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

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व्यापक परिचालन मसौदा

हमारा संदर्भ: सीईडी 46/टी-9 **31 मार्च 2025**

तकनीकी समिति: भारत की राष्ट्रीय भवन निर्माण विषय समिति, सीईडी 46

प्राप्तकर्ता :

1. सिविल अभियांत्रिकी विभाग परिषद, सीईडीसी के सभी सदस्य
2. राष्ट्रीय भवन निर्माण संहिता विषय समिति, सीईडी 46 के सभी सदस्य
3. सीईडी 46 की उपसीमितयों और अन्य कार्यदल के सभी सदस्य
4. रुचि रखने वाले अन्य निकाय।

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

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भारतीय मानक ब्यूरो
(उपभोक्ता मामले, खाद्य एवं सार्वजनिक वितरण मंत्रालय, भारत सरकार)
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Our Reference: CED 46/T-9 **31 March 2025**

National Building Code of India Sectional Committee, CED 46

ADDRESSED TO:

1. All Members of Civil Engineering Division Council, CEDC
2. All Members of the National Building Code Sectional Committee, CED 46
3. All Members of Subcommittees, Panels and Working Groups under CED 46
4. All other interests

Dear Sir/Madam,

Please find enclosed the following draft:

Doc No.	Title
CED 46 (26653) WC	**National Building Code of India Part 6 Structural Design Section 4 Masonry** [Fourth Revision of SP 7 (Part 6 Section 4)] (ICS No. 01.120: 91.040.01)

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: 02 May 2025

Comments if any, may please be made in the enclosed format and emailed at **ced46@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 of editorial nature, kindly permit us to presume your approval for the above document as finalized. However, in case comments, technical in nature are received, then it may be finalized either in consultation with the Chairman, Sectional Committee or referred to the Sectional Committee for further necessary action if so desired by the Chairman, Sectional Committee.

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

Thanking you,

Yours faithfully,

Sd/-
(Dwaipayan Bhadra)
Scientist 'E' / Director and 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/figure, 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 ced46@bis.gov.in shall be appreciated**.

Doc. No.: CED 46 (26653) WC **BIS Letter Ref**: CED 46/T-9

Title: National Building Code of India Part 6 Structural Design Section 4 Masonry [Fourth Revision of SP 7 (Part 6 Section 4)] (ICS No.01.120:91.040.01)

Last date of comments: **02 May 2025**

Name of the Commentator/ Organization: ________________________________

Clause/ Para/ Table/ Figure No. commented	Comments/Modified Wordings	Justification of Proposed Change

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

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Draft Indian Standard

National Building Code of India

PART 6 STRUCTURAL DESIGN

Section 4 Masonry

[Fourth Revision of SP 7 (Part 6/Section 4)]

(ICS No. 01.120: 91.040.01)

National Building Code Sectional Committee, CED 46	**Last Date for Comments: 02 May 2025**

C O N T E N T S

National Building Code Sectional Committee, CED 46

FOREWORD

This Code (Part 6/Section 4) covers the structural design of unreinforced and reinforced masonry elements in buildings. Provisions on reinforced brick and reinforced brick concrete floors and roofs have also been dealt with.

This Section was first published in 1970 and subsequently revised in 1983, 2005 and 2016. The first revision of this Section was modified in 1987 through Amendment No. 2 to bring this Section in line with the latest revised masonry code. In the amendment, certain provisions were updated following the revision of IS 1905 'Code of practice for structural use of unreinforced masonry' on which the earlier version was based. In the amendment, requirements of masonry element for stability were modified; in the design of free standing wall, provisions were made for taking advantage of the tensile resistance in masonry under certain conditions; provision regarding effective height of masonry wall between openings was modified; method of working out effective height of wall with a membrane type DPC was modified; the criteria for working out effective length of wall having openings was modified; some general guidelines for dealing with concentrated loads for design of walls were included; and provision of cutting and chases in walls were amplified. The Section was thereafter revised in 2005.

In the second revision of this Section in 2005, the provisions of special considerations in earthquake zones were aligned in line with the revised Indian Standard, IS 4326 : 1993 'Code of practice for earthquake resistant design and construction of buildings'; a new clause covering guidelines for improving earthquake resistance of low strength masonry buildings was added; and reference to design of reinforced brick and reinforced brick concrete floors and roofs was included.

In the third revision of this Section in 2016, the detailed provisions relating to reinforced masonry and durability criteria for selection of masonry mortars had been included. Also, the number of storeys of masonry construction had been limited to 4, in line with the seismic design standards. A new masonry wall construction using rat-trap bond had been included.

As a result of experience gained in the implementation of this Section and feedback received, as well as in view of formulation of new varieties of cement and new types of masonry blocks, a need to revise this Section was again felt. This Section has, therefore, been revised to incorporate the following significant changes:

a) New masonry blocks have been included as masonry unit.
b) The admissibility of different structural systems in the earthquake zones is clarified. The following structural systems are admissible to construct earthquake resistant masonry buildings:

1) Masonry walls with prescriptive bands (MWB),
2) masonry walls with prescriptive bands and vertical reinforcement (MWBR),
3) Confined masonry walls (CMW), and
4) Reinforced masonry walls (RMW).

c) The clause on confined masonry has been updated.
d) The clause on rat trap masonry has been updated.
e) Reference to all the concerned Indian Standards has been updated.

Structural design requirements of this section are largely based on following Indian Standards:

a) IS 1905:1987 'Code of practice for structural use of unreinforced masonry (*third revision*)',
b) IS 10440:1983 Code of practice for construction of RB and RBC floors and roofs,
c) IS 13828 : 1993 'Improving earthquake resistance of low strength masonry buildings – Guidelines' and
d) IS 13920 (Part 5):20xx Earthquake resistant design and detailing of structures — Code of practice Part 5 Buildings (*Second revision*) (*under preparation*)

Buildings with MWB and MWBR systems shall not be of height more than 15 m subject to a maximum of four storeys, measured from the base of the buildings to the roof slab or ridge level.

Until further research material is available on rat-trap bond masonry, its usage is proposed to be limited up to two storeys.

A reference to SP 20: 1991 'Handbook on masonry design and construction (*first revision*)', may be useful. In the formulation of this Section, inputs were also derived from the IITK-GSDMA's publication, 'Guidelines for structural use of reinforced masonry'.

All standards, whether given herein above or cross-referred to in the main text of this section, are subject to revision. The parties to agreement based on this section are encouraged to investigate the possibility of applying the most recent editions of the standards.

For the purpose of deciding whether a particular requirement of this Section 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 Section.

Draft Indian Standard

National Building Code of India

PART 6 STRUCTURAL DESIGN

Section 4 Masonry

[Fourth Revision of SP 7 (Part 6/Section 4)]

(ICS No. 01.120: 91.040.01)

National Building Code Sectional Committee, CED 46 — **Last Date for Comments: 02 May 2025**

1 SCOPE

1.1 This Code (Part 6/Section 4) covers the structural design aspects of unreinforced load bearing and non-load bearing walls, constructed with masonry units permitted in accordance with this Section.

1.2 This Section also deals with the selection of materials, special features of design and construction for masonry construction using rectangular masonry units. It also covers guidelines regarding earthquake resistant buildings constructed using masonry of low strength.

1.3 This Section also covers provisions for design of reinforced brick and reinforced brick concrete floors and roofs.

1.4 This Section also gives the recommendations for structural design aspects of reinforced load bearing and non-load bearing walls, constructed with different types of bricks and blocks.

1.5 The recommendations of the Section do not apply to walls constructed in mud mortars.

2 TERMINOLOGY

2.0 For the purpose of this Section, the following definitions shall apply.

2.1 Bed Block – A block bedded on a wall, column or pier to disperse a concentrated load on a masonry element.

2.2 Bond – Arrangement of masonry units in successive courses to tie the masonry together both longitudinally and transversely; the arrangement is usually worked out to ensure that no vertical joint of one course is exactly over the one in the next course above or below it, and there is maximum possible amount of lap.

2.3 Column, Pier and Buttress

a) *Column* – An isolated vertical load bearing member, width of which does not exceed four times the thickness.
b) *Pier* – A thickened section forming integral part of a wall placed at intervals along the wall, to increase the stiffness of the wall or to carry a vertical concentrated load. Thickness of a pier is the overall thickness including the thickness of the wall or, when bonded into a leaf of a cavity wall, the thickness obtained by treating that leaf as an independent wall (*see* Fig. 1)

c) *Buttress* – A pier of masonry built as an integral part of wall and projecting from either or both surfaces, decreasing in cross-sectional area from base to top.

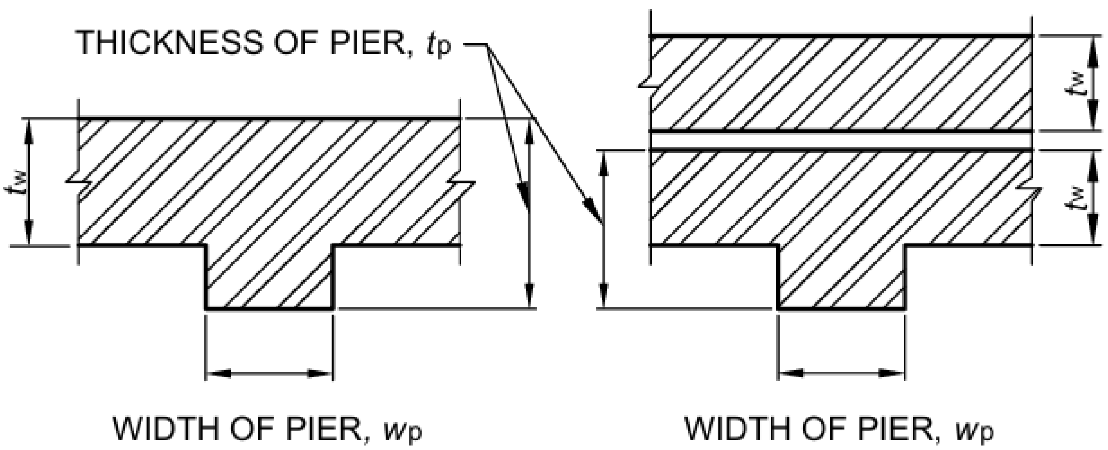


Fig. 1 DEFINITION OF PIER

2.4 Cross-Sectional Area of Masonry Unit – Net cross-sectional area of a masonry unit shall be taken as the gross cross-sectional area minus the area of cellular space. Gross cross-sectional area of cored units shall be determined to the outside of the coring but cross-sectional area of grooves shall not be deducted from the gross cross-sectional area to obtain the net cross-sectional area.

2.5 Curtain Wall – A non-load bearing wall subject to lateral loads. It may be laterally supported by vertical or horizontal structural members, where necessary (*see* Fig. 2).

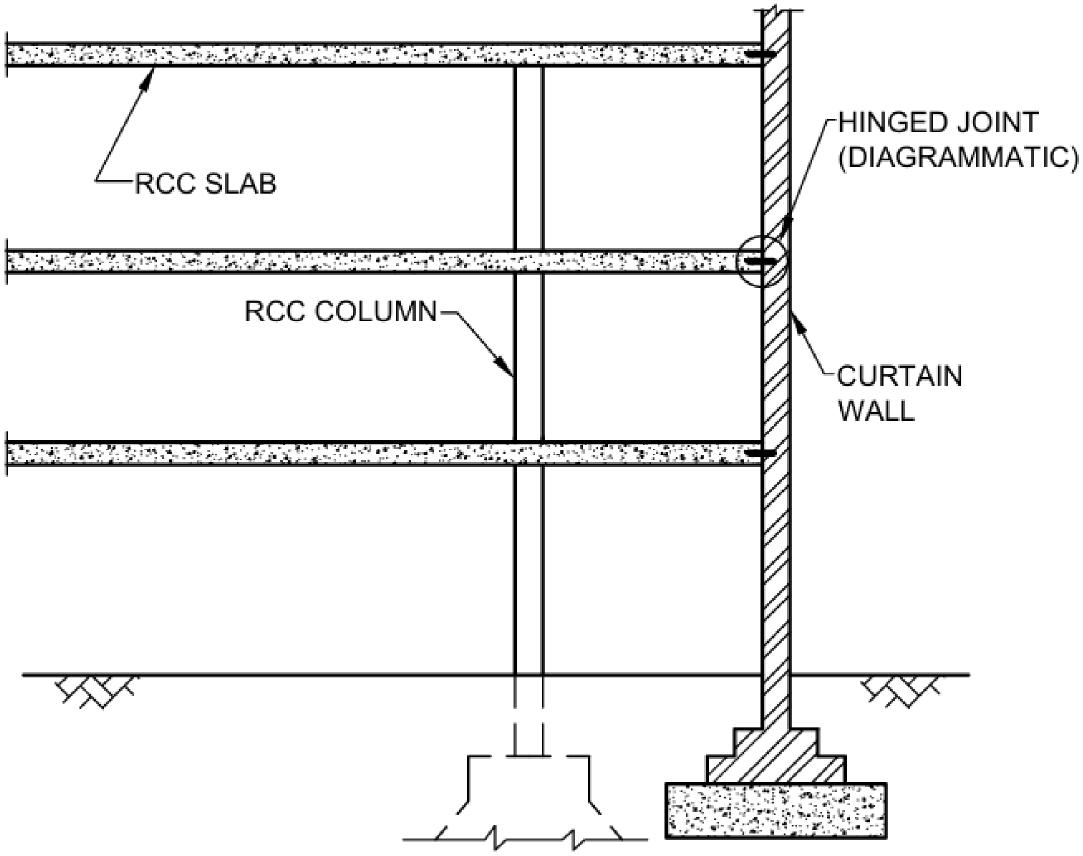
2.6 Effective Height – The height of a wall or column, to be considered for calculating slenderness ratio.

2.7 Effective Length – The length of a wall to be considered for calculating slenderness ratio.

2.8 Effective Thickness – The thickness of a wall or column to be considered for calculating slenderness ratio.

2.9 Hollow Unit – A masonry unit of which net cross-sectional area in any plane parallel to the bearing surface is less than 75 percent of its gross cross-sectional area measured in the same plane.

2.10 Grout – Mortar of pourable consistency.



NOTE – The figure is only illustrative to explain about the term curtain wall and is not intended to be an exhaustive provision.

FIG. 2 A TYPICAL MASONRY CURTAIN WALL

2.11 Joint – A junction of masonry units, which can be classified as:

a) Bed Joint – A horizontal mortar joint upon which masonry units are laid.
b) Cross Joint – A vertical joint, normal to the face of the wall.
c) Wall Joint – A vertical joint parallel to the face of the wall.

2.12 Leaf – Inner or outer section of a cavity wall.

2.13 Lateral Support – A support which enables a masonry element to resist lateral load and/or restrains lateral deflection of a masonry element at the point of support.

2.14 Load Bearing Wall – A wall designed to carry an imposed vertical load in addition to its own weight, together with any lateral load.

2.15 Masonry – An assemblage of masonry units properly bonded together with mortar.

2.16 Masonry Unit – Individual units which are bonded together with the help of mortar to form a masonry element such as wall, column, pier, buttress, etc.

2.17 Partition Wall – An interior non-load bearing wall, one storey or part storey in height.

2.18 Panel Wall – An exterior non-load bearing wall in framed construction, wholly supported at each storey but subjected to lateral loads.

2.19 Shear Wall – A wall designed to carry horizontal forces acting in its plane with or without vertical imposed loads.

2.20 Slenderness Ratio – Ratio of effective height or effective length to effective thickness of a masonry element.

2.21 Types of Walls

a) *Cavity Wall* – A wall comprising two leaves, each leaf being built of masonry units and separated by a cavity and tied together with metal ties or bonding units to ensure that the two leaves act as one structural unit, the space between the leaves being either left as continuous cavity or filled with a non-load bearing insulating and water-proofing material.

b) *Faced Wall* – A wall in which facing and backing of two different materials are bonded together to ensure common action under load (*see* Fig. 3).

NOTE – To ensure monolithic action in faced walls, shear strength between the facing and the backing shall be provided by toothing, bonding or other means.

c) *Veneered Wall* – A wall in which the facing is attached to the backing but not so bonded as to result in a common action under load.

