the beam, vertical links shall be provided at a spacing not exceeding *d*/2.

d) Constructions joint shall not be in the regions of the beam having closely spaced transverse reinforcements.

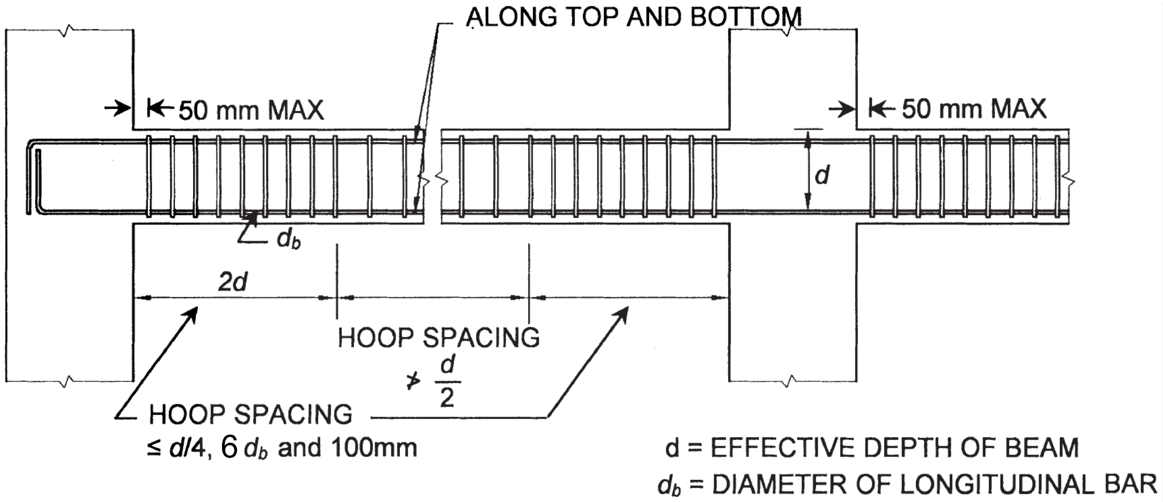


FIG. 39 DETAILS OF TRANSVERSE REINFORCEMENT IN BEAMS

13.2 Columns

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 0.08 f_{ck}.

13.2.1 *Geometry*

For columns of shapes other than rectangular and circular (such as 'T', 'X' and '+' shaped), which form part of the lateral load resisting system, appropriate design and detailing shall be carried out using specialist literature.

13.2.1.1 The minimum dimension of a column shall not be less than:

a) 20 d_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. 40).

13.2.1.2 The cross-section aspect ratio (that is, ratio of smaller dimension to larger dimension of the cross-section of a column or inclined member) shall be more 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 **13.5**.

13.2.2 *Ductile Design*

13.2.2.1 *Design axial force – bending moment capacity*

The Axial Force Capacity and Bending Moment Capacity of a beam at the critical sections shall be estimated considering P-M interaction. The design longitudinal steel shall be the largest obtained considering Design Axial Force and Design Bending Moment Demands determined from linear structural analysis of the structure under the action of loads combined as per the load combinations corresponding to Strength Design as per Limit State Method of design for concrete structures specified in IS 456.

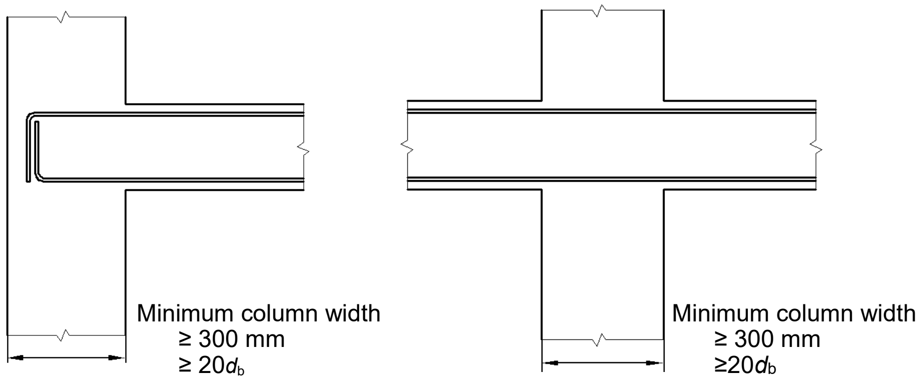


FIG. 40 MINIMUM SIZE OF RC COLUMNS BASED ON DIAMETER OF LARGEST LONGITUDINAL REINFORCEMENT BAR IN BEAMS FRAMING INTO IT

13.2.2.2 *Design shear force capacity*

The shear force capacity of a column at the critical sections shall be determined consistent with the principles of capacity design, as enumerated below:

13.2.2.2.1 The *Design Shear Force Capacity* of a column shall be at least larger of,

a) Design Shear Force Demands determined from linear structural analysis of the structure under the action of loads combined as per the load combinations corresponding to *Strength Design* as per Limit State Method of design for concrete structures specified in IS 456; and
b) Design Shear Force Demands considering factored gravity shear force demand and equilibrium shear force when plastic hinges are formed at both ends of the beam (*see* Fig. 35), given by:
   i) Sway of the structure to the right:
   $$V_u = 1.4\frac{\left(M_u^{As} + M_u^{Bh}\right)}{h_{st}}$$, and
   i) Sway of the structure to the 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 sagging and hogging moments of resistance of the beam sections *A* and *B* meeting at the beam-column joint and swaying laterally (*see* Fig. 41). These shall be estimated as per the *Limit State Method* of design given in IS 456. h_{st} is clear storey height of the column. V_A^{DL} and V_B^{DL} are factored shear forces at ends *A* and *B*, respectively, due to dead loads acting on the span, and V_A^{LL} and V_B^{LL} due to imposed loads.

13.2.2.2.2 In the estimation of Design Shear Force Capacity of a RC column, contribution of the following alone shall be considered over the plastic hinge length of 2d from the face of the support or of 2d from the exterior faces of the intermediate beam (if any) meeting the main beam:
   a) Closed-loop (with 135° hook ends) transverse steel,
   b) Straight transverse anchored to the longitudinal steel bars, and

c) Uncracked concrete in the cross-section.

The contributions of bent-up bars and inclined links shall not be considered.

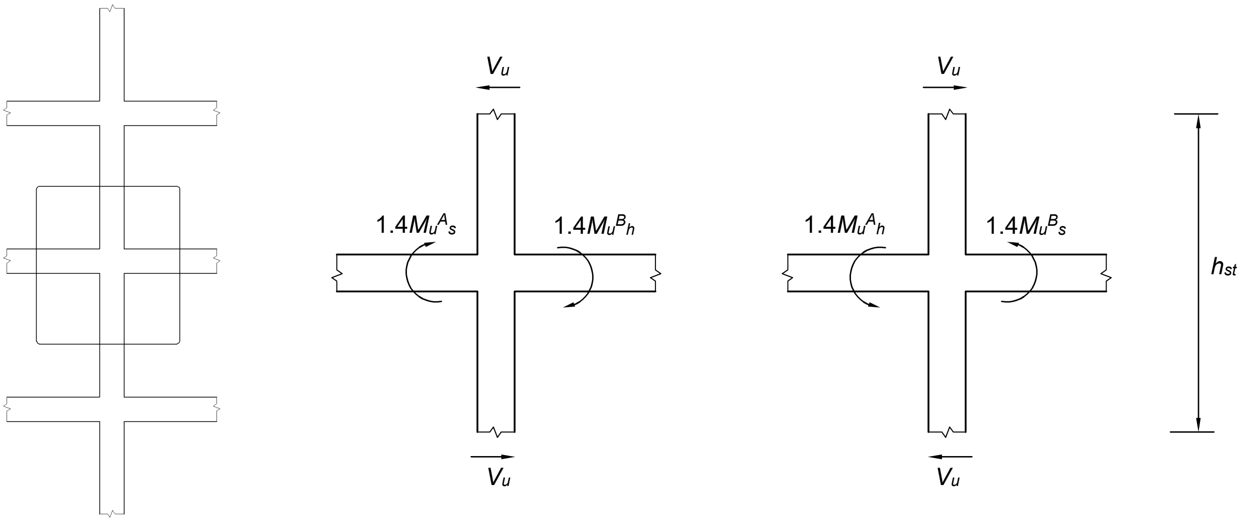


FIG. 41 EQUILIBRIUM DESIGN SHEAR FORCE DEMAND ON COLUMN WHEN PLASTIC HINGES ARE FORMED AT BEAM ENDS

13.2.2.3 *Limiting demands*

The provisions hereunder shall be complied with when estimating the stress-resultant demands on columns,

a) *Axial Stress Ratio*

The *Design Axial Force Demand* specified for *Strength Design* shall be limited to both of the following:

i) Considering Gravity Load combinations alone: to the Axial Load corresponding to the Balanced Point as specified in IS 456 for Limit State Design, and

ii) Considering load combinations involving earthquake loads: to $0.4 f_{ck}$.

b) *Design Shear Strength of Concrete under Tension*

Concrete shall be considered to carry no shear when the member is in tension.

13.2.2.4 *Relative member strength ratio*

a) *Column-to-Beam Strength Ratio*

At each beam-column joint of a moment-resisting frame identified to be part of the lateral load resisting system, the sum of nominal design *bending moment capacities* of columns meeting at that joint (with nominal *bending moment capacity* calculated corresponding to factored axial load, for shaking in the direction of the lateral force under consideration) along each principal plane shall be at least 1.4 times the sum of nominal design *bending moment capacities* of beams meeting at that joint for shaking in the direction (*see* Fig. 42), given by:

$$\left(M_u^{c,bot,top,X-} + M_u^{c,top,bot,X-}\right) > 1.4\left(M_u^{b,L,h,X-} + M_u^{b,R,s,X-}\right),$$
$$\left(M_u^{c,bot,top,X+} + M_u^{c,top,bot,X+}\right) > 1.4\left(M_u^{b,L,s,X+} + M_u^{b,R,h,X+}\right),$$

$$\left(M_u^{c,bot,top,Y-} + M_u^{c,top,bot,Y-}\right) > 1.4\left(M_u^{b,L,h,Y-} + M_u^{b,R,s,Y-}\right), \text{ and}$$
$$\left(M_u^{c,bot,top,Y+} + M_u^{c,top,bot,Y+}\right) > 1.4\left(M_u^{b,L,s,Y+} + M_u^{b,R,h,Y+}\right),$$

where,

$M_u^{c,bot,top,X-}$ = Nominal design *bending moment capacity* at top of bottom column meeting at that joint, for earthquake shaking along -X direction,

$M_u^{c,top,bot,X-}$ = Nominal design *bending moment capacity* at bottom of top column meeting at that joint, for earthquake shaking along -X direction,

$M_u^{b,L,s,X-}$ = Nominal design *bending moment capacity* in sagging at the face of the joint of left beam meeting at that joint, for earthquake shaking along -X direction,

$M_u^{b,R,h,X-}$ = Nominal design *bending moment capacity* in hogging at the face of the joint of right beam meeting at that joint, for earthquake shaking along -X direction,

$M_u^{c,bot,top,X+}$ = Nominal design *bending moment capacity* at top of bottom column meeting at that joint, for earthquake shaking along +X direction,

$M_u^{c,top,bot,X+}$ = Nominal design *bending moment capacity* at bottom of top column meeting at that joint, for earthquake shaking along +X direction,

$M_u^{b,L,h,X+}$ = Nominal design *bending moment capacity* in sagging at the face of the joint of left beam meeting at that joint, for earthquake shaking along +X direction,

$M_u^{b,R,s,X+}$ = Nominal design *bending moment capacity* in hogging at the face of the joint of right beam meeting at that joint, for earthquake shaking along +X direction,

$M_u^{c,bot,top,Y-}$ = Nominal design *bending moment capacity* at top of bottom column meeting at that joint, for earthquake shaking along –Y direction,

$M_u^{c,top,bot,Y-}$ = Nominal design *bending moment capacity* at bottom of top column meeting at that joint, for earthquake shaking along –Y direction,

$M_u^{b,L,s,Y-}$ = Nominal design *bending moment capacity* in sagging at the face of the joint of left beam meeting at that joint, for earthquake shaking along –Y direction,

$M_u^{b,R,h,Y-}$ = Nominal design *bending moment capacity* in hogging at the face of the joint of right beam meeting at that joint, for earthquake shaking along –Y direction,

$M_u^{c,bot,top,Y+}$ = Nominal design *bending moment capacity* at top of bottom column meeting at that joint, for earthquake shaking along +Y direction,

$M_u^{c,top,bot,Y+}$ = Nominal design *bending moment capacity* at bottom of top column meeting at that joint, for earthquake shaking along +Y direction,

$M_u^{b,L,s,Y+}$ = Nominal design *bending moment capacity* in sagging at the face of the joint of left beam meeting at that joint, for earthquake shaking along +Y direction, and

$M_u^{b,R,h,Y+}$ = Nominal design *bending moment capacity* in hogging at the face of the joint of right beam meeting at that joint, for earthquake shaking along +Y direction.

If a beam-column joint does not conform to above, the columns and beams meeting at the joint shall be considered as gravity columns, and not a part of the lateral load resisting system.