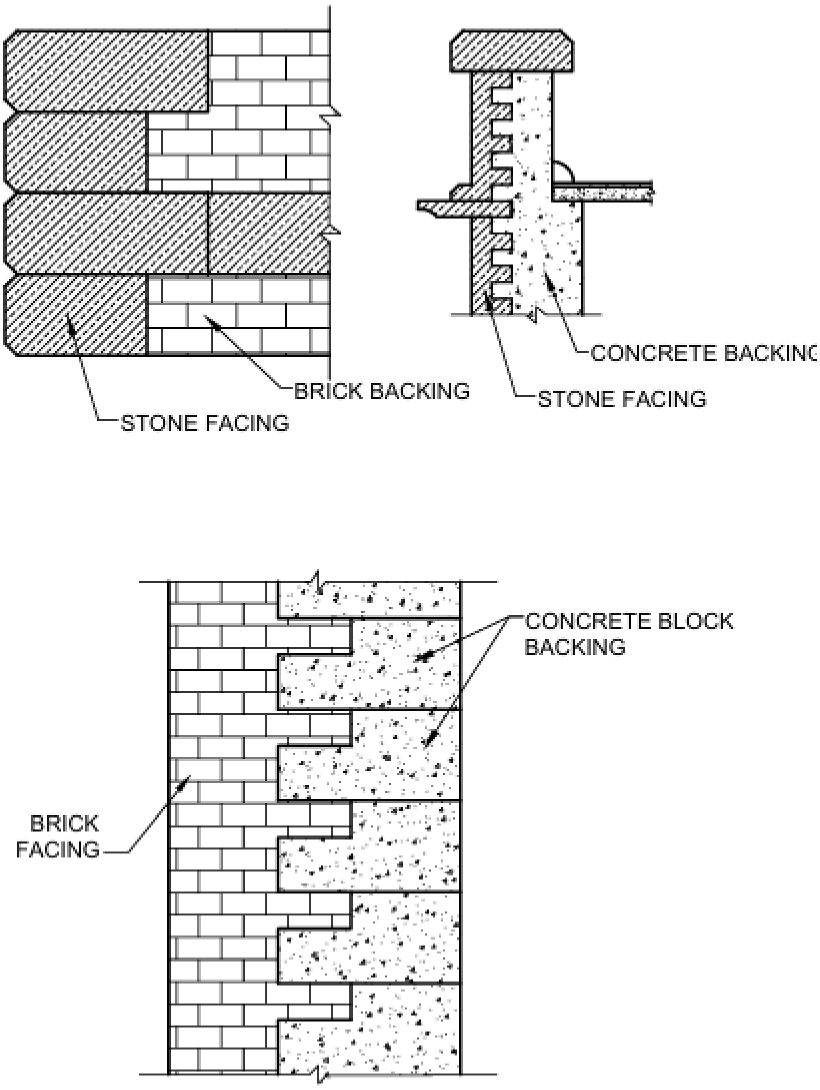
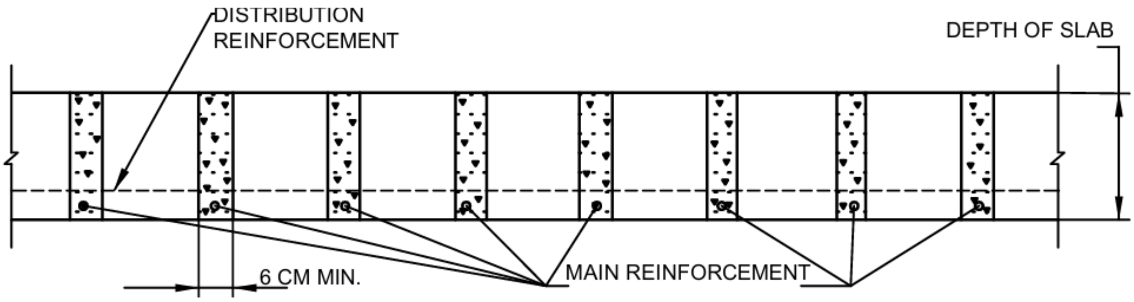


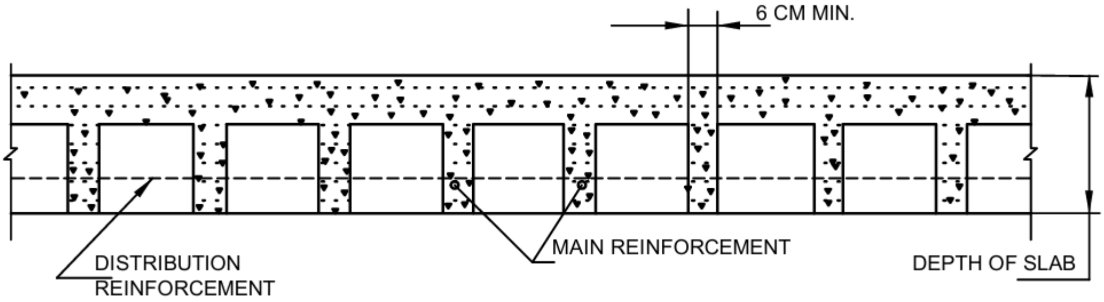
Fig. 3 TYPICAL FACED WALL

2.22 Reinforced Brick Slab *(RB Slab)* – Reinforced brick slab is particularly same as reinforced concrete slab in all its essential features except that brickwork is substituted partly for cement concrete. *See* Fig. 4A for details.

2.23 Reinforced Brick Concrete Slab *(RBC Slab)* – These are reinforced brick slab using concrete in the joints and on the top of bricks. *See* Fig. 4B for details.



4A Cross-Section of RB Slab



4B Cross-Section of RBC Slab

FIG. 4 CROSS-SECTIONS OF RB AND RBC SLABS

3 MATERIALS

3.1 General

The materials used in masonry construction shall be in accordance with Part 5 'Building Materials' of the Code. Storage of materials shall be in accordance with good practice [6-4(1)].

3.2 Masonry Units

Masonry units used in construction shall conform to accepted standards [6-4(2)].

3.2.1 Masonry units may be of the following types:

a) Common burnt clay building bricks,
b) Burnt clay fly ash building bricks,
c) Pulverized fuel ash lime bricks,
d) Pulverized fuel ash cement bricks
e) Stones (in regular sized units),
f) Sand-lime bricks,
g) Concrete blocks (solid and hollow),
h) Lime based blocks,
j) Burnt clay hollow blocks,
k) Gypsum partition blocks,
m) Autoclaved cellular concrete blocks,
n) Stabilized soil blocks, and
p) Concrete stone masonry blocks.

NOTES

1 Gypsum partition blocks are used only for construction of non-load bearing partition walls.

2 Use of other masonry units such as precast stone blocks, fly-ash-lime-gypsum bricks, stabilized soil blocks and other bricks/blocks not covered by the above specifications may also be permitted based on test results.

3.2.2 Masonry units that have been previously used shall not be reused in brickwork or blockwork construction, unless they have been thoroughly cleaned and conform to this Section for similar new masonry units.

3.3 Mortar

Mortar for masonry shall conform to accepted standard [6-4(3)].

3.3.1 Minimum compressive strength for different grades of mortar on the basis of mix proportions as per **3.3.2** are given in Table 1.

Table 1 Compressive Strength for Different Grades of Mortar
(*Clause* 3.3.1)

Sl No.	Grade of Mortar	Minimum Compressive Strength at 28 Days N/mm^2
(1)	(2)	(3)
i)	H1	10
ii)	H2	6.0
iii)	M1	5.0
iv)	M2	3.0
v)	M3	1.5
vi)	L1	0.7
vii)	L2	0.5

3.3.2 Mix proportions and compressive strengths of some of the commonly used mortars are given in Table 2.

Table 2 Mix Proportions and Strength of Mortars for Masonry

(*Clause* 3.3.2)

Sl No.	Mix Proportions (By Mass)					Minimum Compressive Strength at 28 Days in N/mm^2 (with Cement Compressive Strength at 28 Days, in N/mm^2, between 33 and 43)	Minimum Compressive Strength of Cement : Sand Mortar at 28 Days in N/mm^2 (with Cement Compressive Strength at 28 Days, in N/mm^2 ≥ 43)
	Cement	Lime	Lime - Pozzolana Mixture	Pozzolana	Sand		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1	1	¼ C or B	0	0	3	10	-
2(a)	1	¼ C or B	0	0	4	7.5	-
2(b)	1	½ C or B	0	0	4 ½	6	-
2(c)	1	0	0	0	4	6	7
3(a)	1	0	0	0	5	5	6
3(b)	1	1 C or B	0	0	6	3	-
3(c)	0	0	1(LP-40)	0	1 ½	3	-
4(a)	1	0	0	0	6	3	4
4(b)	1	2B	0	0	9	2	-
4(c)	0	1A	0	0	2	2	-
4(d)	0	1B	0	1	1	2	-
4(e)	0	1 C or B	0	2	0	2	-
4(f)	0	0	1(LP-40)	0	1 ¾	2	-
5(a)	1	0	0	0	7	2	3
5(b)	1	3B	0	0	12	1.5	-
5(c)	0	1A	0	0	3	1.5	-
5(d)	0	1B	0	2	1	1.5	-
5(e)	0	1 C or B	0	3	0	1.5	-
5(f)	0	0	1(LP-40)	0	2	1.5	-
6(a)	1	0	0	0	8	1	2
6(b)	0	1B	0	1	2	0.7	-
6(c)	0	1 C or B	0	2	1	0.7	-
6(d)	0	0	1(LP-40)	0	2 ¼	0.7	-
6(e)	0	0	1(LP-20)	0	1 ½	0.7	-
7(a)	0	1B	0	0	3	0.5	-
7(b)	0	1 C or B	0	1	2	0.5	-
7(c)	0	0	1(LP-7)	0	1 ½	0.5	-

NOTES

1 Sand for making mortar should be well graded. In case sand is not well graded, its proportions shall be reduced in order to achieve the minimum specified strength.

2 For mixes in Sl No. 1 and Sl No. 2, use of lime is not essential from consideration of strength as it does not result in increase in strength. However, its use is highly recommended since if improves workability.

3 For mixes in Sl No. 3(a), 4(a), 5(a) and 6(a) either lime C or B to the extent of ¼ part of cement (by mass) or some plasticizer should be added for improving workability.

4 For mixes in Sl No. 4(b) and 5(b), lime and sand should first be ground in mortar mill and then cement added to coarse stuff.

5 It is essential that mixes in SI No. 4(c), 4(d), 5(d), 5(e), 6(b), 6(c), 7(a) and 7(b) are prepared by grinding in a mortar mill.

6 A, B and C denote eminently hydraulic lime, semi-hydraulic lime and fat lime respectively, as specified in appropriate standards listed in Part 5 'Building Materials' of the Code.

3.4 Durability Criteria for Selection of Masonry Mortars

3.4.1 The selection of masonry mortars from durability considerations shall cover both the loading and exposure conditions of the masonry. The requirements for masonry mortar shall generally be as given in **3.4.2** to **3.4.6** (*see also* Table 1).

3.4.2 In the case of masonry exposed frequently to rains and where there is further protection by way of plastering or rendering or other finishes, the grade of mortar shall not be less than M3 but shall preferably be of grade M2. Where no protection is provided, the grade of mortar for external walls shall not be less than M2.

3.4.3 In the case of load bearing internal walls, the grade of mortar shall preferably be M3 or more for high durability but in no case less than L1.

3.4.4 In the case of masonry in foundations laid below damp-proof course, the grades of mortar for use in masonry shall be as specified below:

a) Where soil has little moisture, masonry mortar of grade not less M3 shall be used.

b) Where soil is very damp, masonry mortar of grade preferably M2 or more shall be used. But in no case shall the grade of mortar be less than L1 and M3; and

c) Where soil is saturated with water, masonry mortar of grade M1 shall be used but in no case shall the grade of mortar be less than M 2.

3.4.5 For masonry in buildings subject to vibration of machinery, the grade of mortar shall not be less than M1.

3.4.6 For parapets, where the height is greater than three times the thickness, the grade of masonry mortar used shall not be less than M1. In the case of low parapets, the grade of mortar shall be the same as that used in the wall masonry below.

3.4.7 The grade of mortar for bedding joints in masonry with large concrete blocks shall not be less than M1.

NOTE — For earthquake design consideration, recommended mortar mixes shall be as per Table 14.

4 DESIGN CONSIDERATIONS

4.1 General

Masonry structures gain stability from the support offered by cross walls, floors, roof and other elements such as piers and buttresses. Load bearing walls are structurally more efficient when the load is uniformly distributed and the structure is so planned that eccentricity of loading on the members is as small as possible. Avoidance of eccentric loading by providing adequate bearing of floor/roof on the walls providing adequate stiffness in slabs and avoiding fixity at the supports, etc, is especially important in load bearing walls in multistorey structures. These matters should receive careful consideration during the planning stage of masonry structures.

4.2 Lateral Supports and Stability

4.2.1 *Lateral Supports*

Lateral supports for a masonry element such as load bearing wall or column are intended,

a) to limit slenderness of a masonry element so as to prevent or reduce possibility of buckling of the member due to vertical loads; and
b) to resist horizontal components of forces so as to ensure stability of a structure against overturning.

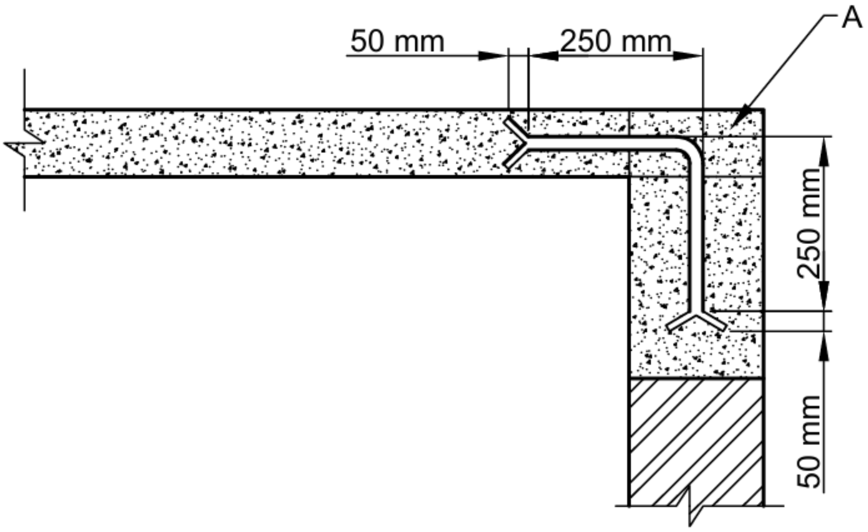
4.2.1.1 Lateral support may be in the vertical or horizontal direction, the former consisting of floor/roof bearing on the wall or properly anchored to the same and latter consisting of cross walls, piers or buttresses.

4.2.1.2 Requirements of **4.2.1**(a) from consideration of slenderness may be deemed to have been met with, if:

a) In case of a wall, where slenderness ratio is based on effective height, any of the following constructions are provided:

   1) RCC floor/roof slab (or beams and slab) irrespective of the direction of span, bears on the supported wall as well as cross walls, to the extent of at least 90 mm;

   2) RCC floor/roof slab not bearing on the supported wall or cross wall is anchored to it with non-corrodible metal ties of 600 mm length and of section not less than 6 mm x 30 mm, and at intervals not exceeding 2 m, as shown in Fig. 5; and



A = Cement concrete only at places where anchors are provided
(200 mm in width in the direction perpendicular to the plane of paper)

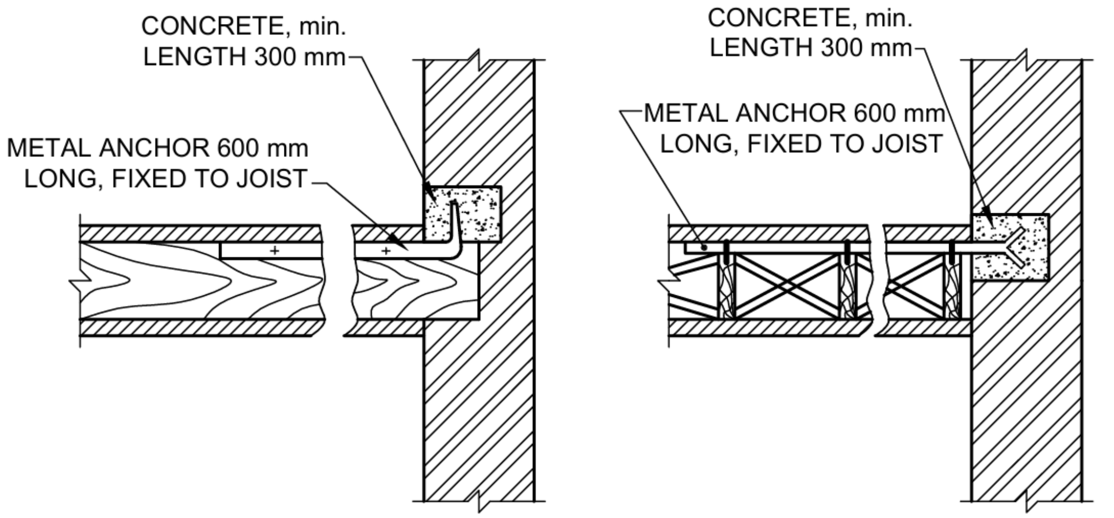
FIG. 5 ANCHORING OF RCC SLAB WITH MASONRY WALL
(WHEN SLAB DOES NOT BEAR ON WALL)

3) Timber floor/roof, anchored by non-corrodible metal ties of length 600 mm and of minimum section 6 mm x 30 mm, securely fastened to joists and built into walls as shown in Fig. 6 and Fig. 7. The anchors shall be provided in the direction of span of timber joists as well as in its perpendicular direction, at intervals of not more than 2 m in buildings up to two storeys and 1.25 m for buildings more than two storeys in height.