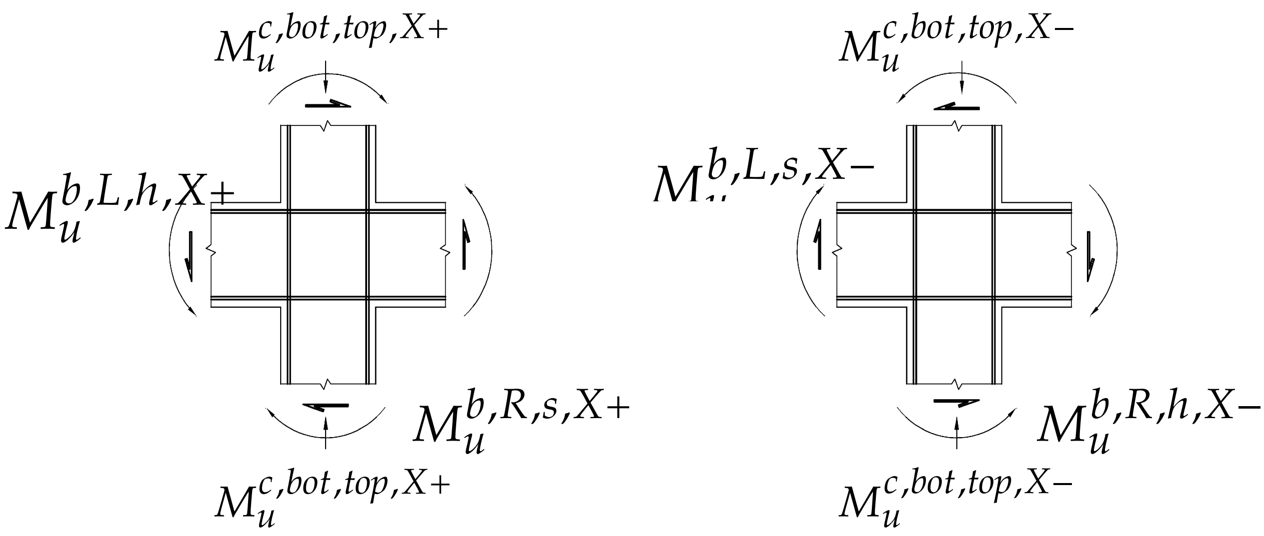
b) The nominal design moments of resistance of:
   i) a beam, and
   ii) a column corresponding to the design axial force induced in the column under the combination of loads involving earthquake shaking effects, based on the design *P-M* interaction diagram,

   shall be estimated based on the principles of mechanics and the limiting strain states of the limit state design method enunciated in IS 456.

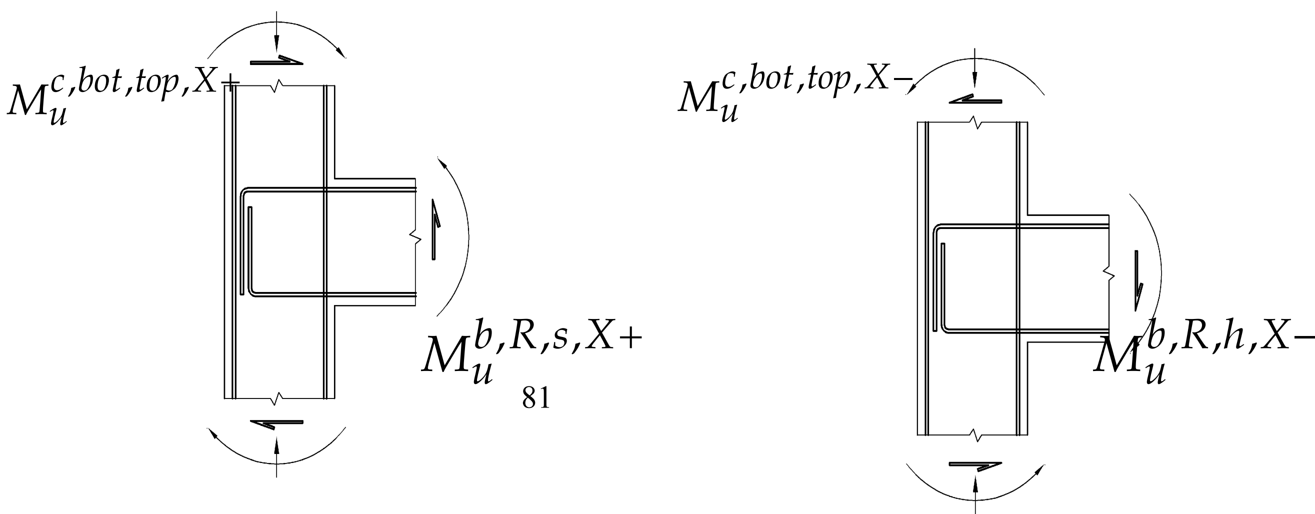
c) This check shall be performed at each joint for both positive and negative directions of shaking in the plane under consideration. Further, in this check, design moments of resistance in beam(s) meeting at a joint shall be considered in the same direction, and similarly the design moments of resistance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

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

e) The provisions of **13.2.2.4(a)** are not applicable for flat slab structure.



42A AN INTERIOR JOINT



$M_u^{c,bot,top,X+}$ $M_u^{c,bot,top,X-}$

42B AN EXTERIOR JOINT

FIG. 42 STRONG COLUMN – WEAK BEAM REQUIREMENT DEPICTED FOR SHAKING ALONG X-DIRECTION

13.2.3 *Ductile Detailing*

13.2.3.1 *Longitudinal reinforcement*

Single bars alone shall be used in earthquake resistant design of buildings; bundled bars shall not be used.

a) Circular columns shall have minimum of 6 bars.

b) *Splicing of Longitudinal Bars*

1) *Lap Splices*

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

i) the spacing of these links shall not exceed 100 mm.
ii) the lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.
iii) lap splices shall be provided only in the central half of clear column height, and not
    1) within a joint, or
    2) within 2d from face of the beam.
iv) not more than 50 percent of area of steel bars shall be spliced at any one section.
v) lap splices shall not be used for bars of diameter larger than 32 mm for which mechanical splicing shall be adopted.

2) *Mechanical Couplers*

Mechanical couplers (conforming to IS 16172) shall be used. Further, only those mechanical splices capable of developing the specified tensile strength of spliced bar shall be permitted. At any section, not more than 50 percent of bars shall be coupled. The next section of couplers shall be at least 300 mm away.

3) *Welded Splices*

In any earthquake zone,
i) Longitudinal bars shall not be welded; and
ii) Transverse reinforcements (hoops, links, and ties) shall not be welded.

13.2.3.2 A column that extends more than 100 mm beyond the confined core owing to architectural requirement (*see* Fig. 43) shall be have at least the minimum longitudinal and transverse reinforcements specified in:

a) this standard, when contribution of this area is considered when estimating strength of columns; and
b) IS 456, when contribution of this area is not considered when estimating strength of columns.

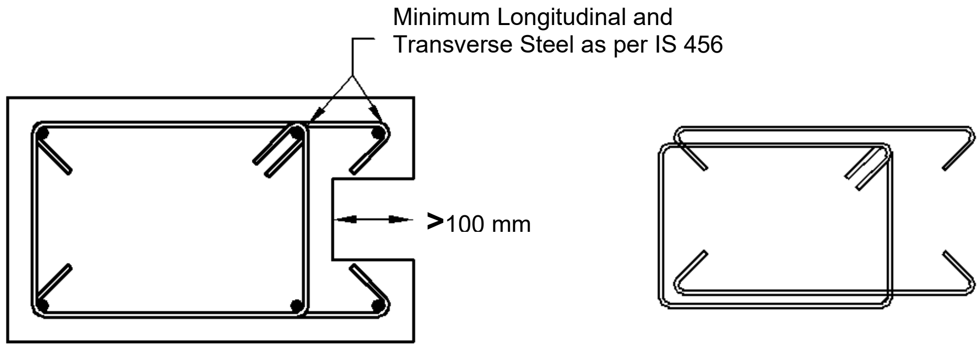


FIG. 43 REINFORCEMENT REQUIREMENT IN COLUMNS WITH PROJECTION MORE THAN 100 MM BEYOND CORE

13.2.3.3 *Transverse reinforcement*

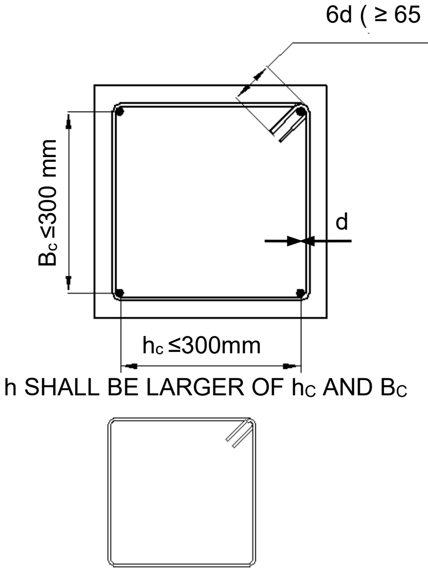
The transverse reinforcement shall comply with the provisions given hereunder,

13.2.3.3.1 Transverse reinforcement shall consist of closed hoops, which:

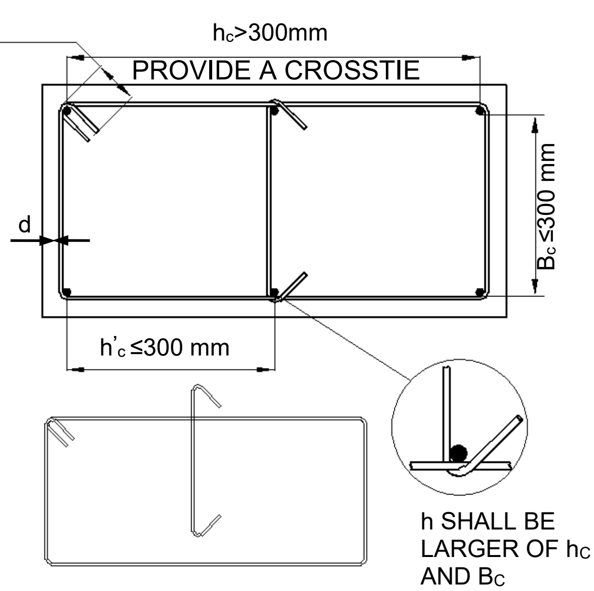
a) Have 135° hook ends with an extension of 10 times its diameter at each end, which are embedded in the confined core of the column (*see* Fig. 44A);
b) Envelope the outside perimeter of all longitudinal bars provided in the cross-section; and
c) Shall be circular in geometry in columns of circular cross-sections, and rectangular in geometry in rectangular columns.

13.2.3.3.2 When rectangular hoops are used:

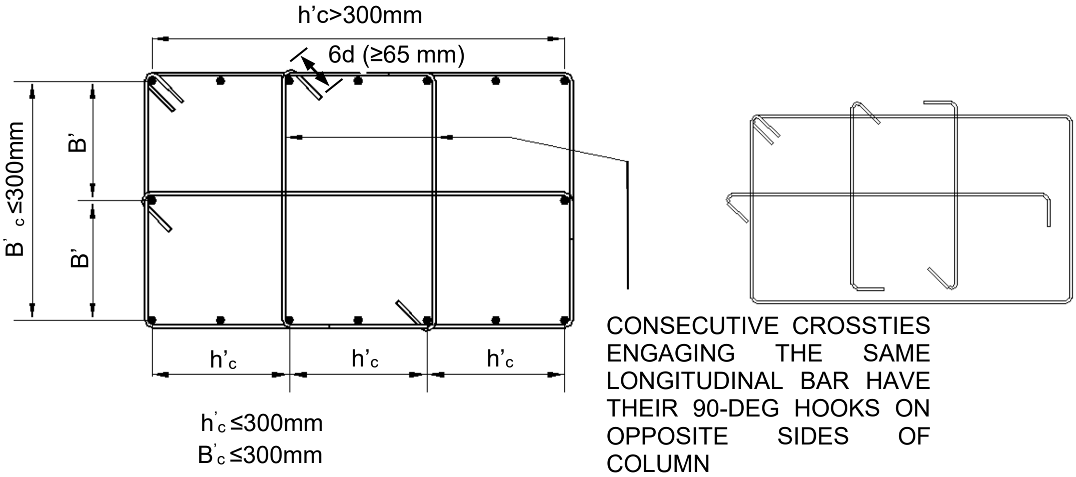
a) The minimum diameter permitted of transverse reinforcement bars shall be:
    1) 8 mm, when diameter of longitudinal bar is less than or equal to 32 mm, and
    2) 10 mm, when diameter of longitudinal bar is more than 32 mm;
b) The maximum center to center spacing of parallel legs of hoops shall be 300 mm;
c) Cross-ties shall be provided, if the length of any side of the hoop exceeds 300 mm (*see* Fig. 44B). These cross-ties shall be placed perpendicular to such hoops. And, the hook ends of the 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. Cross-ties shall have the same diameter as the hoops;
d) The maximum spacing of hoops shall be half the least lateral dimension of the column, except where special confining reinforcement is provided as per **13.2.3.3**;
e) Construction joints shall not be provided in regions of columns with closely spaced transverse reinforcement.



44A



44B



44C

FIG. 44 PLAN DETAILS OF TRANSVERSE REINFORCEMENT IN COLUMNS

13.2.3.3.3 The minimum transverse reinforcement provided shall comply with the following:

a) This special confining reinforcement (*see* Fig. 45) shall
   1) 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.
   2) 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; and
      iii) 100 mm.

3) have area A_{sh} of cross section of the bar forming links or spiral of at least:

i) in circular links or spirals:

$$A_{sh} = Max\left[0.09 s_v D_k \frac{f_{ck}}{f_y}\left(\frac{A_g}{A_k} - 1\right);\ 0.024 s_v D_k \frac{f_{ck}}{f_y}\right]$$

where

s_v = Pitch of spiral or spacing of links,

D_k = Diameter of core of circular column measured to outside of spiral/link,

f_{ck} = Characteristic compressive strength of concrete cube,

f_y = 0.2 percent proof strength of transverse steel reinforcement bars,

A_g = Gross area of column cross-section, and

A_k = Area of concrete core of column = $\frac{\pi}{4} D_{\mathrm{k}}^2$

ii) in rectangular links:

$$A_{sh} = Max\left[0.18 s_v h \frac{f_{ck}}{f_y}\left(\frac{A_g}{A_k} - 1\right);\ 0.05 s_v h \frac{f_{ck}}{f_y}\right]$$

where

h = Longer dimension of rectangular link measured to its outer face, which does not exceed 300 mm (*see* Fig. 44B), and

A_k = Area of confined concrete core in rectangular link measured to its outer dimensions.

h of the link could be reduced by introducing crossties (*see* Fig. 44C). In such cases, A_k shall be measured as overall core area, regardless of link arrangement. Hooks of cross-ties shall engage peripheral longitudinal bars.

b) 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. 46).

c) 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.

d) 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.

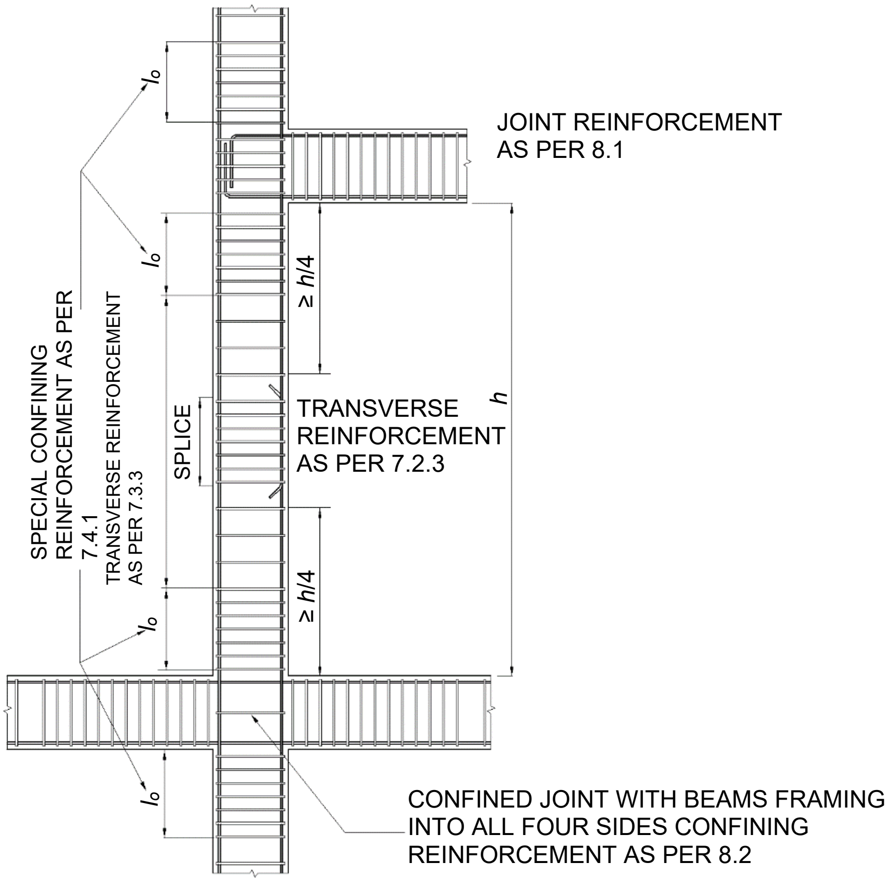


FIG. 45 COLUMN AND JOINT DETAILING

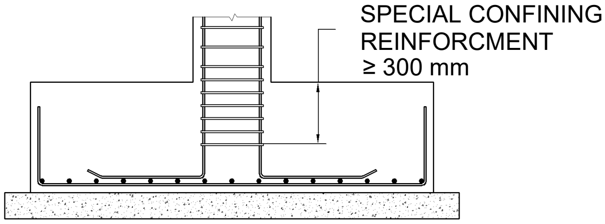


FIG. 46 PROVISION OF SPECIAL CONFINING REINFORCEMENT IN FOOTING

13.2.4 *Gravity Columns*

Gravity columns in buildings shall be designed according to **13.2.4.1** and **13.2.4.2** for bending moments induced when subjected to 'R' times the design lateral displacement under the factored equivalent static design seismic loads given by CED 39 (223434).

13.2.4.1 The provisions in **13.2.4.1** and **13.2.4.1** 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.

a) Gravity columns shall satisfy **13.2.3.1(b)**, **13.2.3.3.1** and **13.2.3.3.2.** But, spacing of links along the full column height shall not exceed 6 times diameter of smallest longitudinal bar or 150 mm.

b) Gravity columns with factored gravity axial stress exceeding $0.4f_{ck}$ shall satisfy (a) above and shall have transverse reinforcement at least one half of special confining reinforcement required by **13.2.3.3.3(a).**

13.2.4.2 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, the following shall be satisfied:

a) Lap and Mechanical Splices shall satisfy **13.2.3.1(b) (1)** and **13.2.3.1(b) (2).**

b) Gravity columns shall satisfy **13.2.3.3**.

13.3 Structural Walls

13.3.1 *Geometry*

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

13.3.1.2 The minimum thickness of special structural walls shall not be less than,

a) 200 mm for buildings with beams not framing into the wall along the direction perpendicular to the minor axis;
b) $20d_b$ thick for buildings with beams framing into the wall along the direction perpendicular to the minor axis at a location where there is a boundary element;
b) 300 mm for buildings with coupled structural walls in any earthquake zone, and for buildings with beams framing into the wall along the direction perpendicular to the minor axis.

The minimum thickness provided must conform to the fire resistance requirements based on occupancy as laid down in IS 456.

13.3.1.3 Only if the bending moment diagram of a vertical member is in single curvature along the full height of the building (even though with some jumps at levels where the beams and slabs frame into the wall), it should be treated as a structural wall. On the other hand, if it is in double curvature along the whole or part of the height of the building, it shall be designed as a column.

The above check on bending moment diagram in vertical members should be made owing to earthquake shaking along both its long and short dimensions in plan.

13.3.1.4 Special structural walls shall be classified based on their overall height H_w to Length L_w ratio, as:

a) Squat walls : $H_w / L_w < 1.0$,
b) Intermediate walls : $1.0 \le H_w / L_w \le 2.0$, and
c) Slender walls : $2.0 < H_w / L_w$.

13.3.1.5 In the design of flanged structural 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.

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

13.3.2 *Ductile Design*

13.3.2.1 *Earthquake demand*

13.3.2.1.1 *Axial force and bending moment demands*

a) These shall be estimated considering all load combinations involving earthquake loads specified for *Strength Design* as per CED 39(22343).
b) The Design Axial Force Demand considering all load combinations involving earthquake loads specified for *Strength Design* shall be limited to the Axial Load corresponding to the *Balanced Point* as specified in IS 456 for *Limit State Design*.

13.3.2.1.2 *Shear force demand*

The design shear force demand on the Structural Wall shall be the larger of the following:

a) Those estimated considering all load combinations involving earthquake loads specified for *Strength Design* as per IS 1893 (1).
b) Those estimated as 1.4 times the design moment capacity estimated as per **13.3.2.2**.

13.3.2.2 *Axial force and bending moment capacities*

a) *Axial Stress Ratio*

The Design Axial Force Demand specified for Strength Design shall be limited to:

1) the Axial Load corresponding to the Balanced Point as specified in IS 456 for Limit State Design under load combinations not involving earthquake loads, and
2) $0.4 f_{ck}$ under load combinations involving earthquake loads.

b) *Design Moment Capacity*

The Design Moment Capacity M_L of the structural wall section subjected to combined bending moment and compressive axial load shall be estimated as specified in IS 456 for Limit State Design, using the principles of mechanics involving equilibrium equations, strain compatibility conditions and constitutive laws.

The Design Moment Capacity M_u of a slender rectangular structural wall section with uniformly spaced vertical reinforcement may be estimated as:

1) $\left(x_u / L_w\right) < \left(x_u^* / L_w\right)$

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

where

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

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

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

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

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

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

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

E_s = Elastic modulus of steel, and

P_u = Factored compressive axial force on wall.

2) $\left(x_u^* / L_w\right) < \left(x_u / L_w\right) < 1.0$

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

where

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

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

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

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

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

where

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

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

The above expressions are not applicable for structural walls with boundary elements.

13.3.2.3 *Shear force capacity*

The Shear Force Capacity V_L of a section of structural wall subjected to combined bending moment and compressive axial load shall be estimated as specified hereunder.

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

$$\tau_v = \frac{V_u}{t_w L_{we}},$$

where

V_u = Factored shear force arising from load combinations involving earthquake load case as mentioned in **13.3.2.1**,

t_w = Thickness of the web, and

L_{we} = Effective depth of wall section (along the length of wall), which may be taken as $0.8L_w$ for rectangular sections.

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

c) When nominal shear stress demand τ_v on a structural wall is

i) more than maximum design shear strength $\tau_{c,max}$ of concrete (as per IS 456), the wall section shall be re-designed;

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

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

which shall not be less than the minimum area of horizontal steel as per 13.3.1.5; and

iii) less than design shear strength τ_c of concrete, horizontal shear reinforcement shall be the minimum area of horizontal steel as per 13.3.1.5.

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

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

13.3.3 *Design of Special Elements and Features*

13.3.3.1 *Boundary elements*

13.3.3.1.1 Boundary elements shall have geometry as specified hereunder:

a) The width of the boundary element shall be at least:

i) the same thickness as that of the wall web, if the boundary element is concealed and no beam frames into the boundary element in the direction perpendicular to the length of the wall;

ii) $20d_b$, with or without boundary elements framing into them.

b) The length of the boundary shall be larger of the following;

i) Twice the thickness of the wall web,

ii) 0.15 times the length of the wall, and

iii) 500 mm.

13.3.3.1.2 Boundary elements shall be provided along the vertical boundaries of walls, when the extreme fiber compressive stress in the wall exceeds $0.20f_{ck}$ 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.

13.3.3.1.3 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.

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

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

13.3.3.1.6 Boundary elements, where required as per **13.3.3.1.2**, shall be provided with special confining reinforcement throughout their height, given by

$$A_{sh} = 0.05 s_v h \left(\frac{f_{ck}}{f_y} \right)$$

and have a spacing not more than,

a) 1/3 of minimum member dimension of the boundary element;

b) 6 times diameter of the smallest longitudinal reinforcement bars; and

c) 100 mm but may be relaxed to 150 mm if maximum distance between cross-ties/parallel legs of links or ties is limited to 200 mm, but need not be less than 100 mm.

13.3.3.1.7 Boundary elements need not be provided, if the entire wall section is provided with special confining reinforcement, as per **13.2.3.3.3(a).**

13.3.3.1.8 When a beam frames into the minor axis of a wall, then boundary elements shall be provided at such locations along the full height of the wall. The design of such a boundary element shall meet the requirement of:

a) Minimum column dimension perpendicular to the minor axis of the walls given in **13.2.1**.
b) Minimum column-to-beam strength ratio given in **13.2.2.4(a)**.

13.3.3.2 *Openings in Walls*

When structural walls have openings, the provisions hereunder shall be complied with.

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

13.3.3.2.1 Additional steel reinforcement shall be provided along all four edges of openings in walls. Further:

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.

13.3.3.3 *Coupling Beams*

When coupling beams connect coplanar special structural walls, they shall be classified as:

a) Flexure-controlled coupling beams if $(L_s/D) > 5$, and
b) Shear-controlled coupling beams if $(L_s/D) \le 5$.

They shall meet the following requirements.

a) Flexure-Controlled Coupling Beams

These beams shall be designed for shear force demand as specified in **13.1.2** and **13.1.3**.

b) Shear-Controlled Coupling Beams

If earthquake induced shear stress τ_{ve} in *Shear-Controlled Coupling Beams* exceeds

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

where L_s is clear span of coupling beam and D overall depth, but is less than $\tau_{c,\max}$, the entire earthquake-induced shear force, bending moment and axial compression shall be resisted by diagonal reinforcement alone. Here, $\tau_{c,\max}$ shall be taken as:

$$\tau_{c,\max} = (5MPa)\delta$$

where

$$\delta = \begin{cases} \left(1 + \dfrac{3P_u}{A_g f_{ck}}\right) \le 1.5 & \text{when beam in compression} \\ \left(1 - \dfrac{12P_u}{A_g f_{ck}}\right) \ge 0.0 & \text{when beam in tension} \end{cases}$$

Further,

1) area of this diagonal reinforcement along each diagonal shall be estimated as:

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

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

2) 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 **13.2.3.3.3** at a spacing not exceeding 100 mm.

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. 47).

If earthquake induced shear stress τ_{ve} in *Shear-Controlled Coupling Beams* exceeds $\tau_{c,\max}$, the coupling beam shall be redesigned to reduce τ_{ve} below $\tau_{c,\max}$.