NOTES

1 In case precast RCC units are used for floors and roofs, it is necessary to interconnect them and suitably anchor them to the cross walls so that they can transfer lateral forces to the cross walls.

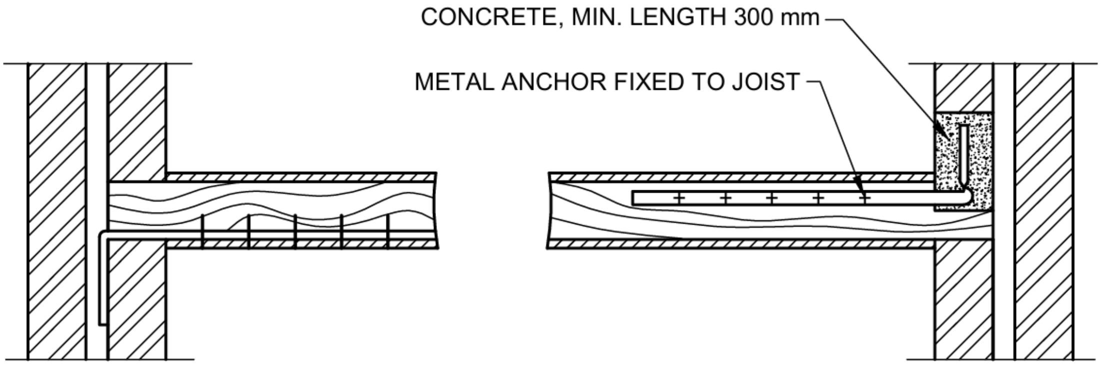
2 In case of small houses of conventional designs, not exceeding two storeys in height, stiffening effect of partitions and cross walls is such that metal anchors are normally not necessary in case of timber floor/roof and precast RCC floor/roof units.



6A JOISTS AT RIGHT ANGLE TO WALL

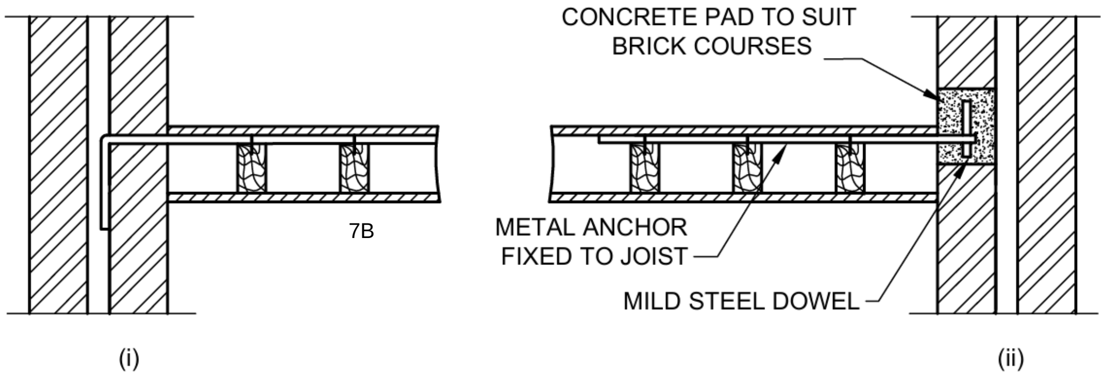
6B JOISTS PARALLEL TO WALL

FIG. 6 TYPICAL DETAILS FOR ANCHORAGE OF SOLID WALLS



(i) (ii)

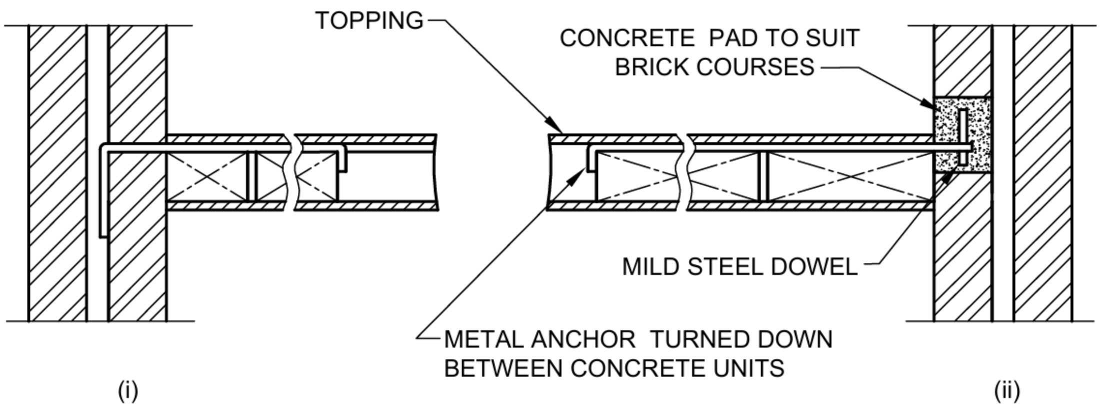
7A TIMBER JOISTS AT RIGHT ANGLES TO WALL



(i) (ii)

7B TIMBER JOISTS PARALLEL TO WALL

7C



7C PRECAST CONCRETE FLOOR UNITS PARALLEL TO WALL

FIG. 7 TYPICAL DETAILS FOR ANCHORAGE OF CAVITY WALLS

b) in case of a wall, when slenderness ratio is based on its effective length; a cross wall/pier/buttress of thickness equal to or more than half the thickness of the supported wall or 90 mm, whichever is more, and length equal to or more than one fifth of the height of wall, is built at right angle to the wall (*see* Fig. 8) and bonded to it according to provision of **4.2.2.2** (d);

c) in case of a column, an RCC or timber beam/RS joist/roof truss, is supported on the column. In this case, the column will not be deemed to be laterally supported in the direction at right angle to it; and

d) in case of a column, an RCC beam forming a part of beam and slab construction is supported on the column, and slab adequately bears on stiffening walls. This construction will provide lateral support to the column, in the direction of both horizontal axes.

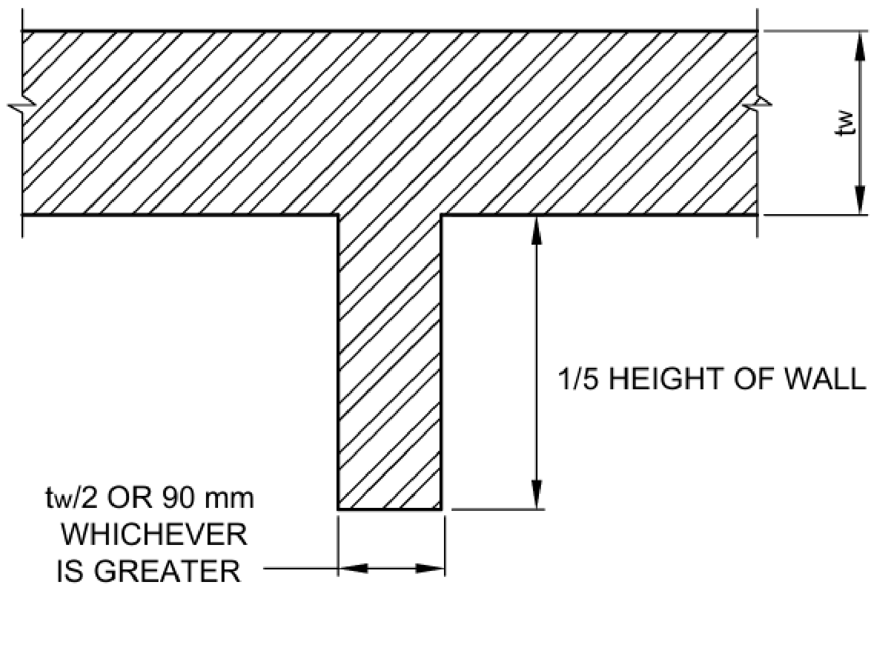


FIG. 8 MINIMUM DIMENSION FOR MASONRY WALL OR BUTTRESS EFFECTIVE LATERAL SUPPORT

4.2.2 *Stability*

A wall or column subjected to vertical and lateral loads may be considered to be provided with adequate lateral support from consideration of stability, if the construction providing the support is capable of resisting some of the following forces:

a) Simple static reactions at the point of lateral support to all the lateral loads; plus
b) 2.5 percent of the total vertical load that the wall or column is designed to carry at the point of lateral support.

4.2.2.1 For the purpose specified in **4.2.2**, if the lateral supports are in the vertical direction, these should meet the requirements given in **4.2.1.2** (a) and should also be capable of acting as horizontal girders duly anchored to the cross wall so as to transmit the lateral loads to the foundations without exceeding the permissible stresses in the cross walls.

4.2.2.2 In case of load bearing buildings up to four storeys, stability requirements of **4.2.2** may be deemed to have been met with, if:

a) height to width ratio of building does not exceed 2;
b) cross walls acting as stiffening walls continuous from outer wall to outer wall or outer wall to a load bearing inner wall, and of thickness and spacings as given in Table 3 are provided. If stiffening wall or walls that are in a line, are interrupted by openings, length of solid wall or walls in the zone of the wall that is to be stiffened shall be at least one-fifth of height of the opening as shown in Fig. 9;
c) floors and roof either bear on cross walls or are anchored to those walls as in **4.2.1.2** such that all lateral loads are safely transmitted to those walls and through them to the foundation; and
d) cross walls are built jointly with the bearing walls and are jointly mortared, or the two interconnected by toothing. Alternatively, cross walls may be anchored to walls to be supported by ties of non-corrodible metal of minimum section 6 mm x 35 mm and length 600 mm with ends bend at least 50 mm; maximum vertical spacing of ties being 1.2 m (*see* Fig. 10).

Table 3 Thickness and Spacing of Stiffening Walls
[*Clause* 4.2.2.2(b)]

Sl No.	Thickness of Load Bearing Wall to be Stiffened	Height [1] of Storey Not to Exceed	Stiffening Wall [1]		
			Thickness not Less Than		Maximum Spacing
			1 to 3 Storeys	4 Storeys	
	mm	m	mm	mm	m
(1)	(2)	(3)	(4)	(5)	(6)
i)	100	3.2	100	—	4.5
ii)	200	3.2	100	200	6.0
iii)	300	3.4	100	200	8.0
iv)	Above 300	5.0	100	200	8.0

[1] Storey height and maximum spacings as given are centre-to-centre dimensions.

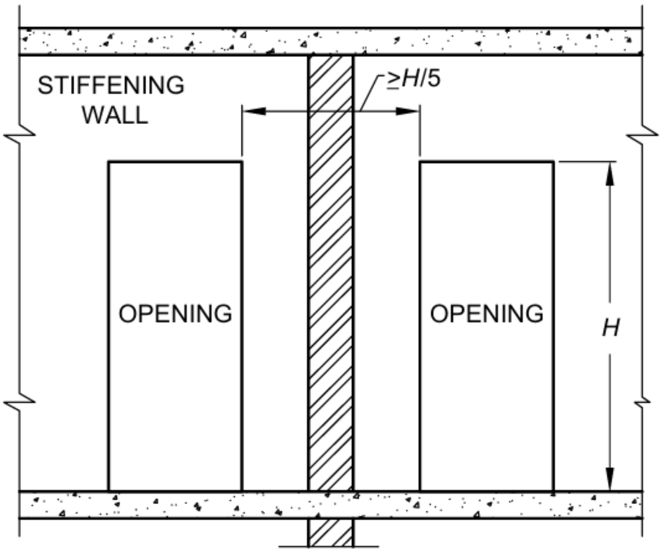


FIG. 9 OPENING IN STIFFENING WALL

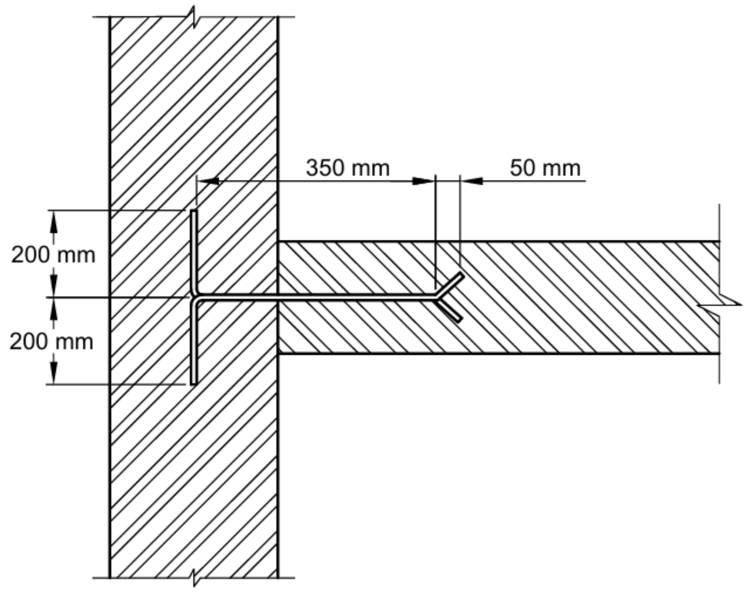


FIG. 10 ANCHORING OF STIFFENING WALL WITH SUPPORT WALL

4.2.2.3 In case of halls exceeding 8.0 m in length, safety and adequacy of lateral supports shall always be checked by structural analysis.

4.2.2.4 A trussed roofing may not provide lateral support unless special measures are adopted to brace and anchor the roofing. However, in case of residential and similar buildings of conventional design with trussed roofing having cross walls, it may be assumed that stability requirements are met with by the cross walls and structural analysis for stability may be dispensed with.

4.2.2.5 Capacity of a cross wall, also called shear wall, sometimes to take horizontal loads and consequently bending moments increases, when parts of bearing walls act as flanges to the cross wall. Maximum overhanging length of bearing wall which could effectively function as a flange should be taken as 12*t* or *H*/6, whichever is less in case of *T*/*I* shaped walls, and 6*t* or *H*/16, whichever is less in case of *L*/*U* shaped walls, where *t* is the thickness of bearing wall and *H* is the total height of wall above the level being considered, as shown in Fig. 11.

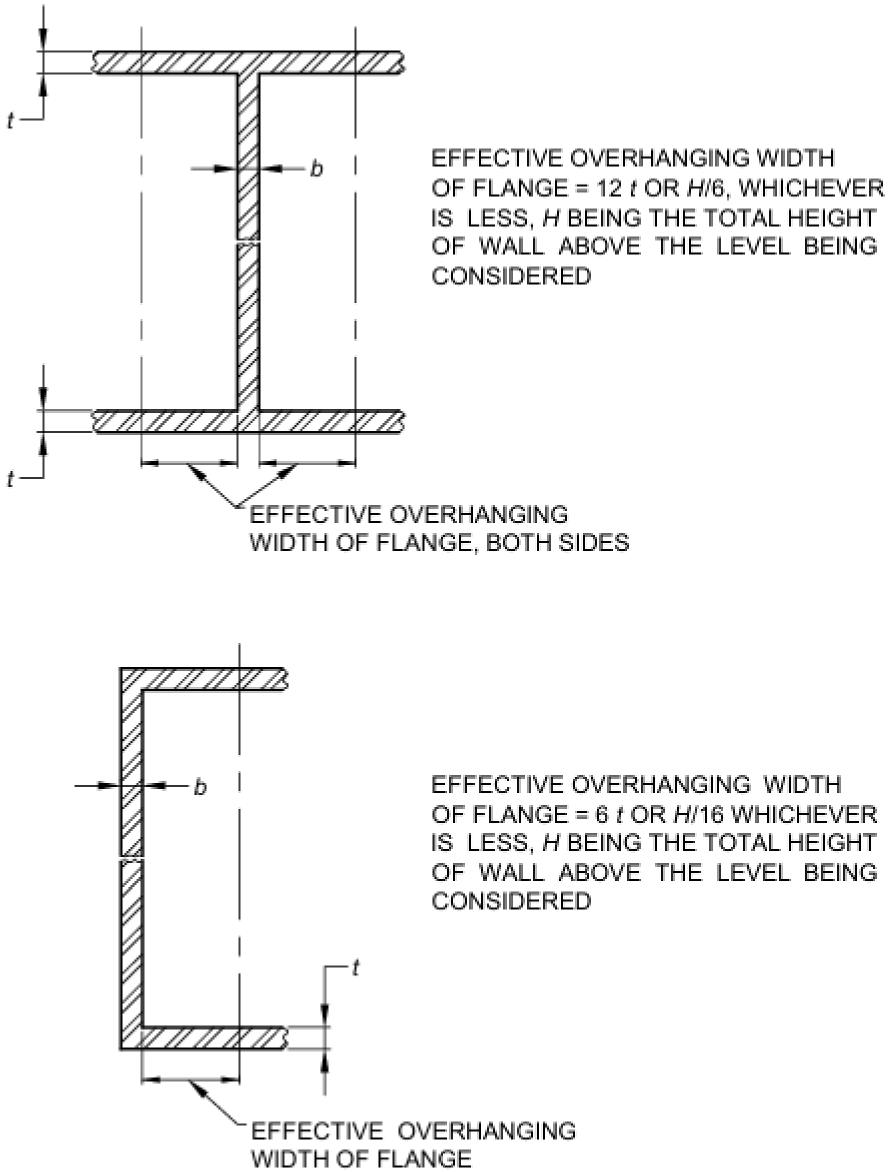


FIG. 11 TYPICAL DETAILS FOR ANCHORAGE OF SOLID WALLS

4.2.2.6 *External walls of basement and plinth*

In case of external walls of basement and plinth, stability requirements of **4.2.2** may be deemed to have been met with, if:

a) bricks used in basement and plinth have a minimum crushing strength of 5 N/mm^2 and mortar used in masonry is of Grade M1 or better;
b) clear height of ceiling in basement does not exceed 2.6 m;
c) walls are stiffened according to provisions of **4.2.2.1**;
d) in the zone of action of soil pressure on basement walls, traffic load excluding any surcharge due to adjoining buildings does not exceed 5 kN/m^2 and terrain does not rise; and
e) minimum thickness of basement walls is in accordance with Table 4. However, if percentage opening in the basement wall for purposes like ventilation are

more than 25 percent, then stability analysis/check need to be performed particularly for resistance to seismic loads.