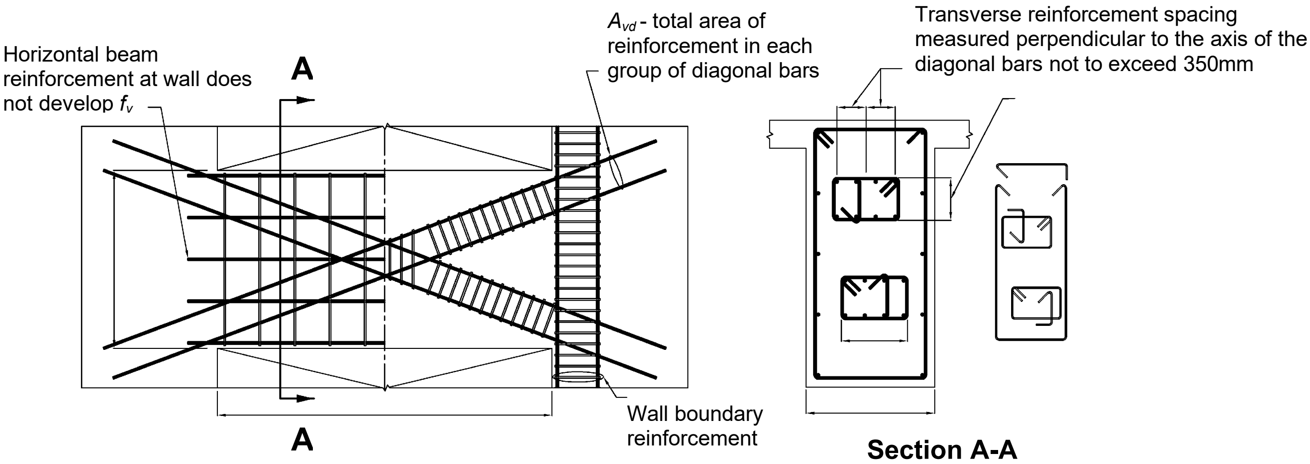


Fig.47 Coupling Beams with diagonal reinforcement bars

13.3.3.4 Construction Joints

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

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

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.

13.3.4 Ductile Detailing

13.3.4.1 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 14 shall be provided along vertical and horizontal directions.

13.3.4.2 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 $0.25\sqrt{f_{ck}}$; 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.

Table 14 Minimum Reinforcement in RC Structural Walls
(*Clause* 13.3.4.1)

Sl No.	Wall Configuration	Reinforcement Details	
		Without Boundary Element	*With Boundary Element*
(1)	(2)	(3)	(4)
i)	**Slender** $2.0 \le \left(\frac{H_w}{L_w}\right)$	$\rho_{h,\min} = 0.0040 + 0.0005\left(\frac{H_w}{L_w} - 2\right)$ $\rho_{v,web,\min} = 0.0080$	$\rho_{h,\min} = 0.0040 + 0.0005\left(\frac{H_w}{L_w} - 2\right)$ $\rho_{v,web,\min} = 0.0025$ $\rho_{v,be,\min} = 0.0080$
ii)	**Intermediate** $1.0 \le \left(\frac{H_w}{L_w}\right) \le 2.0$	$\rho_{h,\min} = 0.0040$ $\rho_{v,web,\min} = 0.0065$	$\rho_{h,\min} = 0.0040$ $\rho_{v,web,\min} = 0.0040$ $\rho_{v,be,\min} = 0.0080$
iii)	**Squat** $\left(\frac{H_w}{L_w}\right) \le 1.0$	$\rho_{h,\min} = 0.0040$ $\rho_{v,\min} = 0.0035 + 0.0060\left(\frac{H_w}{L_w} - 0.5\right)$ ≥ 0.0035	$\rho_{h,\min} = 0.0040$ $\rho_{v,\min} = 0.0025 + 0.0060\left(\frac{H_w}{L_w} - 0.5\right)$ ≥ 0.0025 $\rho_{v,be,\min} = 0.0080$

13.3.4.3 *Longitudinal reinforcement*

13.3.4.3.1 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.

13.3.4.3.2 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.

13.3.4.4 *Transverse reinforcement*

13.3.4.4.1 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.

13.3.4.5 *Development length, splice and anchorage requirement*

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

b) In slender walls (H/Lw>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:
i) L_w above the base of the wall; and
ii) 1/6th of the wall height;
but not larger than $2L_w$.

c) *Splices*

1) *Lap splices*

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

i) The spacing of these links shall not exceed 150 mm.
ii) The lap length shall not be less than the development length of the largest longitudinal reinforcement bar in tension.
iii) Lap splices shall be provided only in the central half of clear wall height, and not,
(1) within a joint; or
(2) within a distance of 2*d* from a location, where yielding of reinforcement is likely to take place; and
iv) Not more than 50 percent of area of steel bars shall be spliced at any one section.

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

2) Mechanical Couplers

Mechanical couplers (conforming to IS 16172) 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.

3) *Welded Splices*

Welding is not permitted in any earthquake zone between reinforcing bars, namely:

i) Between longitudinal steel bars,
ii) Between transverse steel bars, namely links, ties, inserts or any other part of vertical or horizontal reinforcement bar required as per design, and
iii) Between longitudinal and transverse steel bars.

13.4 Beam–Column and Beam–Wall Joints

When a beam transfers bending moment to a column or a structural wall, the area common to the column or structural wall and the beam shall be examined as per the provisions given hereunder.

13.4.1 *Geometry*

13.4.1.1 *Width of beam-column or beam-wall joint*

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

When a beam meets a structural wall perpendicular to the weak axis of the structural wall, then the structural wall shall be provided with a stiffener element (like a column) in line with the structural wall to support the beam, provided the beam is a part of a lateral load resisting system.

13.4.2 *Ductile Design*

13.4.2.1 *Capacity design principle*

The joint shall be designed to resist the demands imposed on it by the columns and beams going into inelastic actions, without any damage.

13.4.2.2 *Shear demand on the joint*

a) Design shear stress demands τ_{jdX} and τ_{jdY} acting horizontally along each of the two principal plan directions X and y, respectively, of the joint shall be estimated as:

$$\tau_{jdX} = \frac{V_{jdX}}{A_{ej}}, \text{ and}$$

$$\tau_{jdY} = \frac{V_{jdY}}{A_{ej}},$$

where

V_{jdX} = Horizontal shear force demand on the joint acting horizontally on the joint in principal plan direction X,

V_{jdY} = Horizontal shear force demand on the joint acting horizontally on the joint in principal plan direction Y,

A_{ej} = Effective shear area of joint
= $b_j w_j$,

in which

b_j = Effective breadth of joint perpendicular to the direction of shear force, and

w_j = Effective width of joint along the direction of shear force (*see* Fig. 48)

$$= \begin{cases} Min[b_b;\ b_c + 0.5h_c] & b_c < b_b \\ b_b & b_c \geq b_b \end{cases},$$

wherein

b_b = Width of beam,

b_c = Width of column, and

h_c = Depth of column in considered direction.

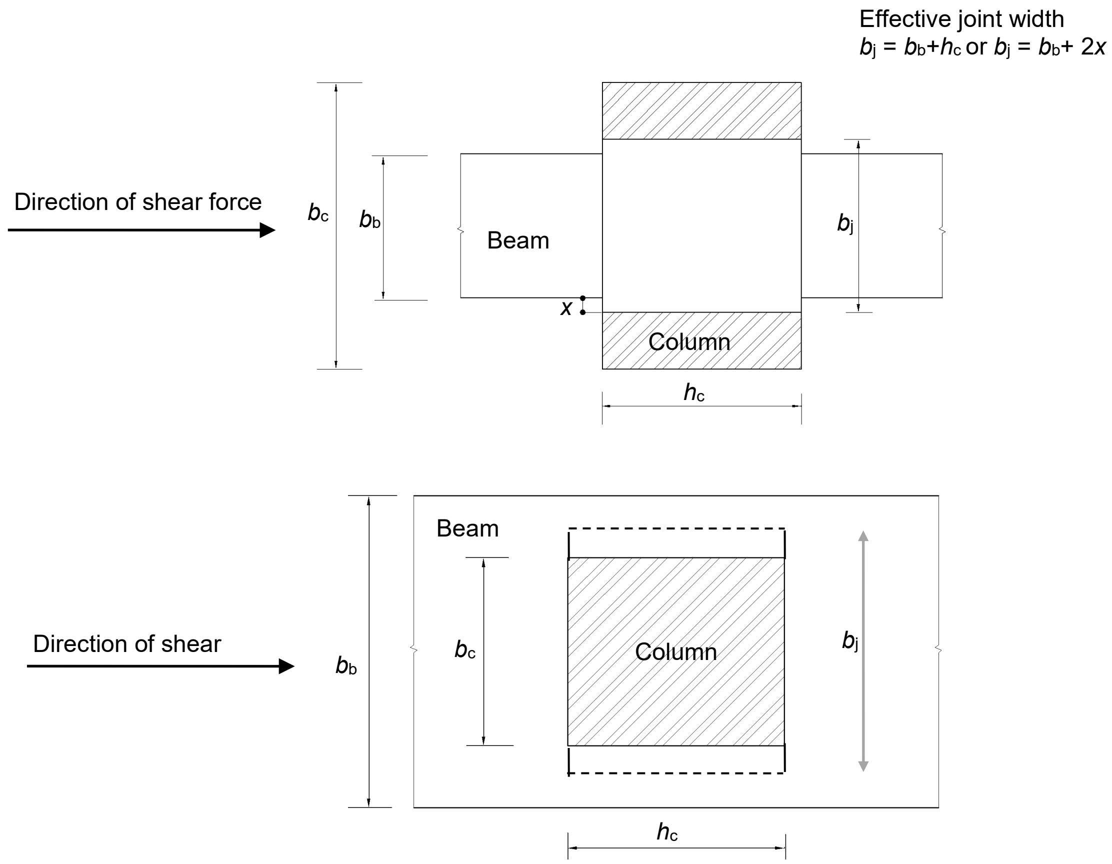


FIG. 48 PLAN VIEW OF A BEAM-COLUMN JOINT SHOWING EFFECTIVE BREADTH AND WIDTH OF JOINT

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

c) Either τ_{jdX} or τ_{jdY} shall not exceed the distortional shear capacity of concrete in the joint estimated using **13.4.2.3**.

13.4.2.3 *Shear Strength Capacity of a Joint*

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

$$\tau_{jc} = \begin{cases} 1.5\sqrt{f_{ck}} & \text{for a joint confined by beams on all 4 of its faces} \\ 1.2\sqrt{f_{ck}} & \text{for a joint confined by beams on 3 of its faces} \\ 1.0\sqrt{f_{ck}} & \text{for other joints} \end{cases},$$

13.4.3 *Ductile Detailing*

13.4.3.1 *Longitudinal*

All longitudinal reinforcement shall be placed inside the column ties placed inside the joint.

13.4.3.2 *Transverse*

a) *Confining Reinforcement in Joints*

1) When all four vertical faces of the joint have beams framing into them covering at least 75 percent of the width on each face,
   i) At least half the special confining reinforcement required as per **13.2.3.3.3(a)** 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 links shall not exceed 150 mm.
2) 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 percent of the width on any face,
   i) Special confining reinforcement required as per **13.2.3.3.3(a)** 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 links shall not exceed 150 mm.

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

13.5 Floor Slabs

13.5.1 *Geometry*

The overall geometry of concrete monolithic slab-beam floors or those consisting of prefabricated or precast elements with reasonable reinforced screed concrete (at least a minimum of 50 mm on floors and of 75 mm on roof, with at least a minimum reinforcement of 6 mm bars spaced at 150 mm centers) as topping, shall be such that the flexible floor diaphragm action is not activated.

13.5.1.1 *Aspect ratio in plan*

The plan aspect ratio shall be less than 3 to ensure rigid diaphragm action.

13.5.1.2 Re-entrant corners

A building is said to have a re-entrant corner in any plan direction, when its structural configuration in plan has a projection of size greater than 15 percent of its overall plan dimension in that direction.

13.5.1.3 *Openings and cut-outs*

Openings in slabs result in flexible diaphragm behaviour, and hence the lateral shear force is not shared by the frames and/or vertical members in proportion to their lateral translational stiffness. The problem is particularly accentuated when the opening is close to the edge of the slab. A building is said to have discontinuity in their in-plane stiffness, when floor slabs have cut-outs or openings of area more than 50 percent of the full area of the floor slab

14 DESIGN OF STRUCTURAL SYSTEMS

Special requirements of special moment resisting frame (SMRF), and special moment resisting frame with special structural wall, designed as lateral load resisting system in buildings, are provided hereunder.

14.1 Special Moment Frame Buildings

Special moment resisting frames (SMRFs) of reinforced concrete shall be designed to satisfy the requirements of this section.

14.1.1 *Basis of Design*

Special moment resisting frames designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through flexural yielding of the beams and minimal yielding of columns. Special moment resisting frames may be used in any Earthquake Zones except in buildings taller than 15 m in Earthquake Zones IV and V [see IS 1893 (Part 1)] and for any buildings (importance factor values). Yielding of beam column joints in SMRFs is not permitted by this standard.

14.1.2 *Analysis*

It is preferable to plan buildings to have independent planar lateral load resisting moment frames in each principal plan directions. In such cases, there are no special analysis requirements. But when two moment frames oriented in orthogonal directions intersect, the possibility of beam yielding in both orthogonal directions simultaneously shall be considered in the design of the common column.

14.1.3 *System Requirements*

The requirements given hereunder shall be satisfied by the building system.

14.1.3.1 *Infill Walls*

Infill walls are permitted to be used in moment frame buildings. The effect of stiffness contributed by the infill shall be appropriately considered as per CED 39 (22345).

14.1.4 *Member Requirements*

The requirements given hereunder shall be satisfied by the component or member of a special moment frame.

14.1.4.1 *Beams*

Requirements specified in **13.1** shall apply.

14.1.4.2 *Columns*

Requirements specified in **13.2** shall apply.

14.1.4.3 *Beam-Column Joints*

Requirements specified in **13.4** shall apply.

14.1.4.4 *Strong Column Weak Beam Design*

Requirements specified in **13.2.2.4** shall apply.

14.2 Special Moment Frame Buildings with Special Structural Walls

Special moment resisting frames with special structural walls designed to act as lateral force resisting system shall satisfy the requirements of this section.

14.2.1 *Basis of Design*

Special moment resisting frames with special structural walls designed in accordance with these provisions are expected to provide inelastic deformation capacity through inelastic action in the structural walls, in beams and relatively minimal yielding of columns. Special moment resisting frames with special structural walls may be used in any Earthquake Zones. Yielding of beam-column and beam-wall joints in special moment resisting frames with special structural walls is not permitted by this standard.

14.2.2 *Analysis*

It is preferable to plan buildings to have independent planar lateral load resisting systems in each principal plan directions. In such cases, there are no special analysis requirements. But when two planar lateral load resisting systems oriented in orthogonal directions intersect, the possibility of beam yielding in both orthogonal directions simultaneously shall be considered in the design of the common column or structural wall. In general, flexural yielding of beams supported on the weaker axis of a structural wall is not permitted.

14.2.3 *System Requirements*

The requirements given hereunder shall be satisfied by the building system.

14.2.3.1 *Infill walls*

Infill walls are permitted to be used in moment frame buildings. The effect of stiffness contributed by the infill shall be appropriately considered as per CED 39 (22345).

14.2.4 *Member Requirements*

The requirements given hereunder shall be satisfied by the component or member of a special moment frame.

14.2.4.1 *Beams*

Requirements specified in **13.1** shall apply.

14.2.4.2 *Columns*

Requirements specified in **13.2** shall apply.

14.2.4.3 *Structural Walls*

Requirements specified in **13.3** shall apply.

14.2.4.4 *Beam-Column Joints*

Requirements specified in **13.4** shall apply.

14.2.4.4 *Strong Column – Weak Beam Design*

Requirements specified in **13.2.2.4** shall apply.

14.3 Dual System Buildings

When a building is provided with a Dual System (having Special Moment Frames and Special Structural Walls) for lateral load resistance [as per CED 39 (22345)] it shall comply with the provisions specified hereunder.

14.3.1 *Structural Configuration*

The following are applicable to structural configuration of buildings with Dual System:

a) Along any one plan direction, at least two planes of dual system shall be provided, and they shall be located such that the building does not have irregularities as described in CED 39 (22345).
b) The building need not have Dual System along both plan directions.
c) The Structural Plan Density of RC structural walls alone along each plan direction shall not be less than that specified in CED 39 (22345) for RC structural walls when used in with moment resisting frames (without being a Dual System).

SECTION 4

ADDITIONAL CRITERIA FOR STEEL BUILDINGS

15 GENERAL SPECIFICATIONS

The additional requirements for earthquake resistant design of steel structures are provided in this section.

15.1 Design Requirement

The general requirements for design and detailing of ductile steel buildings address the following broad aspects:

a) Stability,
b) Stiffness,
c) Strength,
d) Deformability, and
e) Ductility.

Thus, steel buildings shall be designed and detailed as per this standard to resist design earthquake hazard defined in CED 39 (22343).

16 MATERIALS AND SECTIONS

The materials and sections used as part of lateral force resisting systems in steel buildings shall confirm to the following.

16.1 Materials

The provisions of this standard on earthquake resistant design of structural steel buildings are applicable for the following grades of materials:

a) Structural steel sections and plates of grades E250 (B0 or C), E275 (B0 or C), E300 (B0 or C) or E350 (B0 or C) as per IS 2062.
b) Any other equivalent grade of steel satisfying the following:
    i) Characteristic yield stress f_y of structural steel sections and plates in which inelastic action is expected shall not exceed 350 MPa;
    ii) Structural steel shall have minimum elongation of 22 percent;
    iii) Structural steel shall have minimum Charpy V-Notch impact test value of 27J at 0 °C; and
    iv) The ratio of the ultimate strength to the yield strength shall at least be 1.15.

For this standard, the expected yield strength and expected ultimate strength of a member, section or connection shall be determined based on the expected yield stress and expected tensile stress. The expected yield stress and expected tensile stress of a grade of steel shall be considered to be equal to $R_y f_y$ and $R_u f_u$ respectively, where R_y is the ratio of the expected yield stress to the characteristic yield stress f_y, and R_u the ratio of the expected tensile stress to the characteristic tensile stress f_u of that material. In short, R_y and R_u are referred as material strength uncertainty factors; their values for different grades of steel are given in Table 15.

16.1.1 For any other steel satisfying **16.1(b),** values of 1.4 and 1.2 shall be considered for R_y and R_u respectively.

Table 15: Material Strength Uncertainty Factors R_y and R_u

(*Clause* 16.1)

Sl No.	Grade of Steel	Material Strength Uncertainty Factors	
		R_y	R_u
(1)	(2)	(3)	(4)
i)	E250 (B0 or C)	1.4	1.2
ii)	E275 (B0 or C)	1.4	1.2
iii)	E300 (B0 or C)	1.3	1.1
iv)	E350 (B0 or C)	1.2	1.1

16.1.2 All welds, conforming to relevant Indian Standards, shall be made with filler metals satisfying minimum elongation of 20 percent, and Charpy V-Notch impact test value of 47 J at 0°C.

16.1.3 All demand critical welds shall have Charpy V-Notch impact test value of at least 27 J at -20°C.

16.1.4 Bolts, nuts and washers shall conform to relevant Indian Standards, such as IS 1363 (Parts 1, 2 & 3), IS1364 (Parts 1 to 6), IS 3640, IS 3757, IS 4000, IS 5624, IS 6610, IS 6639 and IS 6649.

16.2 Section Classification

Structural sections of lateral load resisting system shall comply with the width-to-thickness requirements specified in Table 16.

Table 16 Limiting Width-to-Thickness Ratios for Compression Elements of Earthquake Resistant Structures
(*Clause* 16.2)

Sl No.	Component	Section Type	Limiting Plate Slenderness	
			Outstanding Flange Width-to-Thickness Ratio	Web Depth-to-Thickness Ratio
(1)	(2)	(3)	(4)	(5)
i)	Beam	Doubly-symmetric **rolled** I-sections	$\frac{9.0\varepsilon}{\sqrt{R_y}}$	$\frac{44.5\varepsilon}{\sqrt{R_y}}$
		Doubly-symmetric **built-up** I-sections		
ii)	Column	Doubly-symmetric **rolled** I-sections	$\frac{9.0\varepsilon}{\sqrt{R_y}}$	$\frac{72.7\varepsilon}{\sqrt{R_y}}(1-1.04C_a)$ for $C_a \leq 0.118$ and $\frac{24.9\varepsilon}{\sqrt{R_y(2.68-C_a)}} \geq \frac{44.4\varepsilon}{\sqrt{R_y}}\sqrt{\frac{E}{R_y f_y}}$ for $C_a > 0.118$
		Doubly-symmetric **built-up** I-sections		
iii)	Brace	Rolled or **built-up** I-sections	$\frac{11.3\varepsilon}{\sqrt{R_y}}$	$\frac{44.4\varepsilon}{\sqrt{R_y}}$
		Closed box sections	$\frac{21.4\varepsilon}{\sqrt{R_y}}$ (Flange width is the flange width minus twice the thickness of the webs)	$\frac{21.4\varepsilon}{\sqrt{R_y}}$
iv)	Links	Doubly-symmetric **rolled** or **built-up** I-sections	$\frac{11.3\varepsilon}{\sqrt{R_y}}$	$\frac{44.4\varepsilon}{\sqrt{R_y}}$
		Closed box sections	$\frac{21.4\varepsilon}{\sqrt{R_y}}$ (Flange width is the flange width minus twice the thickness of the webs)	$\frac{49.4\varepsilon}{\sqrt{R_y}}$

where

$\varepsilon = \sqrt{\frac{250}{f_y}}$, and

$$C_a = \frac{P_u}{P_y / \gamma_{m0}}.$$

17 DESIGN OF MEMBERS AND COMPONENTS

The members, components, connections and joints which are part of lateral force resisting systems in steel buildings shall be designed according to the following.

17.1 Beams

17.1.1 *Sections*

Only doubly-symmetric parallel flange standard rolled sections (as per **IS 808**) or built-up sections, with flange width to thickness ratio and web depth to thickness ratio values less than the limits specified in Table 16, shall be used as beams. The flange to web weld in built-up beams shall be continuous.

17.1.2 *Slenderness*

The ratio of the maximum unbraced length of the compression flange of a beam L_{br} to the radius of gyration r_y about the weaker axis of the beam cross-section shall not exceed 25 over a length of $2d_b$ from the end of the beam to column connection. L_{br}/r_y for the remaining portion of the beam shall not exceed $0.10E/(R_y f_{yb})$.

17.1.3 *Bracing*

The stiffness of bracing shall be as given below:

a) Beams shall be restrained against rotation about their longitudinal axis at supports and at intermediate locations along the length of the beam through the use of internal panel bracing without any external rigid support.
b) The lateral bracing shall be attached at or near the compression flange of the beam.
c) The lateral bracing shall be attached at or near both flanges, near the point of inflection in beams bending in double curvature.

17.1.3.1 *Stiffness of bracing*

The stiffness of bracing shall be as given below:

a) The shear stiffness of the panel bracing system closest to the inflection point in a beam bending in double curvature shall at least be:

$$K_{br} \geq \frac{10 R_y M_{pb}}{L_{br} d_f}$$

where L_{br} is the unbraced length of the beam, and d_f is the distance between centroids of the flanges of the beam.

b) The shear stiffness of the panel bracing system other than near the inflection point in a beam bending in double curvature shall at least be:

$$K_{br} \geq \frac{5 R_y M_{pb}}{L_{br} d_f}$$

17.1.3.2 *Strength of bracing*

The shear strength of the panel bracing system shall at least be:

$$V_{br} \geq \frac{0.025 R_y M_{pb}}{d_f}$$

17.1.3.3 *Special bracing at plastic hinge locations*

Special bracing shall be located adjacent to expected plastic hinge locations. Both flanges of beams shall be laterally braced. The axial strength of such lateral bracing shall be at least:

$$P_{br} \geq \frac{0.06 R_y M_{pb}}{d_f},$$

and the required bracing stiffness shall be as in **17.1.3.1**(b)**.**

17.1.4 *Strength*

The design strength of beam shall satisfy the load combinations in CED 39 (22343), and the overstrength load combinations specified in **15.3** in SCBFs and EBFs, or when a beam is part of diaphragm collector or chord.

17.1.4.1 *Shear Strength*

The design shear strength of the beam at the location of the plastic hinge shall be determined as per IS 800, and it shall be at least equal to the shear demand specified in **17.3.4.**

17.1.5 *Splice*

Beam splices shall be located at least $3d_b$ away from the face of the column or d_b from the line of action of any concentrated force acting on the beam. The design strength of beam splices shall at least be 1.80 times the required strength at the section, except at beam-column connections. Further, design strength of each flange splice plate shall at least be $1.2R_y f_y A_f$, where A_f is the area of the flange being spliced.

17.2 Columns

17.2.1 *Sections*

Only doubly-symmetric parallel flange standard rolled as per IS 808 or built-up sections, with flange width to thickness ratio, and web depth to thickness ratio less than the corresponding limits specified in Table 2, shall be used as columns. The flange to web weld in built-up beams shall be continuous.

17.2.2 *Slenderness*

The slenderness ratio of unbraced length of columns shall be less than 75.

17.2.3 *Bracing*

Columns shall be laterally braced at the supports, and at intermediate locations (if required) along the length of the column with internal panel bracing.

17.2.3.1 *Stiffness of bracing*

The shear stiffness of the panel bracing system, in the direction perpendicular to the longitudinal axis of the columns, shall at least be:

$$K_{br} \geq \frac{3P_u}{L_{br}},$$

where

L_{br} = Unbraced length of the column; and

P_u = Maximum factored axial load.

17.2.3.2 *Strength of bracing*

The shear strength of the panel bracing system shall at least be:

$$V_{br} \geq 0.005P_u$$

17.2.3.3 *Strength of bracing connection*

The connection of the bracing system to the column shall have design strength at least equal to:

$$P_{br} \geq 0.01P_u,$$

subject to minimum design action on connection given in IS 800.

17.2.4 *Strength*

Columns shall have design strength more than the maximum demand arising from the following:

a) Structural analysis based on load combinations specified in **15.3**, and
b) Maximum loads transferred to the column considering 1.2 times the strength of the connected members (beams, braces, etc) determined considering material strength uncertainty factor.

17.2.4.1 Columns in both moment frames and braced frames that are common to intersecting frames aligned along two orthogonal directions, shall consider in design the potential for simultaneous inelasticity from all such frames for determination of the required axial strength.

The direction of application of the load in each such frame shall be selected to produce the most severe load effect on the column.

17.2.4.2 Columns in buildings designed to resist effects of earthquake shaking shall not carry tensile forces.

17.2.5 *Splice*

Column splices shall be in the middle third of the height of the columns, at least 1.0 m away from the beam-to-column moment connection.

The design strength demand on column splices shall be that determined using **15.3**. Further, the design strength of both flange and web splice plates shall at least be $1.2R_y$ times of their respective strengths.

17.3 Beam–Column Joint

At a beam–column joint, the following design aspects shall be addressed:

a) Column to beam strength ratio,
b) Joint panel zone design, and
c) Beam–column connection design.

17.3.1 *Basis of Design*

Flexural plastic hinges are expected to be formed at the end regions of the beams away from the column face. Under this condition, the column and the beam-column joint, including the beam–column connection, is expected to remain elastic and shall be designed as capacity protected elements.