NOTE – In case there is surcharge on basement walls from adjoining buildings, thickness of basement walls shall be based on structural analysis.

Table 4 Minimum Thickness of Basement Walls
[*Clause* 4.2.2.6(e)]

Sl No.	**Height of the Ground Above Basement Floor Level**		**Minimum Nominal Thickness of Basement Wall**
	Wall Loading (Permanent Load) Less than 50 kN/m	Wall Loading (Permanent Load) More than 50 kN/m	
(1)	(2)	(3)	(4)
i)	Up to 1.4 m	Up to 1.75 m	300 mm
ii)	Up to 2 m	Up to 2.5 m	400 mm

4.2.2.7 *Walls mainly subjected to lateral loads*

a) *Free standing wall* – A free standing wall such as compound wall or parapet wall is acted upon by wind force which tends to overturn it. This tendency to over-turning is resisted by gravity force due to self-weight of wall, and also by flexural moment of resistance on account of tensile strength of masonry. Free standing walls shall thus be designed as in **5.5.2.1**. If mortar used for masonry cannot be relied upon for taking flexural tension (*see* **5.4.2**), stability of free standing wall shall be ensured such that stability moment of wall due to self-weight equals or exceeds 1.5 times the overturning moment.
b) *Retaining wall* – Stability for retaining walls shall normally be achieved through gravity action but flexural moment of resistance could also be taken advantage of under special circumstances at the discretion of the designer (*see* **5.4.2**).

4.3 Effective Height

4.3.1 *Wall*

Effective height of a wall shall be taken as shown in Table 5 (*see* Fig. 12).

NOTE – A roof truss or beam supported on a column meeting the requirements of **4.2.2.1** is deemed to provide lateral support to the column only in the direction of the beam/truss.

Table 5 Effective Height of Walls
(*Clause* 4.3.1)

Sl No.	Condition of Support	Effective Height
(1)	(2)	(3)
i)	Lateral as well as rotational restraint (that is, full restraint) at top and bottom. For example, when the floor/roof spans on the walls so that reaction to load of floor/roof is provided by the walls, or when an RCC floor/roof has bearing on the wall (minimum 90 mm), irrespective of the direction of the span foundation footings of a wall give lateral as well as rotational restraint	0.75 H
ii)	Lateral as well as rotational restraint (that is, full restraint) at one end and only lateral restraint (that is, partial restraint) at the other. For example, RCC floor/roof at one end spanning or adequately bearing on the wall and timber floor/roof not spanning on wall, but adequately anchored to it, on the other end	0.85 H
iii)	Lateral restraint, without rotational restraint (that is, partial restraint) on both ends. For example, timber floor/roof, not spanning on the wall but adequately anchored to it on both ends of the wall, that is, top and bottom	1.00 H
iv)	Lateral restraint as well as rotational restraint (that is, full restraint) at bottom but have no restraint at the top. For example, parapet walls with RCC roof having adequate bearing on the lower wall, or a compound wall with proper foundation on the soil.	1.50 H

NOTES

1 *H* is the height of wall between centers of support in case of RCC slabs and timber floors. In case of footings or foundation block, height (*H)* is measured from top of footing or foundation block. In case of roof truss, height (*H*) is measured up to bottom of the tie beam. In case of beam and slab construction, height should be measured from centre of bottom slab to centre of top beam. All these cases are illustrated by means of examples shown in Fig. 13.

2 For working out effective height, it is assumed that concrete DPC, when properly bonded with masonry, does not cause discontinuity in the wall.

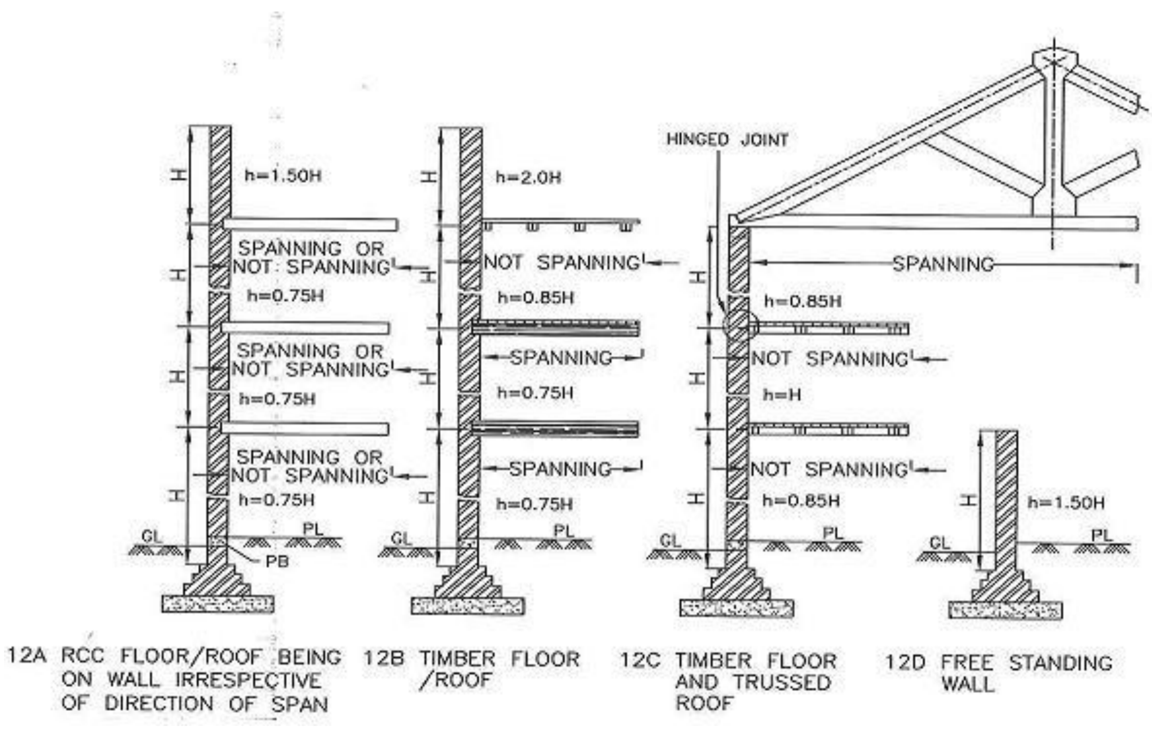
3 Where membrane type damp-proof course or termite shield causes a discontinuity in bond, the effective height of wall may be taken to be greater of the two values calculated as follows :

a) Consider *H* from top of footing ignoring DPC and take effective height as 0.75*H*.

b) Consider H from top of DPC and take effective height as 0.85 *H*.

4 When assessing effective height of walls, floors not adequately anchored to walls shall not be considered as providing lateral support to such walls.

5 When thickness of a wall bonded to a pier is at least two-thirds of the thickness of the pier measured in the same direction, the wall and pier may be deemed to act as one structural element.



NOTE — If Plinth band (PB) is equal to or more than 150 mm, height can be taken from above the plinth band.

FIG. 12 EFFECTIVE HEIGHT OF WALL

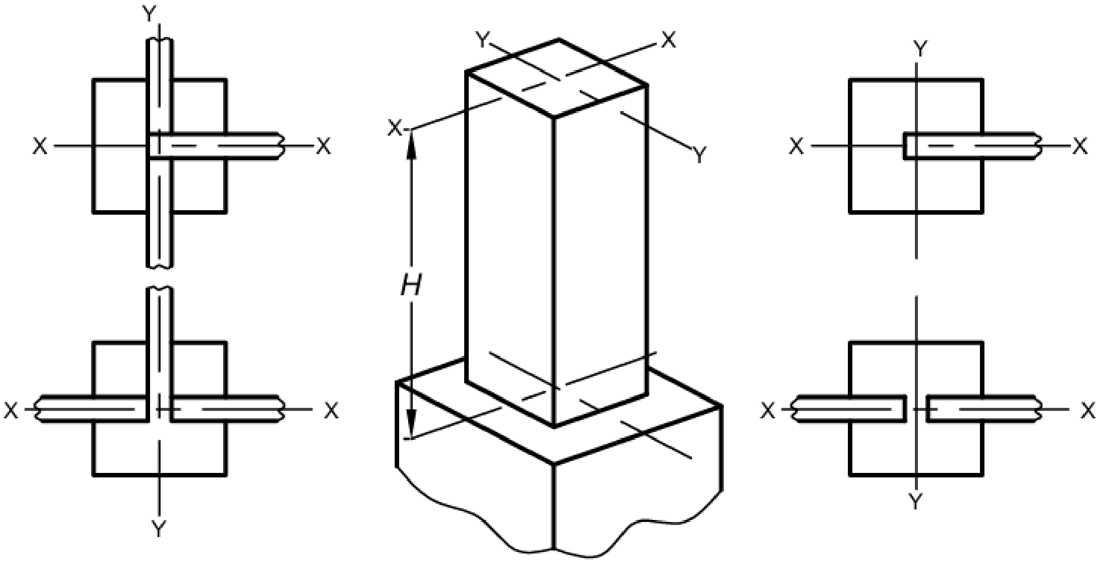
4.3.2 *Column*

In case of a column, effective height shall be taken as actual height for the direction it is laterally supported and as twice the actual height for the direction it is not laterally supported (*see* Fig. 13).

NOTES

1 A roof truss or beam supported on a column meeting the requirements of **4.2.2.1** is deemed to provide lateral support to the column only in the direction of the beam/truss.

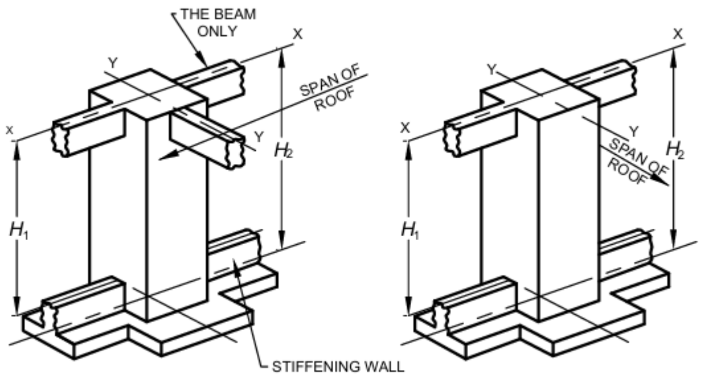
2 When floor or roof consisting of RCC beams and slabs is supported on columns, the columns would be deemed to be laterally supported in both directions.



EFFECTIVE HEIGHT ABOUT AXIS
X - X = 1.0 *H*
Y - Y = 1.0 *H*

13A

EFFECTIVE HEIGHT ABOUT AXIS
X - X = 2.0 *H*
Y - Y = 1.0 *H*



13B 13C

ROOF CONSTRUCTION	*EFFECTIVE HEIGHT ABOVE AXIS FIG. 13B*	*EFFECTIVE HEIGHT ABOVE AXIS FIG. 13C*
WITH PRECAST CONCRETE UNITS OF OF IN- SIDE CONCRETE FLOOR	X - X = 1.0 H_2 Y - Y = 1.0 H_1 Y - Y = 1.5 H_1 (NO TIES)	X - X = 1.5 H_2 Y - Y = 1.0 H_1
WITH LIGHT DECK OR SIMILAR ROOF	X - X = 1.0 H_2 Y - Y = 1.0 H_1 Y - Y = 2.0 H_1 (NO TIES)	X - X = 2.0 H_2 Y - Y = 1.0 H_1

FIG. 13 EFFECTIVE HEIGHT OF WALL

4.3.3 *Openings in Walls*

When openings occur in a wall such that masonry between the openings is by definition a column, effective height of masonry between the openings shall be reckoned as follows:

a) *When wall has full restraint at the top:*

1) Effective height for the direction perpendicular to plane of wall equals 0.75 H plus 0.25 H_1, where H is the distance between supports and H_1 is the height of the taller opening; and

2) Effective height for the direction parallel to the wall equals H, that is, the distance between the supports.

b) *When wall has partial restraint at the top and bottom:*

1) Effective height for the direction perpendicular to plane of wall equals H when height of neither opening exceeds 0.5 H and it is equal to 2 H when height of any opening exceeds 0.5 H; and

2) Effective height for the direction parallel to the plane of the wall equals 2 H.

4.4 Effective Length

Effective length of a wall shall be as given in Table 6.

Table 6 Effective Length of Walls
(*Clause* 4.4)

Sl No. (1)	**Conditions of Support (*See* Fig. 14)** (2)	**Effective Length** (3)
i)	Where a wall is continuous and is supported by cross wall and there is no opening within a distance of H/8 from the face of cross wall (*see* Fig. 14) or Where a wall is continuous and is supported by piers/ buttresses conforming to **4.2.1.2** (b)	0.8 L
ii)	Where a wall is supported by a cross wall at one end and continuous with cross wall at other end or Where a wall is supported by a pier/buttress at one end and continuous with pier/buttress at other end conforming to **4.2.1.2** (b)	0.9 L

iii)	Where a wall is supported at each end by cross wall or Where a wall is supported at each end by a pier/buttress conforming to **4.2.1.2** (b)	1.0 *L*
iv)	Where a wall is free at one end and continuous with a pier/buttress at the other end or Where a wall is free at one end and continuous with a pier/buttress at the other end conforming to **4.2.1.2** (b)	1.5 *L*
v)	Where a wall is free at one end and supported at the other end by a cross wall or Where a wall is free at one end and supported at the other end by a pier/buttress conforming to **4.2.1.2** (b)	2.0 *L*

where

L = length of wall from or between centres of cross wall, piers or buttress; and
H = actual height of wall between centers of adequate lateral support.

NOTE – In case there is an opening taller than 0.5 *H* in a wall, ends of the wall at the opening shall be considered as free. Cross walls shall conform to **4.2.2.2** (d).

4.5 Effective Thickness

Effective thickness to be used for calculating slenderness ratio of a wall or column shall be obtained as in **4.5.1** to **4.5.5**.

4.5.1 For solid walls, faced walls or columns, effective thickness shall be the actual thickness.

4.5.2 For solid walls adequately bonded into piers, buttresses, effective thickness for determining slenderness ratio based on effective height shall be the actual thickness of wall multiplied by stiffening coefficient as given in Table 7. No modification in effective thickness, however, shall be made when slenderness ratio is to be based on effective length of walls.

4.5.3 For solid walls or faced walls stiffened by cross walls, appropriate stiffening coefficient may be determined from Table 7 on the assumption that the cross walls are equivalent to piers of width equal to the thickness of the cross wall and of thickness equal to three times the thickness of stiffened wall.

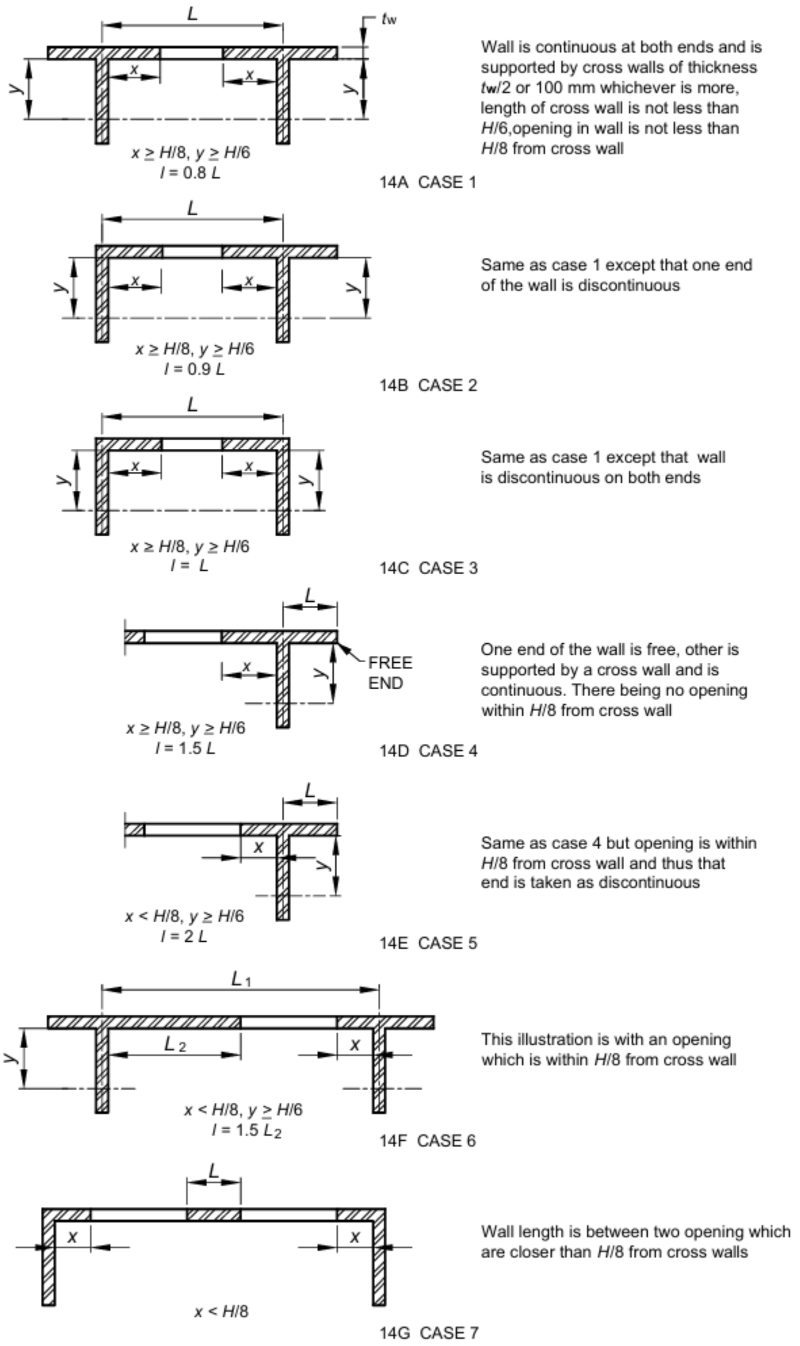


FIG. 14 EFFECTIVE LENGTH OF WALL

Table 7 Stiffening Coefficient for Walls Stiffened by Piers, Buttresses or Cross Walls
(*Clauses* 4.5.2 *and* 4.5.3)

Sl No.	Ratio $\frac{S_p}{w_p}$	Stiffening Coefficient		
		$\frac{t_p}{t_w}=1$	$\frac{t_p}{t_w}=2$	$\frac{t_p}{t_w}=3$ or more

(1)	(2)	(3)	(4)	(5)
i)	6	1.0	1.4	2.0
ii)	8	1.0	1.3	1.7
iii)	10	1.0	1.2	1.4
iv)	15	1.0	1.1	1.2
v)	20 or more	1.0	1.0	1.0

where

S_p = Centre-to-centre spacing of the pier or cross wall,
t_p = Thickness of pier as defined in **2.1.3 (b)** (*see* Fig. 1),
t_w = Actual thickness of the wall proper (*see* Fig. 1), and
w_p = Width of the pier in the direction of the wall or the actual thickness of the cross wall.

NOTE – Linear interpolation between the values given in this table is permissible but not extrapolation outside the limits given.

4.5.4 For cavity walls with both leaves of uniform thickness throughout, effective thickness shall be taken as two-thirds of the sum of the actual thickness of the two leaves.

4.5.5 For cavity walls with one or both leaves adequately bonded into piers, buttresses or cross walls at intervals, the effective thickness of the cavity wall shall be two-thirds of the sum of the effective thickness of each of the two leaves; the effective thickness of each leaf being calculated using **4.5.1** or **4.5.2** as appropriate.

4.6 Slenderness Ratio

4.6.1 *Walls*

For a wall, slenderness ratio shall be effective height divided by effective thickness or, effective length divided by the effective thickness, whichever is less. In case of a load bearing wall, slenderness ratio shall not exceed that given in Table 8. To reduce the slenderness ratio one and a half brick thick wall shall be up to ground level.

Table 8 Maximum Slenderness Ratio for a Load Bearing Wall
(*Clause* 4.6.1)

Number of Storeys	Maximum Slenderness Ratio	
	Using Portland Cement or Portland Pozzolana Cement in Mortar	Using Lime Mortar
(1)	(2)	(3)
Not exceeding 2	27	20

Exceeding 2	27	13

4.6.2 *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 column shall not exceed 12.

4.7 Eccentricity

Eccentricity of vertical loading at a particular junction in a masonry wall shall depend on factors, such as extent of bearing, magnitude of loads, stiffness of slab or beam, fixity at the support and constructional details at junctions. No exact calculations are possible to make accurate assessment of eccentricity. Extent of eccentricity under any particular circumstances has, therefore, to be decided according to the best judgment of the designer. Some guidelines for assessment of eccentricity are given in Annex A.

5 STRUCTURAL DESIGN

5.1 General

The building as a whole shall be analysed by accepted principles of mechanics to ensure safe and proper functioning in service of its component parts in relation to the whole building. All component parts of the structure shall be capable of sustaining the most adverse combinations of loads, which the building may be reasonably expected to be subjected to during and after construction.

5.2 Design Loads

Loads to be taken into consideration for designing masonry components of a structure are:

a) dead loads of walls, columns, floors and roofs;
b) imposed loads of floors and roof;
c) wind loads on walls and sloping roof; and
d) seismic forces.

NOTE – When a building is subjected to other loads, such as vibration from railways; machinery, etc, these should be taken into consideration accordingly to the best judgment of the designer (*see also* Part 6 ‘Structural Design, Section 1 Loads, forces and effects’ of the Code).

5.2.1 The design loads and other forces to be taken for the design of masonry structures shall conform to Part 6 ‘Structural design, Section 1 Loads, forces and effects’ of the Code.

NOTE – During construction, suitable measures shall be taken to ensure that masonry is not liable to damage or failure due to action of wind forces, back filling behind walls or temporary construction loads.

5.3 Load Dispersion

5.3.1 *General*

The angle of dispersion of vertical load on walls shall be taken as not more than 30° from the vertical.

5.3.2 *Arching Action*

Account may also be taken of the arching action of well-bonded masonry walls supported on lintels and beams, in accordance with established practice. Increased axial stresses in the masonry associated with arching action in this way, shall not exceed the permissible stresses given in **5.4**.

5.3.3 *Lintels*

Lintels that support masonry construction shall be designed to carry loads from masonry (allowing for arching and dispersion where applicable), and loads received from any other part of the structure. Length of bearing of lintel at each end shall not be less than 90 mm or one-tenth of the span, whichever is more, and area of the bearing shall be sufficient to ensure that stresses in the masonry (combination of wall stresses, stresses due to arching action and bearing stresses from the lintel) do not exceed the stresses permitted in **5.4** (*see* Annex C).

5.4 Permissible Stresses

5.4.1 *Permissible Compressive Stress*

Permissible compressive stress in masonry shall be based on value of basic compressive stress (f_b) as given in Table 9 and multiplying this value by factors known as stress reduction factor (k_s), area reduction factor (k_a) and shape modification factor (k_p) as detailed in **5.4.1.1** to **5.4.1.3**. Values of basic compressive stress given in Table 9 take into consideration crushing strength of masonry unit and grades of mortar, and hold good for values of slenderness ratio not exceeding 6, zero eccentricity and masonry unit having height to width ratio (as laid) equal to 0.75 or less.

Table 9 Basic Compressive Stresses for Masonry (After 28 days)
(*Clauses* 5.4.1 *and* 6.3.1)

Sl No.	**Mortar Type (Ref Table 1)**	**Basic Compressive Stresses, in N/mm², corresponding to Masonry Units of which Height to Width Ratio does not Exceed 0.75 and Crushing Strength, in N/mm², is not Less than**											
		3.5	5.0	7.5	10	12.5	15	17.5	20	25	30	35	40
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
i)	H1	0.35	0.50	0.75	1.00	1.16	1.31	1.45	1.59	1.91	2.21	2.5	3.05
ii)	H2	0.35	0.50	0.74	0.96	1.09	1.19	1.30	1.41	1.62	1.85	2.1	2.5

iii)	M1	0.35	0.50	0.74	0.96	1.06	1.13	1.20	1.27	1.47	1.69	1.9	2.2
iv)	M2	0.35	0.44	0.59	0.81	0.94	1.03	1.10	1.17	1.34	1.51	1.65	1.9
v)	M3	0.25	0.41	0.56	0.75	0.87	0.95	1.02	1.10	1.25	1.41	1.55	1.78
vi)	L1	0.25	0.36	0.53	0.67	0.76	0.83	0.90	0.97	1.11	1.26	1.4	1.06
vii)	L2	0.25	0.31	0.42	0.53	0.58	0.61	0.65	0.69	0.73	0.78	0.85	0.95

NOTES

1 The table is valid for slenderness ratio up to 6 and loading with zero eccentricity.
2 The values given for basic compressive stress are applicable only when the masonry is properly cured.
3 Linear interpolation is permissible for units having crushing strengths between those given in the table.
4 The permissible stress for random rubble masonry may be taken as 75 percent of the corresponding stress for coarsed walling of similar materials.
5 The strength of ashlar masonry (natural stone masonry of massive type with thin joints) is closely related to intrinsic strength of the stone and allowable working stress in excess of those given in the table may be allowed for such masonry at the discretion of the designer.
6 For calculation of basic compressive stress of stabilized soil block having thickness 100 mm or more, reference to specialist literature may be made.
7 If the work is inadequately supervised, strength should be reduced to three-fourth or less at the discretion of the designer.

Alternatively, basic compressive stress may be based on results of prism test given in Annex B on masonry made from masonry units and mortar to be actually used in a particular job.

5.4.1.1 *Stress reduction factor*

This factor, as given in Table 10, takes into consideration the slenderness ratio of the element and also the eccentricity of loading.

Table 10 Stress Reduction Factor for Slenderness Ratio and Eccentricity
(*Clause* 5.4.1.1)

Sl No.	**Slenderness Ratio**	**Eccentricity of Loading Divided by the Thickness of the Member**					
		0	1/24	1/12	1/6	1/4	1/3
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	6	1.00	1.00	1.00	1.00	1.00	1.00
ii)	8	0.95	0.95	0.94	0.93	0.92	0.91
iii)	10	0.89	0.88	0.87	0.85	0.83	0.81
iv)	12	0.84	0.83	0.81	0.78	0.75	0.72
v)	14	0.78	0.76	0.74	0.70	0.66	0.66
vi)	16	0.73	0.71	0.68	0.63	0.58	0.53
vii)	18	0.67	0.64	0.61	0.55	0.49	0.43
viii)	20	0.62	0.59	0.55	0.48	0.41	0.34
ix)	22	0.56	0.52	0.48	0.40	0.32	0.24
x)	24	0.51	0.47	0.42	0.33	0.24	—
xi)	26	0.45	0.40	0.35	0.25	—	—

xii)	27	0.43	0.38	0.33	0.22	—	—

NOTES

1 Linear interpolation between values is permitted.
2 Where in special cases the eccentricity of loading lies between 1/3 and 1/2 of the thickness of the member, the stress reduction factor should vary linearly between unity and 0.20 for slenderness ratio of 6 and 20, respectively.
3 Slenderness ratio of a member for sections within 1/8 of the height of the member above or below a lateral support may be taken to be 6.

5.4.1.2 *Area reduction factor*

This factor takes into consideration smallness of the sectional area of the element and is applicable when sectional area of the element is less than 0.2 m^2. The factor $k_a = 0.7 + 1.5A$, A being the area of section, in m^2.

5.4.1.3 *Shape modification factor*

This factor takes into consideration the shape of the unit, that is, height to width ratio (as laid) and is given in Table 11. This factor is applicable for units for crushing strength up to 15 N/mm^2.

Table 11 Shape Modification Factor for Masonry Units
(*Clause* 5.4.1.3)

Sl No.	**Height to Width Ratio of Units (As Laid)**	**Shape Modification Factor (k_p) For Units Having Crushing Strength in N/mm^2**			
		Upto 5.0	**7.5**	**10.0**	**15.0**
(1)	(2)	(3)	(4)	(5)	(6)
i)	Up to 0.75	1.0	1.0	1.0	1.0
ii)	1.0	1.2	1.1	1.1	1.0
iii)	1.5	1.5	1.3	1.2	1.1
iv)	2.0 to 4.0	1.8	1.5	1.3	1.2

NOTE – Linear interpolation between values is permissible.

5.4.1.4 *Increase in permissible compressive stresses allowed for eccentric vertical loads, lateral loads under certain conditions*

In members subjected to eccentric and/or lateral loads, increase in permissible compressive stress is allowed as follows:

a) When resultant eccentricity ratio exceeds 1/24 but does not exceed 1/6, 25 percent increase in permissible compressive stress is allowed in design.

b) When resultant eccentricity ratio exceeds 1/6, 25 percent increase in permissible stress is allowed but the area of the section under tension shall be disregarded for computing the load carrying capacity of the member.

NOTE – When resultant eccentricity ratio of loading is 1/24 or less, compressive stress due to bending shall be ignored and only axial stress need to be computed for the purpose of design.

5.4.1.5 *Increase in permissible compressive stress for walls subjected to concentrated loads*

When a wall is subjected to a concentrated load (a load being taken to be concentrated when area of supporting wall equals or exceeds three times the bearing area), certain increase in permissible compressive stress may be allowed because of dispersal of the load. Since, according to the present state of art, there is diversity of views in regard to manner and extent of dispersal, design of walls subjected to concentrated loads may, therefore, be worked out as per the best judgment of the designer. Some guidelines in this regard are given in Annex C.

5.4.2 *Permissible Tensile Stress*

As a general rule, design of masonry shall be based on the assumption that masonry is not capable of taking any tension. However, in case of lateral loads normal to the plane of wall, which causes flexural tensile stress, as for example, panel, curtain partition and free standing walls, flexural tensile stresses as follows may be permitted in the design for masonry:

a) Grade M1 or better mortar

1) 0.07 N/mm^2 for bending in the vertical direction where tension developed is normal to bed joints.

2) 0.14 N/mm^2 for bending in the longitudinal direction where tension developed is parallel to bed joints, provided crushing strength of masonry units is not less than 10 N/mm^2.

b) Grade M2 mortar

1) 0.05 N/mm^2 for bending in the vertical direction where tension developed is normal to bed joints.

2) 0.10 N/mm^2 for bending in the longitudinal direction where tension developed is parallel to bed joints, provided crushing strength of masonry units is not less than 7.5 N/mm^2.

NOTES

1 No tensile stress is permitted in masonry in case of water-retaining structures in view of water in contact with masonry. Also no tensile stress is permitted in earth-retaining structures, in view of the possibility of presence of water at the back of such walls.

2 Allowable tensile stress in bending in the vertical direction may be increased to 0.1 N/mm^2 for M1 mortar and 0.07 N/mm^2 for M2 mortar in case of boundary walls/compound at the discretion of the designer, since there is not much risk to life and property in the event of failure of such walls.

5.4.3 *Permissible Shear Stress*

In case of walls built in mortar not leaner than Grade M1 (*see* Table 1) and resisting horizontal forces in the plane of the wall, permissible shear stress calculated on the area of bed joints, shall not exceed the value obtained by the formula given below, subject to a maximum of 0.5 N/mm^2:

$$f_s = 0.1 + f_d/6$$

f_d = compressive stress due to dead loads, in N/mm^2; and
f_s = permissible shear stress, in N/mm^2.

5.4.4 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.

5.5 Design Thickness/Cross-Section

5.5.1 *Walls and Columns Subjected to Vertical Loads*

Walls and columns bearing vertical loads shall be designed on the basis of permissible compressive stress. Design consists in determining thickness in case of walls and section in case of columns in relation to strength of masonry units and grade of mortar to be used, taking into consideration various factors, such as slenderness ratio, eccentricity, area of section, workmanship, quality of supervision, etc, subject further to provisions of **5.5.1.1** to **5.5.1.4**.

5.5.1.1 *Solid walls*

Thickness used for design calculation shall be the actual thickness of masonry computed as the sum of the average dimensions of the masonry units specified in the relevant standard, together with the specified joint thickness. In masonry with raked joints, thickness shall be reduced by the depth of raking of joints for plastering/pointing.

5.5.1.2 *Cavity walls*

a) Thickness of each leaf of a cavity wall shall not be less than 75 mm.
b) Where the outer leaf is half masonry unit in thickness, the uninterrupted height and length of this leaf shall be limited so as to avoid undue loosening of ties due to differential movements between the two leaves. The outer leaf shall, therefore, be supported at least at every third storey or at every 10 m of height, whichever is less and at every 10 m or less along the length.
c) Where the load is carried by both leaves of a wall of a cavity construction, the permissible stress shall be based on the slenderness

ratio derived from the effective thickness of the wall as given in **4.5.4** or **4.5.5**. The eccentricity of the load shall be considered with respect to the centre of gravity of the cross-section of the wall.

d) Where the load is carried by one leaf only, the permissible stress shall be the greater of values calculated by the following two alternative methods:

1) The slenderness ratio is based on the effective thickness of the cavity wall as a whole as given in **4.5.4** or **4.5.5** and on the eccentricity of the load with respect to the centre of gravity of the cross-section of the whole wall (both leaves) (This is the same method as where the load is carried by both the leaves but the eccentricity will be more when the load is carried by one leaf only).
2) The slenderness ratio is based on the effective thickness of the loaded leaf only using **4.5.1** and **4.5.2**, and the eccentricity of the load will also be with respect to the centre of gravity of the loaded leaf only.