17.3.2 *Column to Beam Strength Ratio*

At a beam–column joint, the following strength ratio shall be satisfied:

$$\frac{\sum M_{pc}}{\sum M_{bo}} = \frac{\sum Z_{pc} f_{yc}\left(1-\frac{P_u}{P_d}\right)}{\sum 1.1 R_y Z_{pb} f_{yb}} > 1.4$$

where Z_{pc} and Z_{pb} are the plastic section modulus and f_{yc} and f_{yb} are the characteristic yield strength of column and beam cross-sections respectively, P_u is the maximum factored axial compressive load and P_d is the design strength under axial compression, and R_y is the material uncertainty factor corresponding to the grade of steel in beams.

17.3.2.1 The above requirement need not be satisfied at the roof level.

17.3.3 *Joint Panel Zone*

Shear yielding of joint panel zone (JPZ) shall be limited. Use of continuity and doubler plates is permitted (Fig. 49).

17.3.3.1 *Panel zone demand*

The shear force demand at the face of the column flanges shall approximately be taken as:

$$V_{pzd} = \sum\left(\frac{1.1R_y f_y Z_{pb}}{0.95 d_b}\right)$$

17.3.3.2 *Panel zone capacity*

The design shear strength capacity shall be:

$$V_{pzc} = \frac{f_y}{\gamma_{m0}\sqrt{3}}\left(0.95d_c\right)t_{pz}$$

where

t_{pz} = thickness of the panel zone, including thickness of doubler plates if provided, and

γ_{m0} = 1.1.

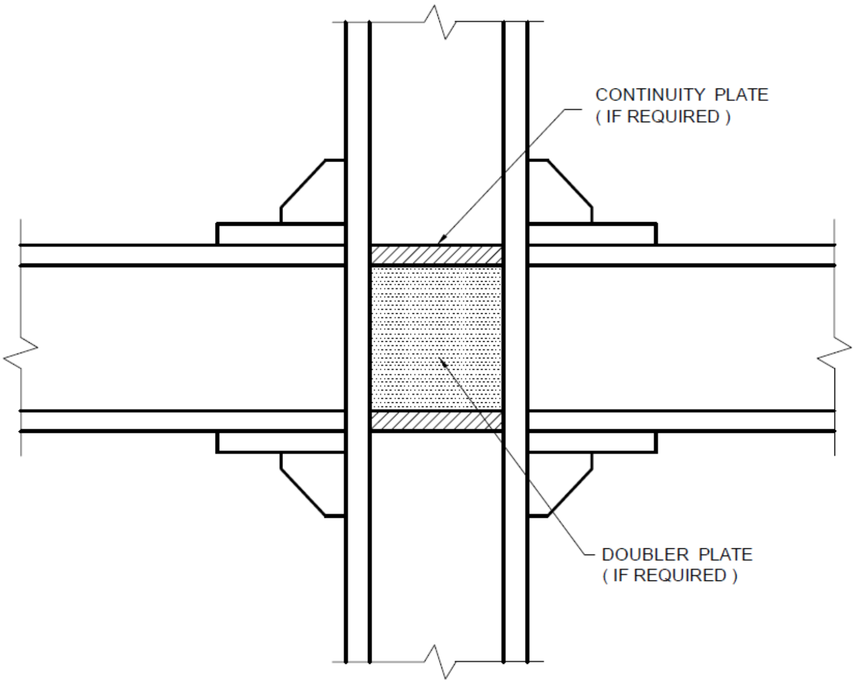


FIG. 49 TYPICAL INTERIOR REINFORCED BEAM-COLUMN JOINT

17.3.3.3 *Panel zone thickness*

The individual thickness of the column web and doubler plates (when provided), shall be more than:

$$\frac{d_{\mathrm{pz}} + w_{\mathrm{pz}}}{90}$$

where

$d_{\mathrm{pz}} = d_{\mathrm{b}} - 2t_{\mathrm{bf}}$ of the deeper beam, in mm; and

$w_{\mathrm{pz}} = d_{\mathrm{c}} - 2t_{\mathrm{cf}}$, in mm.

17.3.3.4 *Doubler plate*

Doubler plates shall be provided when the thickness of the column web within the panel does not satisfy the strength requirements. It is permitted to use doubler plates with or without continuity plates. When continuity plates are not provided, doubler plates shall extend at least 150 mm beyond the deeper beam flange levels on either sides of the panel zone.

17.3.3.5 *Continuity plate*

Continuity plates shall be provided when,

$$\frac{6.25 f_y t_{cf}^2}{1.1} \leq 1.2 R_y f_y b_{bf} t_{bf}$$

When provided, thickness of continuity plate shall not be less than thickness of the thinner beam flange on either side of the column, and width not less than the distance of the tip of the wide beam flange from the face of the column web. If different grades of steel are used, appropriate values of f_y shall be used on either side of the inequality.

17.3.4 *Beam–Column Connection*

Fully-restrained, reinforced beam–column connections shall be used in moment frames, capable of transferring at least a bending moment of $1.1 R_y f_y Z_{pb}$, and shear demand determined based on capacity design principle considering, (i) beams to be bending in double curvature, (ii) plastic hinges of strength $1.1 R_y f_y Z_{pb}$ assumed to act at a distance $d_b/2$ from the end of the connection, and (iii) gravity load required to be carried.

17.3.4.1 *Welded beam–column connection*

In general, beam flanges shall be connected to column flanges using complete joint penetration groove welds, while beam web shall be connected to the column flange using either a complete joint penetration groove weld extending between weld access holes, or using a bolted single plate shear connection.

A weld access hole detail shall be adopted to ensure that the location of maximum plastic strain does not occur at the interface between the beam web and beam flange but is entirely in the beam web (Fig. 50); the typical details for rolled beam shapes welded from one side steel backing plates shall include:

a) Standard bevel,
b) Angle not to exceed 25°,
c) Radius not less than 10 mm, and
d) Dimension not less than ¾ t_{bf} or 20 mm, whichever is greater.

Use of no weld access hole detail is not permitted. The baking bar shall be removed after welding and the surfaced finished by grinding.

In general, a cover plated welded flange connection with welded rib plates at both beam flange levels is preferable.

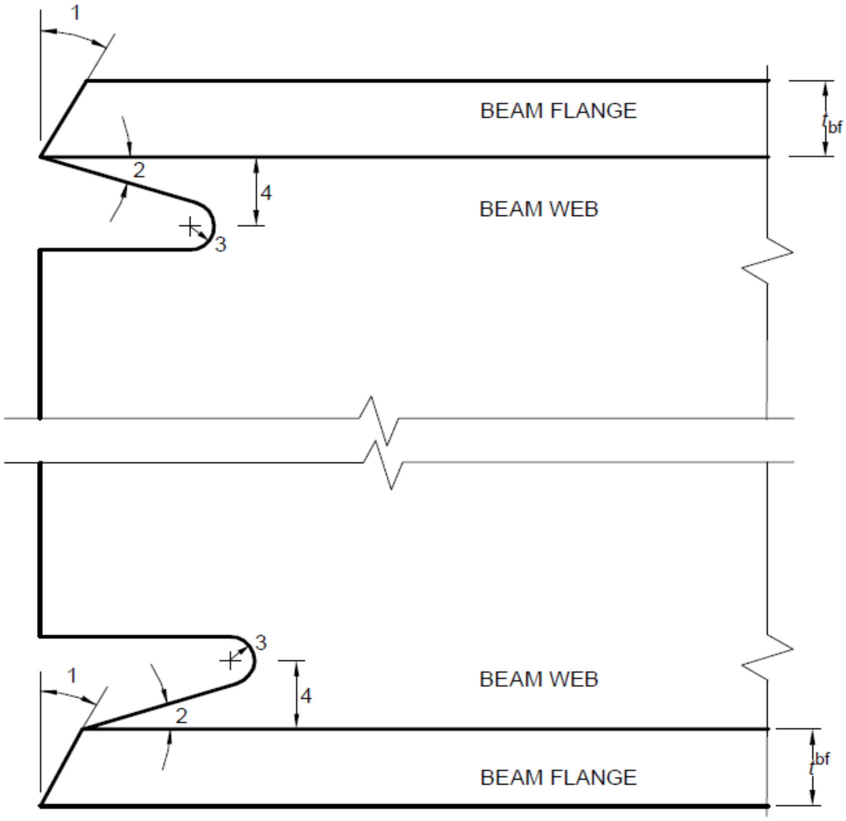


FIG. 50 WELD ACCESS HOLE

17.4. Column Base

Column bases may have any form of embedded connection or anchor bolted base plate connection. The degree of fixity offered by a connection should be established and used in structural analysis.

17.4.1 *Anchor Bolted Base Plate Connection*

Anchor bolted base plate connections at column bases shall be designed to prevent the following:

a) Bearing failure of concrete under compression,
b) Pullout cone failure of concrete due to tensile force in anchor bolts,
c) Side face blowout failure of concrete due to tensile force in anchor bolts with headed or hooked ends,
d) Wedge-cone failure of concrete due to shear force in anchor bolts,
e) Pryout cone failure of concrete due to shear force in anchor bolts, and
f) Bolt-concrete bond slip failure.

17.4.2 *Strength*

The required design strength of the steel elements at the column base, including base plate, anchor bolts, stiffening plates, and shear lug elements shall be determined for the load combinations given in **15.3**.

17.4.3 *Fixed Column Base*

Fixed column base connections and supporting foundation shall be designed to resist moment demand of $1.1R_yM_{pc}$ and shear demand equal to $2.2R_yM_{pc}/H_c$, *where* M_{pc} and H_c are the plastic moment capacity and the clear height of the column between the beam connections, respectively.

17.4.4 *Pinned Column Base*

In lieu of detailed calculations establishing rotational stiffness (based on the degree of fixity) and bending moment strength characteristics, it is permitted to analyse and design anchor bolted base plate connections at column bases in buildings as pinned connection. In such cases, the connection and supporting foundation shall be designed for minimum moment of $0.5R_yM_{yc}$, where M_{yc} is the yield moment capacity of the column section, in addition to shear force demand equal to $1.1R_yM_{pc}/H_c$.

17.5 Structural Braces

Structural braces may be used to impart lateral stiffness and strength to building frames. Such braces shall be provided in selected bays over the full height of the building frame. Braces part of concentrically braced frames (CBF) shall also conform to the requirements given hereunder.

17.5.1 *Sections*

Standard rolled or built-up sections or closed box sections, with flange width to thickness ratio and web depth to thickness ratio less than the limits specified in Table 16, shall be used as braces. The weld between the elements of built-up section shall be continuous.

17.5.2 *Slenderness*

The effective slenderness ratio of braces shall be less than 160.

In built-up braces, at least two connectors shall be provided at uniform spacing between the ends. The slenderness ratio of individual elements between the connectors shall be less than 0.4 times the governing effective slenderness ratio of the built-up brace.

Sum of the shear strengths of the connectors shall be greater than the expected tensile strength of each element, except when buckling of the brace about its critical bucking axis does not cause shear in the connectors. Connectors shall not be located within the middle one-fourth of the clear brace length.

17.5.3 *Effective Area*

Brace effective net area shall not be taken less than the gross cross-sectional area of the brace. Where reinforcement on braces is used, the following requirements shall apply:

a) The characteristic yield strength of the reinforcement, when provided in the form of steel plates, shall be at least equal to the characteristic yield strength of the brace, and
b) The connections of the reinforcement to the brace shall have sufficient strength to develop the expected reinforcement strength on each side of a reduced section.

17.5.4 *Bracing Connection*

The required strength in tension, compression, and flexure of brace connections (including beam-to-column connections if part of the braced-frame system) shall be determined as required in the following. These required strengths are permitted to be considered independently without interaction.

17.5.4.1 *Tensile Strength*

The tensile strength of brace connections shall at least be the lesser of the following:

a) The expected yield strength in tension of the brace, determined as maximum of $1.1R_y f_y A_g$ and $R_u f_u A_n$, and
b) The maximum load effect, indicated by analysis as in **15.6**, that can be transferred to the brace by the system.

17.5.4.2 *Compressive strength*

The compressive strength of brace connections shall at least be equal to the brace strength in compression, generally as governed by buckling.

17.5.4.3 *Accommodation of brace buckling*

Brace connections shall be designed to withstand the flexural forces and rotations imposed by brace buckling. Connections satisfying either of the following provisions are deemed to satisfy this requirement:

a) *Required Flexural Strength*
Brace connections designed to withstand the flexural forces imposed by brace buckling shall have a required flexural strength equal to 1.1 times the expected brace flexural strength. The expected brace flexural strength shall be determined as $R_y f_y Z_p$ of the brace about the critical buckling axis.

b) *Rotation Capacity*
Brace connections shall be designed to withstand the rotations imposed by brace buckling. Inelastic rotation of the connection is permitted.

17.5.4.4 *Gusset plates*

To accommodate brace buckling, gusset plates shall be detailed to undergo out-of-plane bending and welds that attach a gusset plate directly to a beam flange or column flange shall be designed to have shear strength per unit length equal to $R_y f_y t_p / \sqrt{3}$, where t_p is the thickness of the gusset plate.

17.6. *Shear Links*

Shear links may be used as structural fuse in eccentrically braced frames (EBF) as specified in this standard. These links are expected to be subjected to combined action of bending moment and shear force and undergo yielding under earthquake effects.

17.6.1 *Sections*

Links shall be I-shaped cross sections (standard rolled wide-flange sections or built-up sections), or built-up box sections, with flange width to thickness ratio and web depth to thickness ratio less than the limits specified in Table 16. Hollow Structural Steel (HSS) sections shall not be used as links. Further:

a) The web or webs of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted;
b) For links made of built-up cross sections, complete-joint-penetration groove welds shall be used to connect the web (or webs) to the flanges; and
c) Links of built-up box sections shall have a moment of inertia, I_y, about an axis in the plane of the EBF greater than $0.67I_x$, where I_x is the moment of inertia of the link about an axis perpendicular to the plane of the EBF.

17.6.2 *Link Shear Strength*

The design shear strength of link shall be the lower value of the following:

a) For shear yielding: $\dfrac{V_{pL}}{\gamma_{m0}}$

where

$$V_{pL} = \begin{cases} \dfrac{f_y}{\sqrt{3}} A_{wL} & \text{for } P_u/P_y \le 0.15 \\ \dfrac{f_y}{\sqrt{3}} A_{wL} \sqrt{1 - \left(\dfrac{P_u}{P_y}\right)^2} & \text{for } P_u/P_y > 0.15 \end{cases};$$

$$A_{wL} = \begin{cases} (d_L - 2t_f)t_w & \text{, for I - shaped link sections,} \\ 2(d_L - 2t_f)t_w & \text{, for box link sections.} \end{cases}$$

P_u = Factored axial load in the link,

P_y = $f_y A_{gL}$,

A_{gL} = Gross cross-sectional area of link,

d_L = Overall depth of link,

t_f = Thickness of flange,

t_w = Thickness of web, and

γ_{m0} = 1.1

b) For flexural yielding: $\dfrac{2M_{pL}}{e\gamma_{m0}}$

where

$$M_{pL} = \begin{cases} f_y Z_{pL} & \textit{for } P_u/P_y \le 0.15 \\ f_y Z_{pL}\left[1 - \left(\dfrac{P_u}{P_y}\right)\right] & \textit{for } P_u/P_y > 0.15 \end{cases}; \text{ and}$$

Z_{pL} = Plastic section modulus of link about the bending axis.

e = Length of link as per **17.6.3.**

17.6.3 *Length of Link*

The length of link, *e,* shall be less than $1.6M_{\mathrm{pL}}/V_{\mathrm{pL}}$, where M_{pL} and V_{pL} are the plastic bending moment capacity and plastic shear capacity of the link, as per **17.6.2**. Further, the following shall be satisfied:

If $P_{\mathrm{u}}/P_{\mathrm{y}}$ > 0.15, the length of the link shall be limited as follows:

$$e \le \begin{cases} 1.6M_{\mathrm{pL}}/V_{\mathrm{pL}} & \textit{for}\ \ \overline{\rho} \le 0.5 \\ 1.6M_{\mathrm{pL}}\left(1.15-0.3\overline{\rho}\right)/V_{\mathrm{pL}} & \textit{for}\ \ \overline{\rho} > 0.5 \end{cases}$$

where

$$\overline{\rho} = \frac{P_{\mathrm{u}}/P_{\mathrm{y}}}{V_{\mathrm{u}}/V_{\mathrm{y}}}\text{, and}$$

$$V_{\mathrm{y}} = \frac{f_y}{\sqrt{3}}A_{wL}.$$

17.6.4 *Stiffeners for I-Shaped Link Sections*

Web of links shall be stiffened to prevent premature shear buckling. The required strength of fillet welds connecting a stiffener to the link web shall be $f_{\mathrm{y}}A_{\mathrm{st}}$, where A_{st} is the horizontal cross-sectional area of the link stiffener. The required strength of fillet welds connecting the stiffener to the link flanges is $f_{\mathrm{y}}A_{\mathrm{st}}/4$.

17.6.4.1 *End web stiffeners*

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $\left(b_{\mathrm{f}}-2t_{\mathrm{w}}\right)$and a thickness not less than the larger of $0.75t_{\mathrm{w}}$ or 10 mm, where b_{f} and t_{w} are the link flange width and link web thickness, respectively.

17.6.4.2 *Intermediate web stiffeners*

Links shall be provided with intermediate vertical web stiffeners spaced at intervals not exceeding $\left(30\mathrm{t}_{w}-0.2d\right)$.

17.6.5 *Stiffeners for box link sections*

Web of links shall be stiffened to prevent premature buckling. The required strength of fillet welds connecting a stiffener to the link web shall be $f_{\mathrm{y}}A_{\mathrm{st}}$, where A_{st} is the horizontal cross-sectional area of the link stiffener.

17.6.5.1 *End web stiffeners*

Full-depth web stiffeners shall be provided on one side of each link web at the diagonal brace connection. These stiffeners are permitted to be welded to the outside or inside face of the link webs. These stiffeners shall each have a width not less than $b/2$, where b is the inside width of the box section. These stiffeners shall each have a thickness not less than the larger of $0.75t_{\mathrm{w}}$ or 10 mm.

17.6.5.2 *Intermediate web stiffeners*

Box links shall be provided with full-depth intermediate web stiffeners welded either to the outside or inside face of the link webs as follows:

a) When web depth-to-thickness ratio is greater than $\frac{19\varepsilon}{\sqrt{R_y}}$, full-depth web stiffeners shall be provided on one side of each link web, spaced at intervals not exceeding $20t_w - (d - 2t_f)/8$; and

b) When web depth-to-thickness ratio is less than or equal to $\frac{19\varepsilon}{\sqrt{R_y}}$, no intermediate web stiffeners are required.

18 SPECIAL REQUIREMENTS FOR STRUCTURAL SYSTEMS

Special requirements of special moment resisting frame (SMRF), special concentrically braced frame (SCBF), and eccentrically braced frame (EBF), designed as lateral load resisting system in buildings, are provided hereunder.

18.1 Special Moment Resisting Frames

Special moment resisting frames (SMRFs) of structural steel shall be designed to satisfy the requirements of this section.

18.1.1 *Basis of Design*

Special moment resisting frames designed in accordance with these provisions are expected to provide significant inelastic deformation capacity through flexural yielding of the beams, limited yielding of panel zones, and minimal yielding of columns except at base. Special moment resisting frames may be used in any Earthquake Zones except in buildings taller than 15 m in Earthquake Zones IV and V [*see* CED 39 (22343)] and for any buildings (importance factor values). Yielding of beam to column connections in SMRFs shall not be permitted by this standard.

18.1.2 *Analysis*

It is preferable to plan buildings to have independent planar lateral load resisting moment frames in each principal plan directions. In such cases, there are no special analysis requirements. But when two moment frames oriented in orthogonal directions intersect, the possibility of beam yielding in both orthogonal directions simultaneously shall be considered in the design of the common column.

18.1.3 *System Requirements*

The requirements given hereunder shall be satisfied by the building system.

18.1.3.1 *Beams*

Beams in SMRFs are permitted to carry gravity loads through composite action with reinforced concrete slab. For lateral load action, composite action shall not be considered. Further, abrupt changes in beam flanges, through actions like drilling of holes or trimming of flange width, and use of shear studs are prohibited in the beam end regions of length at least twice the depth of the beam where flexural plastic hinges are expected to be formed.

18.1.3.2 *Beam column connections*

Beam to column connections shall be capable of accommodating storey drift angle of 0.04 radians, without loss of strength exceeding 15 percent of the beam plastic moment capacity.

18.1.3.3 *Column to beam strength ratio*

The requirements specified in **17.3.2** shall apply.

18.1.4 *Member Requirements*

The requirements given hereunder shall be satisfied by the building component or member

18.1.4.1 *Beams*

Requirements specified in **17.1** shall apply.

18.1.4.2 *Columns*

Requirements specified in **17.2** shall apply.

18.1.4.3 *Joint panel zone*

Requirements specified in **17.3.3** shall apply.

18.1.4.4 *Beam Column Connections*

Requirements specified in **17.3.4** shall apply.

18.1.4.5 *Protected zones*

The region at each end of a beam equal to twice the depth of the beam subjected to inelastic straining shall be designated as a protected zone. Steel headed stud anchors and other fabrication and erection attachments shall not be placed on beam flanges within the protected zone.

18.1.4.6 *Column splices*

Requirements specified in **17.2.5** shall apply.

8.1.4.7 *Demand critical welds*

The following welds shall be designed as demand critical welds:

a) Groove welds at column splices;
b) Welds at column-to-base plate connections, except when,

i) Column plastic hinge at, or near the base plate is precluded by conditions of restraint, or
ii) There is no net tension under load combinations including the overstrength earthquake load; and

c) Welds in beam–column connections.

18.2 Special Concentrically Braced Frames

Special concentrically braced frames (SCBF) of structural steel shall be designed to satisfy the requirements given hereunder. Collector beams that connect SCBF braces shall be considered to be part of SCBF.

18.2.1 *Basis of Design*

In SCBFs, members shall be concentrically connected. But eccentricities less than the beam depth are permitted if the resulting member and connection forces are addressed in the design, and the eccentricities do not change the expected source of inelastic deformation capacity of the building. SCBF designed in accordance with these provisions are expected to provide inelastic deformation capacity primarily through brace buckling and yielding of the brace in tension.

18.2.2 *Analysis*

The following shall be satisfied in the analysis of SCBFs.

18.2.2.1 The required strength of braces shall be determined based on the analysis required by CED 39(22343). Further, the required strength of braces shall not exceed the design strength in axial compression as per IS 800 (*see* **17.1.2**).

18.2.2.2 The required strength of capacity-protected elements (columns, beams, struts, collectors, and connections) shall be taken as the larger force determined from the following analysis:

a) An analysis in which all braces are assumed to resist forces corresponding to their expected strength in compression or in tension;
b) An analysis in which all braces in tension are assumed to resist forces corresponding to their expected strength and all braces in compression are assumed to resist their expected post-buckling strength; and
c) For multi-tiered braced frames, analyses representing progressive yielding and buckling of the braces from weakest tier to strongest. Analyses shall consider both directions of frame loading.

The expected tensile strength (T_e) of braces in tension shall be taken as:

$$T_e = R_y f_y A_g$$

And, the expected compressive strength (P_e) of the braces shall be taken as:

$$P_e = R_y \gamma_{m0} P_d$$

where P_d is the design compressive strength as determined using IS 800 (*see* 1**7.1.2**).

The expected post-buckling strength of the braces in compression shall be taken as 0.2 times the expected compressive strength (P_e). And, the bracing connections shall be assumed to remain elastic.

18.2.3 *System Requirements*

The following system requirements shall be satisfied (see Fig. 51).

18.2.3.1 *Diagonal and X-braced frames*

Diagonal and X-braced frames are permitted to be used in SCBF.

18.2.3.2 *V- and inverted V-braced frame*

V- and inverted V-braced frames are permitted to be used in SCBF. In such systems, the beams that are intersected by braces away from beam-to-column connections shall satisfy the following requirements:

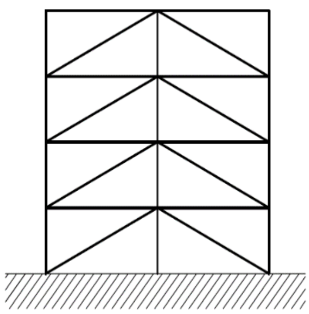
a) Beams shall be continuous between columns and adequately braced to prevent lateral torsional buckling; and
b) As a minimum, one set of lateral braces is required at the point of intersection of the V type (or inverted V-type) braced frames, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

18.2.3.3 *Continuity of load path*

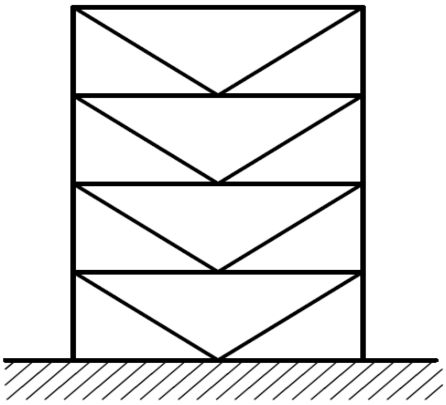
For the purpose of this standard, a line of braces is defined as a single line, or parallel lines with a plan offset of 10 percent or less of the building dimension perpendicular to the line of braces interconnected adequately through rigid diaphragm. A diaphragm shall be considered to be rigid if the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is less than 1.2 times the average displacement of the entire diaphragm.

18.2.3.4 *Lateral force distribution*

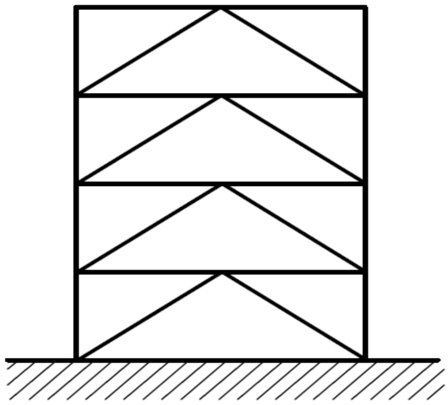
Braces shall be provided in alternate directions along each line of braces. Along any line of bracing, braces shall be provided such that for lateral loading in either direction, tension braces resist between 30 to 70 percent of the total horizontal load.