In either alternative, only the actual thickness of the load bearing leaf shall be used in arriving at the cross-sectional area resisting the load (*see* **5.5.1.1**).

5.5.1.3 *Faced wall*

The permissible load per length of wall shall be taken as the product of the total thickness of the wall and the permissible stress in the weaker of the two materials. The permissible stress shall be found by using the total thickness of the wall when calculating the slenderness ratio.

5.5.1.4 *Veneered wall*

The facing (veneer) shall be entirely ignored in calculations of strength and stability. For the purpose of determining the permissible stress in the backing, the slenderness ratio shall be based on the thickness of the backing alone.

5.5.2 *Walls and Columns Mainly Subjected to Lateral Loads*

5.5.2.1 *Free standing walls*

a) Free standing walls, subjected to wind pressure or seismic forces shall be designed on the basis of permissible tensile stress in masonry or stability as in **4.2.2.4**. However, in seismic Zones II, free-standing walls may be apportioned without making any design calculations with the help of Table 12 provided the mortar used is of grade not leaner than M1.

b) If there is a horizontal damp-proof course near the base of the wall, that is, not capable of developing tension vertically, the minimum wall thickness should be the greater of that calculated from either,

1) the appropriate height to thickness ratio given in Table 12 reduced by 25 percent, reckoning the height from the level of the damp-proof course; or

2) the appropriate height to thickness ratio given in Table 12 reckoning the height from the lower level at which the wall is restrained laterally.

Table 12 Height to Thickness Ratio of Free Standing Walls Related to Wind Speed
[*Clause* 5.5.2.1(a) *and* (b)]

Sl No.	**Design Wind Pressure** N/mm^2	**Height to Thickness Ratio**
(1)	(2)	(3)
i)	Up to 285	10
ii)	575	7
iii)	860	5
iv)	1 150	4

NOTES

1 For intermediate values, linear interpolation is permissible.
2 Height is to be reckoned from 150 mm below ground level or top of footing/foundation block, whichever is higher, and up to the top edge of the wall.
3 The thickness should be measured including the thickness of the plaster.

5.5.2.2 *Retaining walls*

Normally masonry of retaining walls shall be designed on the basis of zero-tension, and permissible compressive stress. However, in case of retaining walls for supporting horizontal thrust from dry materials, retaining walls may be designed on the basis of permissible tensile stress at the discretion of the designers.

5.5.3 *Walls and Columns Subjected to Vertical as well as Lateral Loads*

For walls and columns, stress worked out separately for vertical loads as in **5.5.1** and lateral loads as in **5.5.2** shall be combined and elements designed on the basis of permissible stress.

5.5.4 *Walls Subjected to In-Plane Bending and Vertical Loads (Shear Walls)*

Walls subjected to in-plane bending and vertical loads, that is, shear walls shall be designed on the basis of no tension with permissible shear stress and permissible compressive stress.

5.5.5 *Non-Load Bearing Walls*

Non-load bearing walls, such as panel walls, curtain walls and partition walls which are mainly subjected to lateral loads, according to present state of art, are not capable of precise design and only approximate methods based on some tests are available. Guidelines for approximate design of these walls are given in Annex D.

6 GENERAL REQUIREMENTS

6.1 Methods of Construction

6.1.1 *General*

Construction of the following types of load bearing and non-load bearing masonry walls shall be carried out in accordance with good practice [6-4(4)]:

a) Brickwork,
b) Stone masonry,
c) Hollow concrete block masonry,
d) Gypsum partition blocks,
e) Autoclaved cellular concrete block masonry, and
f) Lightweight concrete block masonry.

6.1.2 *Construction of Buildings in Seismic Zones*

Special features of construction for earthquake resistant masonry buildings in Zones II, III, IV and V shall be applicable according to **8**.

6.2 Minimum Thickness of Walls from Consideration other than Structural

Thickness of walls determined from consideration of strength and stability may not always be adequate in respect of other requirements, such as resistance to fire, thermal insulation, sound insulation and resistance to damp penetration for which reference may be made to the appropriate Parts/Sections of the Code, and thickness suitably increased, where found necessary.

6.3 Workmanship

6.3.1 *General*

Workmanship has considerable effect on strength of masonry and bad workmanship may reduce the strength of brick masonry to as low as half the intended strength. The basic compressive stress values for masonry as given in Table 9 holds good for commercially obtainable standards of workmanship with reasonable degree of supervision.

6.3.2 *Bedding of Masonry Units*

Masonry units shall be laid on a full bed of mortar with frog, if any, upward such that cross-joints and wall joints are completely filled with mortar. Masonry units which are moved after initial placement shall be re-laid in fresh mortar, discarding the disturbed mortar.

6.3.3 *Bond*

Cross-joints in any course of one brick thick masonry wall shall be not less than one-fourth of a masonry unit in horizontal direction from the cross-joints in the course below. In masonry walls more than one brick in thickness, bonding through the thickness of wall shall be provided by either header units or by other equivalent means in accordance with good practice [6-4(5)].

6.3.4 *Verticality and Alignment*

All masonry shall be built true and plumb within the tolerances prescribed below; care shall be taken to keep the perpends properly aligned:

a) Deviation from vertical within a storey shall not exceed 6 mm per 3 m height.
b) Deviation in verticality in total height of any wall of a building more than one storey in height shall not exceed 12.5 mm.
c) Deviation from position shown on plan of any brickwork shall not exceed 12.5 mm.
d) Relative displacement between load bearing walls in adjacent storeys intended to be in vertical alignment shall not exceed 6 mm.
e) Deviation of bed-joint from horizontal in a length of 12 m shall not exceed 6 mm subject to a maximum deviation of 12 mm.
f) Deviation from the specified thickness of bed joints, cross-joints and perpends shall not exceed one-fifth of the specified thickness.

NOTE – These tolerances have been specified from the point of view of their effect on the strength of masonry. The permissible stress recommended in **5.3** may be considered applicable only, if these tolerances are adhered to.

6.4 Joints to Control Deformation and Cracking

Special provision shall be made to control or isolate thermal and other movements so that damage to the fabric of the building is avoided and its structural sufficiency preserved. Design and installation of joints shall be done according to the appropriate recommendations in accordance with good practice [6-4(6)].

6.5 Chases, Recesses and Holes

6.5.1 Chases, recesses and holes are permissible in masonry only if these do not impair strength and stability of the structure.

6.5.2 In masonry, designed by structural analysis, all chases, recesses and holes shall be considered in structural design and detailed in building plans.

6.5.3 When chases, recesses and holes have not been considered in structural design and are not shown in drawings, these may be provided, subject to the constraints and precautions specified in **6.5.3.1** to **6.5.3.10**.

6.5.3.1 As far as possible, services should be planned with the help of vertical chases and use of horizontal chases should be avoided.

6.5.3.2 For load bearing walls, depth of vertical and horizontal chases shall not exceed one-third and one-sixth of the wall thickness, respectively.

6.5.3.3 Vertical chases shall not be closer than 2 m in any stretch of wall and shall not be located within 345 mm of an opening or within 230 mm of a cross wall that serves as a stiffening wall for stability. Width of a vertical chase shall not exceed thickness of wall in which it occurs.

6.5.3.4 When unavoidable horizontal chases of width not exceeding 60 mm in a wall having slenderness ratio not exceeding 15 may be provided. These shall be located in the upper or lower middle third height of wall at a distance not less than 600 mm from a lateral support. No horizontal chase shall exceed 1 m in length and there shall not be more than 2 chases in any one wall. Horizontal chases shall have minimum mutual separation distance of 500 mm. Sum of lengths of all chases and recesses in any horizontal plane shall not exceed one-fourth the length of the wall.

6.5.3.5 Holes for supporting put-logs of scaffolding shall be kept away from bearings of beams, lintels, and other concentrated loads. If unavoidable, stresses in the affected area shall be checked to ensure that these are within safe limits.

6.5.3.6 No chase, recess or hole shall be provided in any stretch of a masonry wall, the length of which is less than four times the thickness of wall, except when found safe by structural analysis.

6.5.3.7 Masonry directly above a recess or a hole, if wider than 300 mm, shall be supported on a lintel. No lintel, however, is necessary in case of a circular recess or hole exceeding 300 mm in diameter provided upper half of the recess or hole is built as a semi-circular arch of adequate thickness and there is an adequate length of masonry on the sides of openings to resist the horizontal thrust.

6.5.3.8 As far as possible, chases, recesses and holes in masonry should be left (inserting sleeves, where necessary) at the time of construction of masonry so as to obviate subsequent cutting. If cutting is unavoidable, it should be done without damage to the surrounding or residual masonry. It is desirable to use such tools for cutting which depend upon rotary and not on heavy impact for cutting action.

6.5.3.9 No chase, recess or hole shall be provided in half-brick load bearing wall, excepting the minimum number of holes needed for scaffolding.

6.5.3.10 Chases, recesses or holes shall not be cut into walls made of hollow or perforated units, after the units have been incorporated in masonry.

6.6 Corbelling

6.6.1 Where corbelling is required for the support of some structural element, maximum projection of masonry unit should not exceed one-half of the height of the unit or one-half of the built-in part of the unit and the maximum horizontal projection of the corbel should not exceed one-third of the wall thickness.

6.6.2 The load per unit length on a corbel shall not be greater than half of the load per unit length on the wall above the corbel. The load on the wall above the corbel, together with four times the load on the corbel, shall not cause the average stress in the supporting wall or leaf to exceed the permissible stresses given in **5.4**.

6.6.3 It is preferable to adopt header courses in the corbelled portion of masonry from considerations of economy and stability.

7 REINFORCED BRICK AND REINFORCED BRICK CONCRETE FLOORS AND ROOFS

The construction and design of reinforced brick and reinforced brick concrete floors and roof shall be in accordance with good practices [6-4(7)].

8 SPECIAL CONSIDERATION FROM EARTHQUAKE POINT OF VIEW

The design and construction of masonry buildings shall be governed by provisions of **5**, except as modified by the provisions of this section for those elements participating in lateral force resistance. The design and construction for earthquake resistant masonry buildings are given in **8, 9** and **10.** The earthquake resistant design and detailing of masonry buildings having the following structural systems:

a) Masonry Walls with Prescriptive Bands (MWB) ,
b) Masonry Walls with Prescriptive Bands and Vertical Reinforcements (MWBR),
c) Confined Masonry Walls (CMW) (see **9**), and
d) Reinforced Masonry Walls (RMW) (see **10**).

Buildings with MWB and MWBR systems shall not be of height more than 15 m subject to a maximum of four storeys, measured from the base of the buildings to the roof slab or ridge level.

8.1 For the purpose of **8** ~~and **9**~~, the following definitions shall apply in addition to those given in **2**.

8.1.1 *Separation section* **–** A gap of specified width between adjacent buildings or parts of the same building either left uncovered or covered suitably to permit movement in order to avoid pounding due to earthquake.

8.1.1.1 *Crumple section* **–** The separation gap filled with appropriate material that crumples or factures in the event of an earthquake.

8.1.2 *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.
8.1.3 *Load Bearing Wall* — A wall designed to resist axial force, shear force in its own plane and bending moment about its major axis.
8.1.4 *Box System* — A building made of masonry load bearing walls and horizontal floors.
8.1.5 *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.

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

8.1.7 *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).

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

8.1.9 *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.

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

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

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

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

8.1.12 *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.

8.1.13 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.

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

8.1.15 *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.

8.1.16 *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.

8.1.17 *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.

8.1.18 *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.

8.1.19 *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.

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

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

8.1.22 *Band* – A reinforced concrete or reinforced brick runner provided in the walls to tie them together and to impart horizontal bending strength in them.

NOTE — For low strength masonry buildings, wooden runners may also be used as runner.

8.2 General Principles

The general principles given in **8.2.1** to **8.2.9** shall be observed in construction of earthquake resistant buildings.

8.2.1 *Lightness*

Since the earthquake force is a function of mass, the weight of the building shall be as minimum as possible, consistent with structural safety and functional requirements. Roofs and upper storeys of buildings, in particular, should be designed as light as possible.

8.2.2 *Continuity of Construction*

8.2.2.1 As far as possible, the parts of the building should be tied together in such a manner that the building acts as one unit.

8.2.2.2 For parts of buildings between separation or crumple sections to expansion joints, floor slabs shall be continuous throughout as far as possible. Concrete slabs shall be rigidly connected or integrally cast with the support beams.

8.2.2.3 Additions to the structures shall be accompanied by the provision of separation or crumple sections between the new and the existing structures as far as possible, unless positive measures are taken to establish continuity between the existing and the new construction.

8.2.2.4 Alteration to the building structure shall be done by maintaining its structural stability by ensuring proper load path.

8.2.3 *Projecting and Suspended Parts*

8.2.3.1 Projecting parts shall be avoided as far as possible. If the projecting parts cannot be avoided, they shall be properly reinforced and firmly tied to the main structure, and their design shall be in accordance with good practice [6-4(8)].

8.2.3.2 Ceiling plaster shall preferably be avoided. When it is unavoidable, the plaster shall be as thin as possible.

8.2.3.3 Suspended ceiling shall be avoided as far as possible. Where provided they shall be light, adequately framed and secured to which electrical fixtures shall be fully secured.

8.2.4 *Building Configuration*

8.2.4.1 Masonry buildings shall have walls placed uniformly along the two principal plan directions. Buildings with the irregularities listed in **5.1** of IS 1893 (Part 2) perform poorly during earthquake shaking, and should be avoided. When these irregularities appear in a building, the guidance given in **5.1** of IS 1893 (Part 2) for the respective irregularity shall be followed. In order to minimize torsion and stress concentration, provisions given in **8.2.4.2** to **8.2.4.4** should be complied with as relevant.

8.2.4.2 The building should have a simple rectangular plan and be symmetrical both with respect to mass and rigidity so that the centre of mass and rigidity of the building coincide with each other in which case no separation sections other than expansion joints are necessary. For provision of expansion joints reference may be made to [6-4(6)].

8.2.4.3 If symmetry of the structure is not possible in plan, elevation or mass, provision shall be made for torsional and other effects due to earthquake forces in the structural design or the parts of different rigidities may be separated through crumple sections. The length of such building between separation sections shall not preferably exceed three times the width.

NOTE – As an alternative to separation section to reduce torsional moments, the centre of rigidity of the building may be brought close or coincident to the centre of mass by adjusting the locations and/or sizes of columns and walls.

8.2.4.4 Buildings having plans with shapes like L, T, E and Y shall preferably be separated into rectangular parts by providing separation sections at appropriate places. Typical examples are shown in Fig. 15.

NOTES

1 The buildings with small lengths of projections forming L, T, E or Y shapes need not be provided with separation section. In each cases the length of the projection may not exceed 15 to 20 percent of the total dimension of the building in the direction of the projection (*see* Fig. 16).

2 For buildings with minor asymmetry in plan and elevation, separation sections may be omitted.

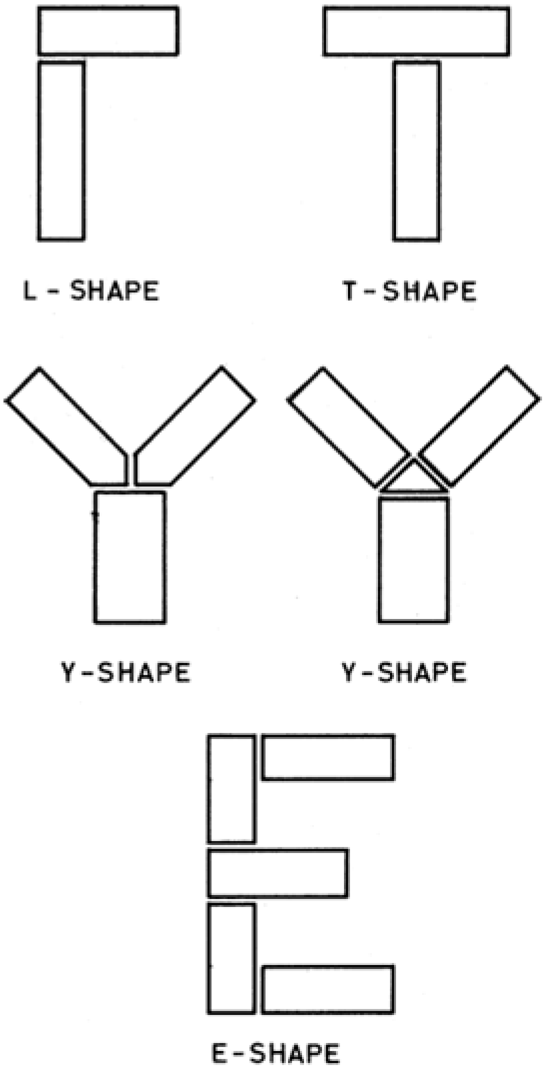
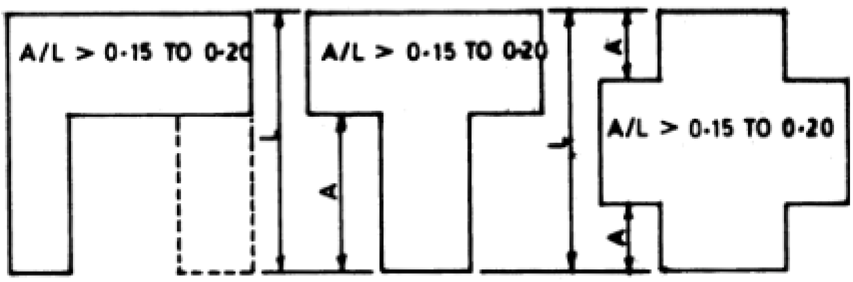
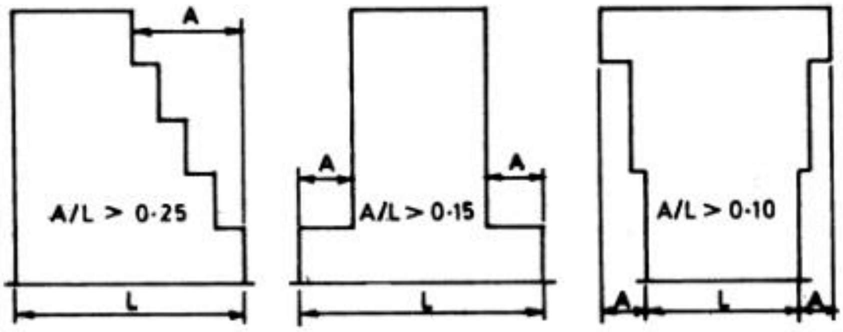


FIG. 15 TYPICAL SHAPES OF BUILDING
WITH SEPARATION SECTIONS



16A PLAN IRREGULARITIES



16B VERTICAL IRREGULARITIES

FIG. 16 PLAN AND VERTICAL IRREGULARITIES

8.2.5 *Strength in Various Directions*

The structure shall be designed to have adequate strength against earthquake effects along both the horizontal axes. The design shall also be safe considering the reversible nature of earthquake forces.

8.2.6 *Foundations*

The structure shall not be founded on such soils which shall subside or liquefy during an earthquake, resulting in large differential settlements (*see also* **8.3.3.3**).

8.2.7 *Ductility*

The main structural elements and their connection shall be designed to have a ductile failure. This shall enable the structure to absorb energy during earthquakes to avoid sudden collapse of the structure. Providing reinforcing steel in masonry at critical sections, as specified in this Section shall not only increase strength and stability but also ductility. The details for achieving ductility in reinforced concrete structures is given in good practice [6-4(11)].

8.2.8 *Damage to Non-Structural Parts*

Suitable details shall be worked out to connect the non-structural parts with the structural framing so that the deformation of the structural frame leads to minimum damage of the non-structural elements.

8.2.9 *Fire Safety*

Fire frequently follows an earthquake and therefore, buildings shall be constructed to make them fire resistant in accordance with the provisions of Part 4 'Fire and Life Safety' of the Code for fire safety.

8.3 Special Construction Features

8.3.1 *Separation of Adjoining Structures*

8.3.1.1 Separation of adjoining structures or parts of the same structure is required for structures having different total heights or storey heights and different dynamic characteristics. This is to avoid collision during an earthquake.

8.3.1.2 Minimum width of separation gaps as mentioned in **8.3.1.1**, shall be as specified in Table 13. The design seismic coefficient to be used shall be in accordance with **5** of Part 6 'Structural Design, Section 1 Loads, Forces and Effects' of the Code.

Table 13 Gap Width for Adjoining Structures
(*Clause* 8.3.1.2)

Sl. No.	**Buildings Categories for Earthquake Resisting Features:**	**Gap Width per Storey for Adjoining Masonry Building** mm
(1)	(2)	(3)
i)	Frames with shear walls	15
ii)	Category B1)	10
iii)	Category C[1)]	10
iv)	Category D[1)]	15
v)	Category E[1)]	20

[1)] See Table 14

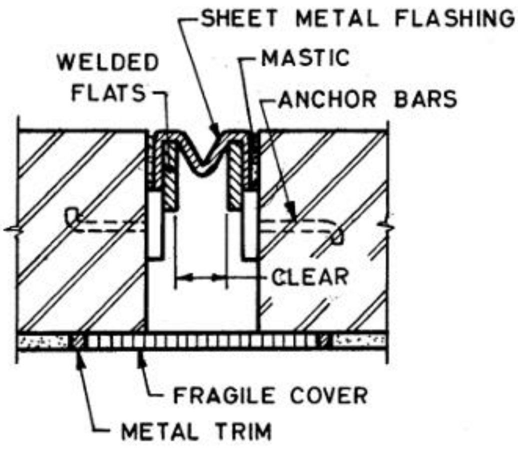
NOTE – Minimum total gap shall be 25 mm. For any other value of A_h the gap width shall be determined proportionately.

8.3.1.3 Where separation is necessary, a complete separation of the parts shall be made except below the plinth level. The plinth beams, foundation beams and footings may be continuous. Where separation sections are provided in a long building, they shall take care of movement owing to temperature changes also.

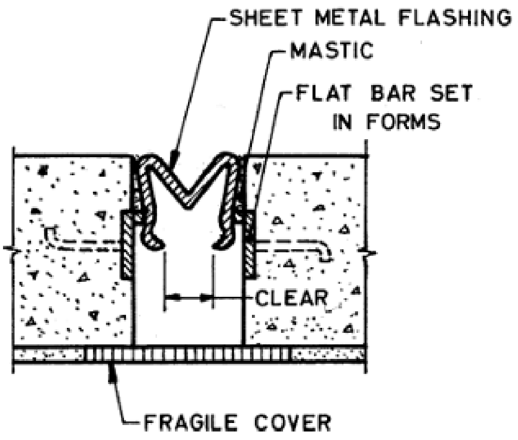
8.3.2 *Separation or Crumple Section*

8.3.2.1 In case of framed construction, members shall be duplicated on either side of the separation or crumple section. As an alternative, in certain cases, such duplication may not be provided, if the portions on either side can act as cantilevers to take the weight of the building and other relevant loads.

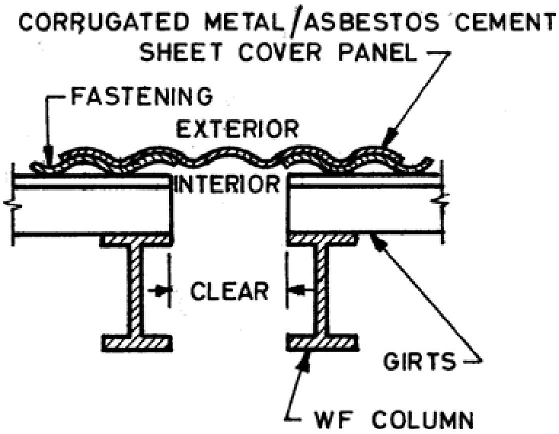
8.3.2.2 Typical details of separation and crumple sections are shown in Fig. 17. For other types of joint details, reference may be made to [6-4(5)].



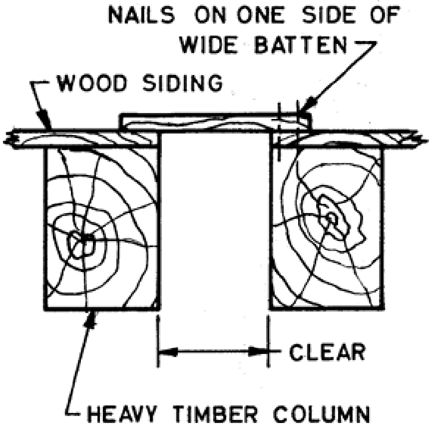
17A BRICK WALL



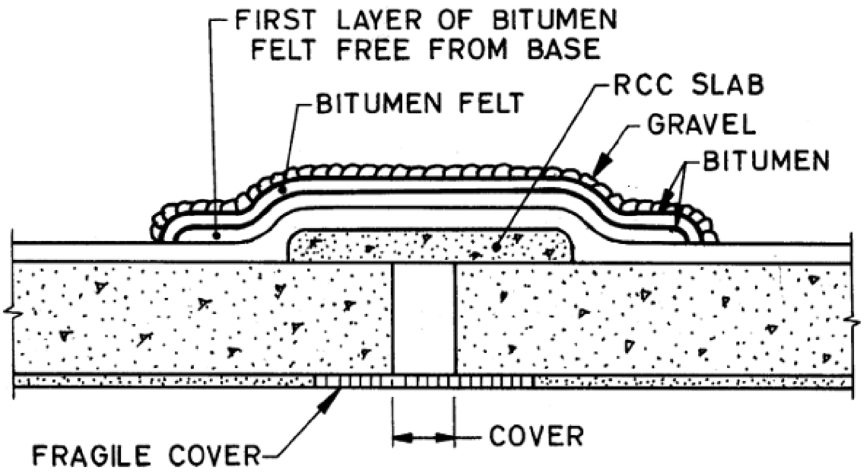
17B CONCRETE WALLS



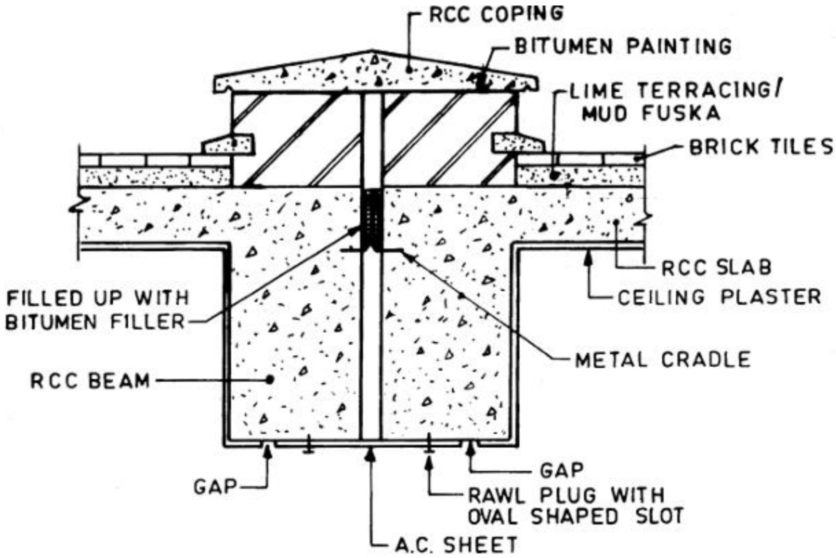
17C METAL SLIDING INDUSTRIAL WORK



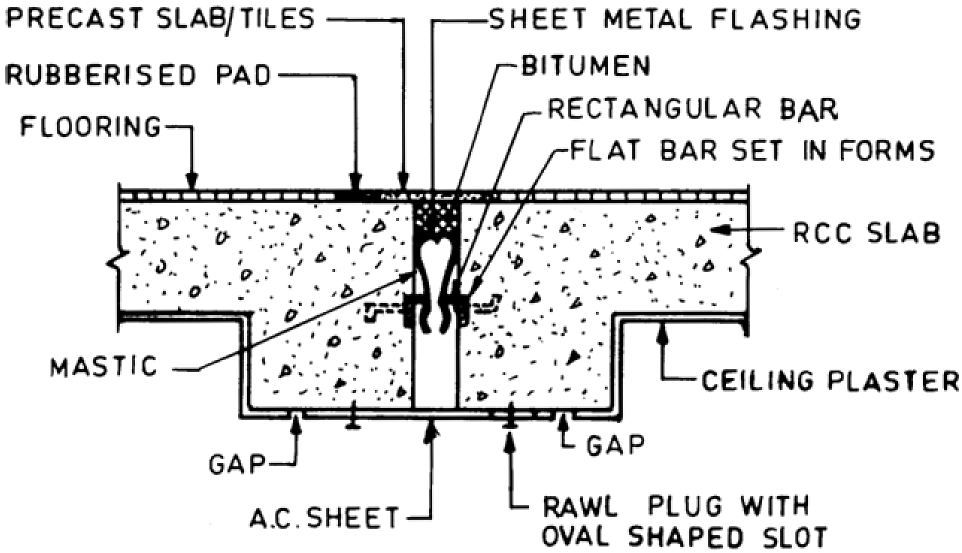
17D WOOD SHEATHING INDUSTRIAL WORK



17E RCC SLAB ON ROOF SURFACE



17F SEPARATION JOINT DETAILS AT ROOF



17G SEPARATION AT FLOOR LEVEL

FIG. 17 TYPICAL DETAILS OF SEPARATION OR CRUMPLE SECTION

8.3.3 *Foundations*

8.3.3.1 For the design of foundations, the provisions of Part 6 ‘Structural Design Section 1 Loads, Forces and Effects and Section 2 Soils and Foundation’ of the Code shall generally be followed.

8.3.3.2 The subgrade below the entire area of the building shall preferably be of the same type of the soil. Wherever this is not possible, a suitably located separation or crumple section shall be provided.

8.3.3.3 Loose fine sand, soft silt and expansive clays should be avoided. If unavoidable, the building shall rest either on a rigid raft foundation or on piles taken to a firm stratum. However, for light constructions the following measures may be taken to improve the soil on which the foundation of the building may rest:

a) Sand piling, and
b) Soil stabilization.

8.3.3.4 *Isolated footings for columns*

All the individual footings or pile caps where used in Type III Soft soils (Table 1 of good practice [6-4(9)], shall be connected by reinforced concrete ties at least in two directions approximately at right angles to each other. For buildings with no basement, the ties may be placed at or below the plinth level and for buildings with basement they may be placed at the level of basement floor. They shall need to be designed to carry the load of the panel walls also.

NOTE – The ties may not be necessary where structural floor connects the columns at or below the plinth level.

8.3.3.4.1 Where ties are used, their sections shall be designed to carry in tension as well as in compression, an axial load not less than the earthquake force in the direction the tie acting on the heavier of the columns connected, but the sections shall not be less than 200 mm x 200 mm with M20 concrete reinforced with 4 bars of 12 mm diameter plain mild steel bars or 10 mm diameter high strength deformed bars, one at each corner, bound by 8 mm diameter stirrups not more than 150 mm apart.

NOTE – In working out the buckling strength of ties, the lateral support provided by the soil may be taken into account. Calculations show that for such buried ties, lateral buckling is not a problem and the full section of the tie may be taken effective as a short column.

8.3.3.4.2 In the case of reinforced concrete slab, the thickness shall not be less than 1/50th of the clear distance between the footings, but not less than 100 mm in any case. It shall be reinforced with not less than 0.15 percent mild steel bars or 0.12 percent high strength deformed bars in each direction placed symmetrically at top and bottom.

8.3.4 *Roofs and Floors*

8.3.4.1 Flat roof or floor shall not preferably be made of tiles of ordinary bricks supported on steel, timber or reinforced concrete joists, nor they shall be of a type which in the event of an earthquake is likely to be loosened and parts or all of which may fall. If this type of construction cannot be avoided, the joists should be blocked at ends and bridged at intervals such that their spacing is not altered during an earthquake.

8.3.4.1.1 For pitched roofs, corrugated iron or asbestos sheets shall be used in preference to country, Allahabad or Mangalore tiles or other loose roofing units. All roofing materials shall be properly tied to the supporting members. Heavy roofing materials shall generally be avoided.

8.3.4.2 *Pent roofs*

8.3.4.2.1 All roof trusses shall be supported on reinforced concrete or reinforced brick band (*see* **8.6.4.3**). The holding down bolts shall have adequate size and length as required for earthquake forces in accordance with **5** of Part 6 'Structural Design Section 1 Loads, Forces and Effects' of the Code.

Where a trussed roof adjoins a masonry gable, the ends of the purlins shall be carried on and secured to a plate or bearer which shall be adequately bolted to reinforced concrete or reinforced brick band at the top of gable end masonry (*see* **8.6.4.4**).

NOTE – Hipped roof in general have shown better structural behaviour during earthquakes than gable ended roofs.

8.3.4.2.2 At tie level all the trusses and the gable end shall be provided with diagonal braces in plan so as to transmit the lateral shear due to earthquake force to the gable walls acting as shear walls at the ends as specified in **8.6.4**.

8.3.4.3 *Jack arches*

Jack arched roofs or floors, where used shall be provided with mild steel ties in all spans along with diagonal braces in plan to ensure diaphragms actions.

8.3.5 *Staircases*

8.3.5.1 The inter-connection of the stairs with the adjacent floors should be appropriately treated by providing sliding joints at the stairs to eliminate their bracing effect on the floors. Large stair halls shall preferably be separated from the rest of the building by means of separation or crumple sections.