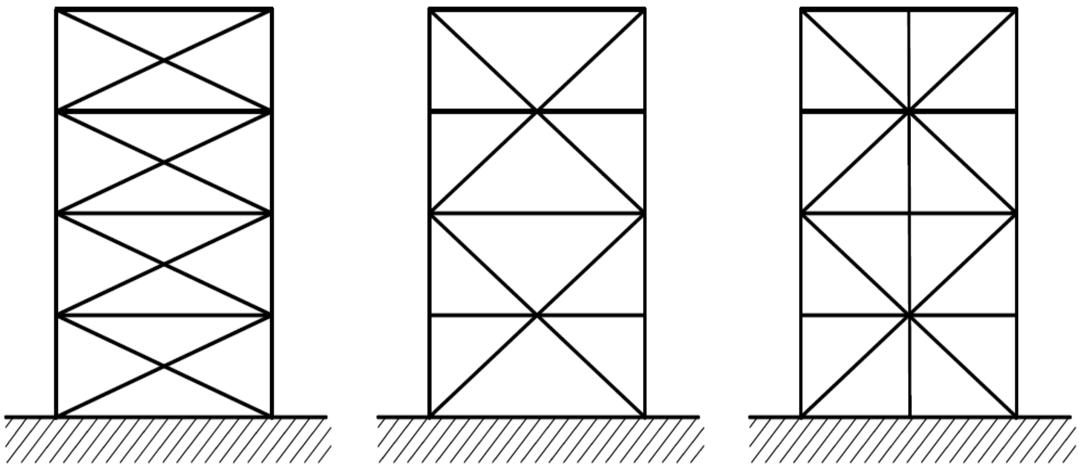
51A SINGLE DIAGONAL BRACED FRAME



51B V-BRACED FRAME



51C INVERTED V-BRACED FRAME



51D X-BRACED FRAMES

FIG. 51 CONCENTRICALLY BRACED FRAMES

18.2.3.5 *Multi-tiered braced frames*

A special concentrically braced frame is permitted to be configured as a multi-tiered braced frame (MT-SCBF) when the following requirements are satisfied:

a) Braces shall be used in opposing pairs at every tier level.
b) Struts shall satisfy the following requirements:
   1) Horizontal struts shall be provided at every tier level;
   2) Struts that are intersected by braces away from strut-to-column connections shall also meet the requirements stated in **18.2.2.2** (b). When brace buckling occurs out-of-plane, torsional moments arising from brace buckling shall be considered when verifying lateral bracing or minimum out-of-plane strength and stiffness requirements. The torsional moments shall correspond to 1.1$R_y M_p$ of the brace about the critical buckling axis, but need not exceed forces corresponding to the flexural resistance of the brace connection, where M_p is the plastic bending moment.
c) Columns shall satisfy the following requirements:
   1) Columns shall be torsionally braced at every strut-to-column connection location.
   2) Columns shall have sufficient strength to resist forces arising from brace buckling. These forces shall correspond to 1.1$R_y M_p$ of the brace about the

critical buckling axis, but need not exceed forces corresponding to the flexural resistance of the brace connections.

3) For all load combinations, columns subjected to axial compression shall be designed to resist bending moments due to second-order and geometric imperfection effects. As a minimum, imperfection effects are permitted to be represented by an out-of-plane horizontal notional load applied at every tier level and equal to 0.006 times the vertical load contributed by the compression brace intersecting the column at the tier level.
4) Lateral drift in each tier in a multi-tiered concentrically braced frame shall not exceed 0.4 percent of the tier height.

18.2.3.6 *K-braced frames*

K-braced frames shall not be used in SCBF.

18.2.4 *Member Requirements*

The requirements specified hereunder shall be satisfied by the component or member.

18.2.4.1 *Sections*

Columns, beams, braces and struts in multi-tiered concentrically braced in shall comply with the width-to-thickness requirements specified in Table 16.

18.2.4.2 *Braces*

Requirements specified in **17.5** shall apply.

18.2.4.3 *Protected zones*

The protected zone of SCBF shall satisfy **15.2**, and shall include the following:

a) For braces, the centre one-quarter of the brace length and a zone adjacent to each connection equal to the brace depth in the plane of buckling; and
b) Elements that connect braces to beams and columns.

18.2.4.4 *Beams*

In addition to satisfying requirements of **17.1**, beams shall be checked for axial load arising due to analysis case **18.2.2.2** (b).

18.2.4.5 *Beam-to-column connections*

Where a brace-gusset plate assembly connects to both members at a beam-to-column connection, the connection assembly shall be designed to resist moment equal to $1.1R_y f_{yb} Z_{pb}$. Also, the sum of the expected column flexural strengths shall exceed $1.1R_y f_{yb} Z_{pb}$.

This connection moment shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength earthquake load.

18.2.4.6 *Column splices*

Requirements specified in **17.2.5** shall apply.

In addition, column splices shall be designed to develop at least 50 percent of the plastic flexural strength, M_p, of the smaller connected member, and have shear strength greater than $\sum M_p / H_c$,

where

H_c = Clear height of the column between beam connections, and

$\sum M_p$ = Sum of the plastic flexural strengths, at the top and bottom ends of the column.

18.2.4.7 *Demand critical welds — see* **18.1.4.7**.

18.3 Eccentrically Braced Frame

Eccentrically braced frames (EBF) of structural steel shall be designed to satisfy the requirements given hereunder.

18.3.1 *Basis of Design*

These provisions are applicable to braced frames for which one end of each brace intersects a beam at an eccentricity from the intersection of the centerlines of the beam and an adjacent brace or column, forming a link that is subject to shear and flexure (*see* Fig. 52).

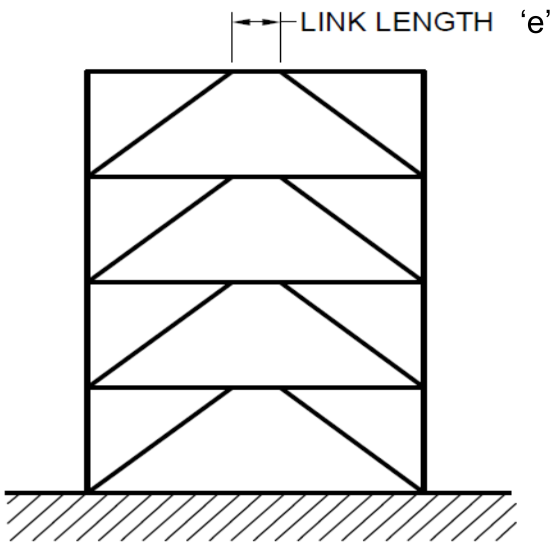


FIG. 52 ECCENTRICALLY BRACED FRAME

Eccentricities less than the beam depth are permitted in the brace connection away from the link if the resulting member and connection forces are addressed in the design and do not change the expected source of inelastic deformation capacity.

EBF designed in accordance with these provisions are expected to provide significant inelastic deformation capacity primarily through shear (or flexural) yielding in the links.

Links shall not be connected directly to columns.

8.3.2 *Analysis*

The requirements specified hereunder shall be satisfied in the analysis of EBFs.

8.3.2.1 The shear demand on the links shall be determined based on the analysis required by CED 39(22343).

8.3.2.2 The required strength of capacity-protected elements (columns, beams, diagonal braces and connections) shall be determined based on the expected overstrength capacity of the link to be taken as $1.1R_yS_h$ times the design strength of the link as per **17.6.2**, where S_h (a factor to account for strain hardening and strain rate) is equal to 1.25 for I-shaped links and 1.4 for box shaped links.

Exception: When the link and beam have the same section and is continuous,

a) the required strength of the beam outside the link shall be determined based on 0.9 times the expected link overstrength capacity; and
b) the design capacity of the beam shall be calculated based on the expected material yield stress.

18.3.3 *System Requirements*

The requirements specified hereunder shall be satisfied.

18.3.3.1 *Link rotation angle*

The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift. The link rotation angle shall not exceed 0.08 rad.

18.3.3.2 *Bracing of link*

Bracing shall be provided at both the top and bottom flanges of the link at the ends of the link for I-shaped sections. Such bracings shall have stiffness and strength as specified in **17.1.3.3**.

18.3.4 *Member Requirements*

The requirements specified hereunder shall be satisfied.

18.3.4.1 *Basic requirements*

Brace members, beams outside the links and columns shall satisfy width-to-thickness limitations specified in **16.2**. Apart from columns, the beams and braces in EBFs may be subjected to significant axial and bending forces; hence their design capacities shall be determined as for beam-column members as per IS 800.

18.3.4.2 *Links*

The link length *e* shall be considered to extend from brace connection to brace connection for centre links. In addition, requirements specified in **17.6** shall apply.

18.3.4.3 *Protected zones*

The protected zones of EBFs are the links. Use of shear studs on the links is prohibited.

18.3.4.4 *Beam-to-column connections*

Where a brace or gusset plate connects to both members at a beam-to-column connection, the connection assembly shall be designed to resist moment taken equal to $1.1R_y f_{yb} Z_{pb}$. Also, the sum of the expected column flexural strengths shall exceed $1.1R_y f_{yb} Z_{pb}$.
This connection moment shall be considered in combination with the required strength of the brace connection and beam connection, including the diaphragm collector forces determined using the overstrength earthquake load.

18.3.4.5 *Braces*

Braces in EBFs shall be designed not to yield in tension or buckle in compression corresponding to shear force in link taken equal to $1.21R_y$ times the design strength of connected link as per **17.6.2**, and shall satisfy the requirements of **15.2**.

18.3.4.6 *Brace connections*

When oversized holes are used, the required strength for the limit state of bolt slip need not exceed the earthquake load effect determined using the overstrength earthquake load. Connections of braces designed to resist a portion of the link end moment shall be designed as fully restrained.

18.3.4.7 *Column splices*

Requirements specified in **17.2.5** shall apply.

In addition, column splices shall be designed to develop at least 50 percent of the plastic flexural strength, M_p, of the smaller connected member, and have shear strength greater than $\sum M_p / H_c$,
where

H_c = Clear height of the column between beam connections,

$\sum M_p$ = Sum of the plastic flexural strengths, *at* the top and bottom ends of the column.

18.3.4.8 *Demand critical welds*

The following welds shall be designed as demand critical welds:

a) Groove welds at column splices;
b) Welds at column-to-base plate connections, except when

1) Column plastic hinge at, or near the base plate is precluded by conditions of restraint, or
2) There is no net tension under load combinations including the overstrength earthquake load;

c) Welds in beam–column connections;
d) Connections of braces designed to resist a portion of the link end moment;
e) In built-up beams, welds within the link connecting the webs to the flanges.

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ANNEX A
(*Clause* 2)

LIST OF CROSS REFERRED INDIAN STANDARDS

IS Number	*Title*
CED 39 (22343)	Draft Indian Standard Criteria for earthquake resistant design of structures Part 1 General Provisions
CED 39 (22345)	Draft Indian Standard Criteria for earthquake resistant design of structures Part 2 Buildings
456 : 2000	Plain and Reinforced Concrete (*fourth revision*)
800 : 2007	General Construction in Steel (*second revision*)
875 (Part 1 : 1987) (Part 2 : 1987) (Part 3 : 2015) (Part 4 : 1987) (Part 5 : 1987)	Design Loads (other than Earthquake) for Buildings and Structures: : Dead loads – Unit weights of building material and stored materials (*Second Revision*) : Imposed Loads (*Second Revision*) : Wind Loads (*Third Revision)* : Snow Loads (*Second Revision)* : Special Loads and Load Combinations (*second revision*)
1343 : 2017	Prestressed Concrete (*second revision*)
1905 : 1987	Structural Use of Unreinforced Masonry (*third revision*)
2062 : 2011	Hot Rolled Medium and High Tensile Structural Steel – Specification (*seventh revision*)
13920 (Part 1) : 2023	Specifications for Earthquake-Resistant Design and Detailing of Structures (*second revision*) Part 1 General Provisions
13935 (Part 1) : 2023 (Part 2) : 2023	Principles for Earthquake Safety Assessment and Retrofit of Structures (*second revision*) : General Provisions : Buildings
16172 : 2014	Reinforcement Couplers for Mechanical Splices of Bars in Concrete – Specification
1786 : 2008	High Strength Deformed Steel Bars and Wires for Concrete Reinforcement (*fourth revision*)
16172 : 2014	Reinforcement Couplers for Mechanical Splices of Bars in Concrete – Specification
814 : 2004	Covered Electrodes for Manual Metal Arc Welding of Carbon and Carbon Manganese Steel – Specification (*sixth revision*)
816 : 1969	Use of Metal Arc Welding for General Construction in Mild Steel – *Code of Practice* (*first revision*)
1363 (Part 1) : 2019	Hexagon Head Bolts, Screws and Nuts of Product Grade C (*fifth revision*)

(Part 2) : 2018 (Part 3) : 2018	: Hexagon Head Bolts (Size Range M5 to M64) : Hexagon Head Screws (Size Range M5 to M64) : Liquid Retaining Nuts (Size Range M5 to M64)
1364 (Part 1) : 2018 (Part 2) : 2018 (Part 3) : 2018 (Part 4) : 2003 (Part 5) : 2002 (Part 6) : 2018	Hexagon Head Bolts, Screws and Nuts of Product Grade A and B : Hexagon Head Bolts (Size Range M1.6 to M64) (*fifth revision*) : Hexagon Head Screws (Size Range M1.6 to M64) (*fifth revision*) : Hexagon Head Nuts (Size Range M1.6 to M64) (*fifth revision*) : Hexagon Thin Nuts (Chamfered) (Size Range M1.6 to M64) (*fourth revision*) : Hexagon Thin Nuts - Product Grade B (Unchamfered) (Size Range M1.6 to M64) (*fourth revision*) : Hexagon Nuts (Style 2) (Size Range M1.6 to M64) (*first revision*)
3640 : 1982	Hexagon Fit Bolts – *Specification* (*first revision*)
3757 : 1985	High Strength Structural Bolts – Specification (*second revision*)
4000 : 1992	High Strength Bolt in Steel Structures – *Code of Practice* (*first revision*)
5624 : 2021	Foundation Bolts – *Specification* (*second revision*)
6610 : 1972	Heavy Washers for Steel Structures – Specification
6639 : 1972	Hexagon Bolts for Steel Structures – Specification
6649 : 1985	Hardened and Tempered Washers for High Strength Structural Bolts and Nuts – *Specification* (*first revision*)
12778 : 2004	Hot Rolled Parallel Flange Steel Sections for Beams, Columns and Bearing Piles – *Dimensions and Section Properties* (*first revision*)

ANNEX B
(*Foreword*)

(Committee Composition will be added after finalization)

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