8.3.5.2 Three types of stair construction may be adopted as described below:

a) *Separated staircases* – One end of the staircase rests on a wall and the other end is carried by columns and beams which have no connection with the floors. The gap at the vertical joints between the floor and the staircase may be covered either with a tread plate attached to one side of the joint and sliding on the other side, or covered with some appropriate material which could crumple or fracture during an earthquake without causing structural damage. The supporting members, columns or walls, are isolated from the surrounding floors by means of separation or crumple sections. A typical example is shown in Fig. 18.

b) *Built-in staircase* – When stairs are built monolithically with floors, they can be protected against damage by providing rigid walls at the stair opening. An arrangement, in which the staircase is enclosed by two walls, is given in Fig. 19. In such cases, the joints, as mentioned in respect of separated staircases, shall not be necessary.

   The two walls mentioned above, enclosing the staircase, shall extend through the entire height of the stairs and to the building foundations.

c) *Staircases with sliding joints* – In case it is not possible to provide rigid walls around stair openings for built-in staircase or to adopt the separated staircases, the staircases shall have sliding joints so that they shall not act as diagonal bracing.

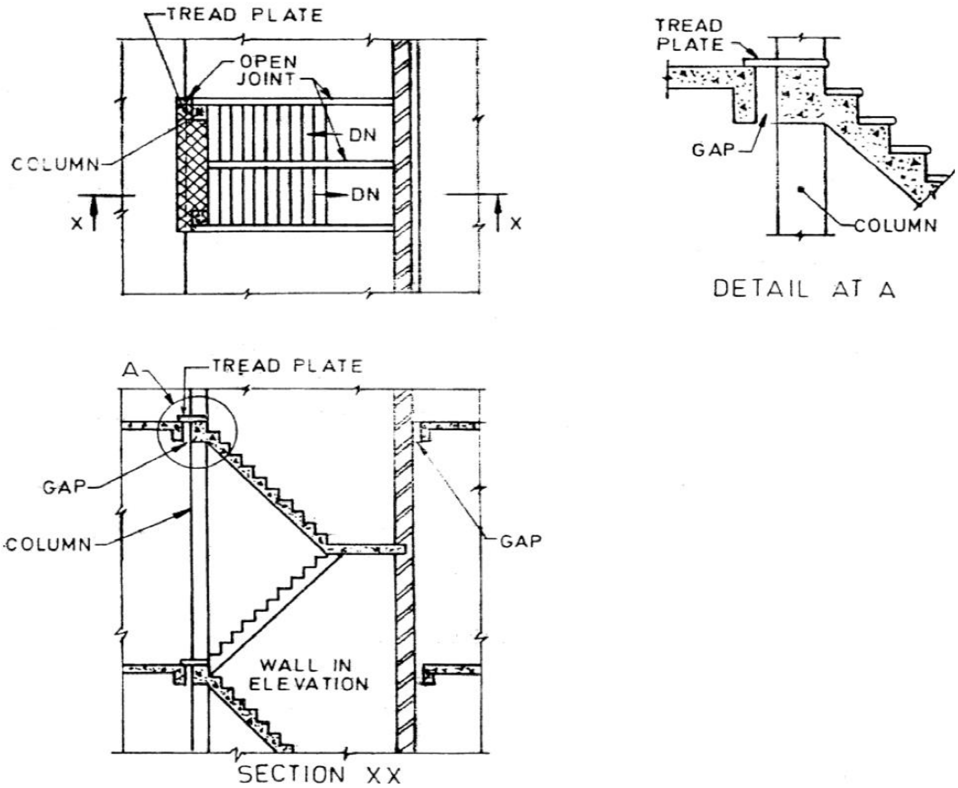


FIG. 18 SEPARATED STAIRCASE

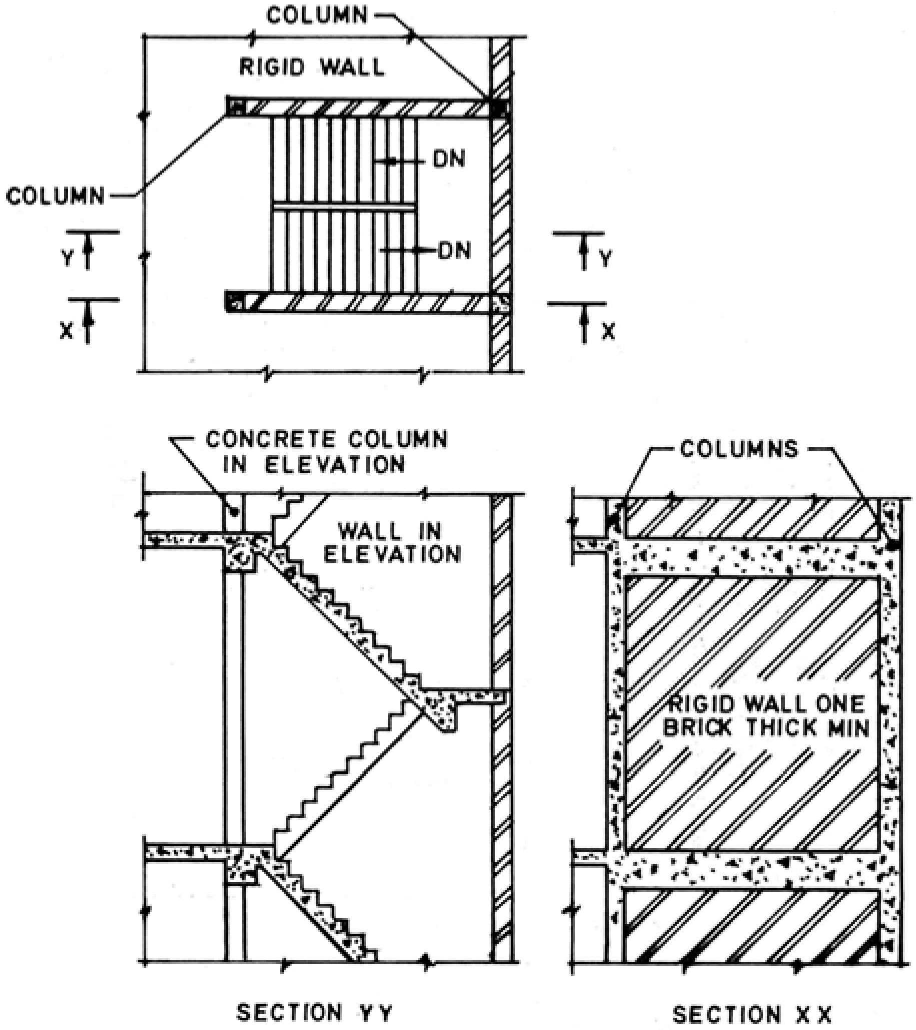


FIG. 19 RIGIDLY BUILT-IN STAIRCASE

8.3.6 *Framing of Thin Load Bearing Walls* (*see* Fig. 20)

Load bearing walls can be made thinner than 200 mm say 150 mm inclusive of plastering on both sides. Reinforced concrete framing columns and collar beams shall be necessary to be constructed to have full bond with the walls. Columns are to be located at all corners and junctions of walls and spaced not more than 1.5 m apart but so located as to frame up the doors and windows. The horizontal bands or ring beams are located at all floors, roof as well as lintel levels of the openings. The sequence of construction between walls and columns shall be first to build the wall up to 4 to 6 courses height leaving toothed gaps (tooth projection being about 40 mm only) for the columns and second to pour M20 (1 : 1½ : 3) concrete to fill the columns against the walls using wood forms only on two sides. The columns steel should be accurately held in position all along. The band concrete should be cast on the wall masonry directly so as to develop full bond with it.

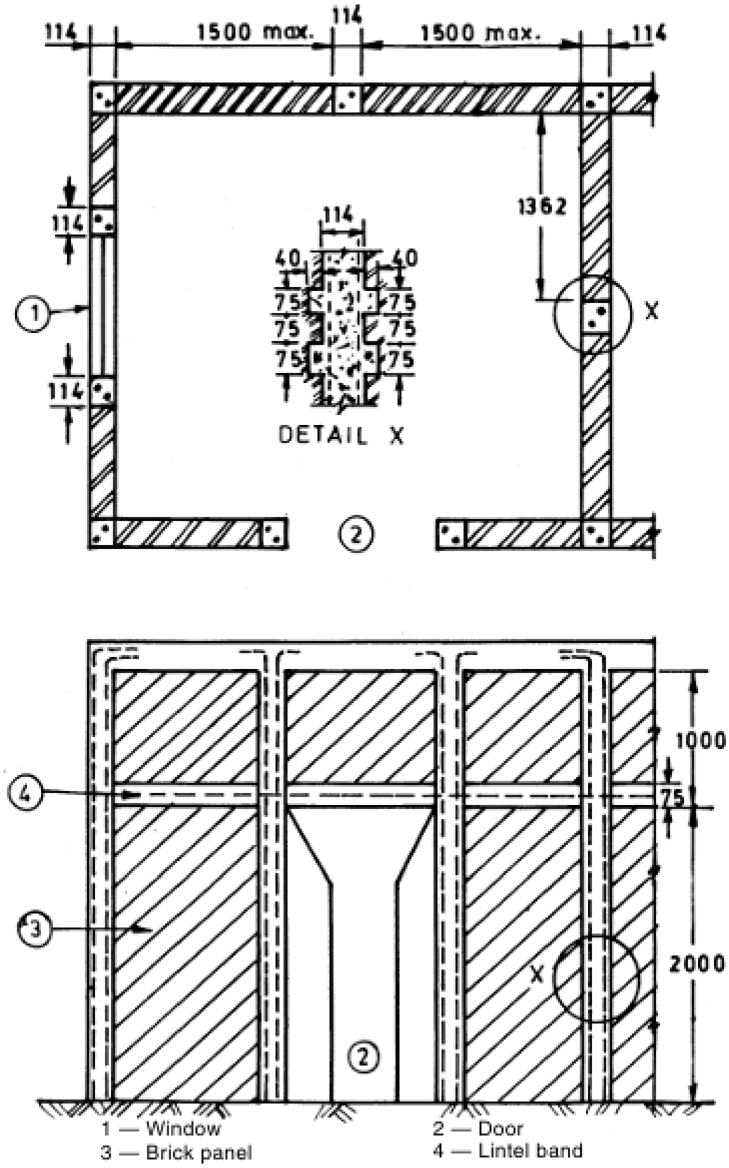
Such construction may be limited to only two storeys maximum in view of its vertical load carrying capacity. The horizontal length of walls between cross walls shall be restricted to 7 m and the storey height to 3 m.

8.3.7 *Reinforcing Details for Hollow Block Masonry*

The following details may be followed in placing the horizontal and vertical steel in hollow block masonry using cement-sand or cement-concrete blocks.

8.3.7.1 *Horizontal band*

U-shaped blocks may be used for construction of horizontal bands at various levels of the storeys as shown in Fig. 21, where the amount of horizontal reinforcement shall be taken 25 percent more than that given in Table 18 and provided by using four bars and 6 mm diameter stirrups. Other continuity details shall be followed, as shown in Fig. 25.



All dimensions in millimeters

FIG. 20 FRAMING OF THIN LOAD-BEARING BRICK WALLS

NOTES

1 Detailing of steel around the window openings will be same as for door openings with additional 75 mm band at sill level.
2 Figures are for just explanation purposes and the same shall not be used as design guide.

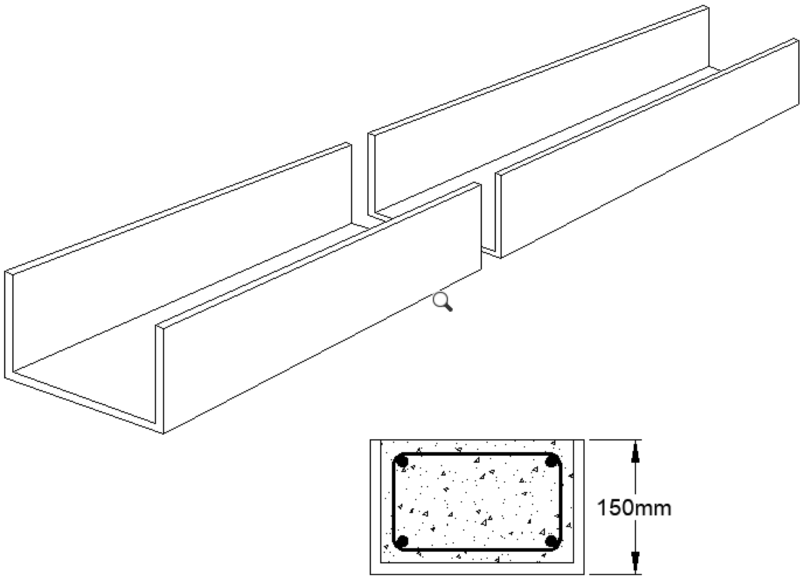


Fig. 21 U - BLOCKS FOR HORIZONTAL BANDS

8.3.7.2 *Vertical reinforcement*

Bars, as specified in Table 19 shall be located inside the cavities of the hollow blocks, one bar in each cavity (*see* Fig. 29). Where more than one bar is planned these can be located in two or three consecutive cavities. The cavities containing bars are to be filled by using micro-concrete 1 : 2 : 3 or cement coarse sand mortar 1 : 3, and properly rodded for compaction. The vertical bars should be spliced by welding or overlapping for developing full tensile strength. For proper bonding, the overlapped bars should be tied together by winding the binding wire over the lapped length. To reduce the number of overlaps, the blocks may be made U-shaped as shown in Fig. 29 which shall avoid lifting and threading of bars into the hollows.

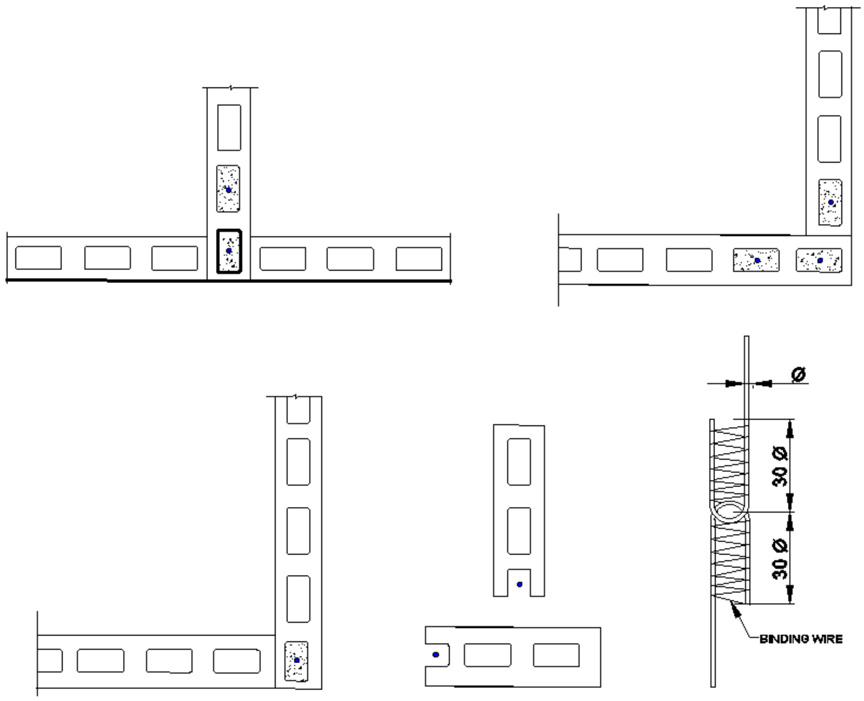


FIG. 22 VERTICAL REINFORCEMENT IN CAVITIES

8.4 DESIGN CONSIDERATIONS

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

8.4.1 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 design loads imposed on the structure as per the load combinations specified in **7.5** of IS 1893 (Part 1).

8.4.1.1 *Working Stress Method*

In the Working Stress Method of design of members, permissible stresses given in Table 14 shall govern the earthquake resistant design of members of the masonry building systems specified in this section.

8.4.2 Bands and Vertical Elements

Masonry buildings, other than those with reinforced masonry walls and confined masonry walls, 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).

8.4.2.1 *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.

8.4.2.2 *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 shall be maintained for the reinforcement.

The vertical thickness of RC Band shall be at least:

a) 75 mm, where two longitudinal bars are specified, one on each face; and

b) 150 mm, where four bars are specified, one at each corner.

8.4.2.3 *Ductile Detailing*

The bands shall be made of reinforced concrete of grade not leaner than M25 or reinforced brickwork in cement mortar not leaner than 1:3. In coastal areas, the filling mortar with water proofing admixture shall be used.

Table 14 Permissible stresses in materials to be used design of members of masonry buildings
(*Clause* 8.4.1.1)

Sl No.	Compression		Tension	Shear
	Axial	Axial and Flexural	Axial and Flexural	
(1)	(2)	(3)	(4)	(5)
	a) Masonry Wall with Prescriptive Bands (MWB) b) Masonry Wall with Prescriptive Bands and Vertical Reinforcements (MWBR)			
i)	**5.4.1** and Table 9 apply	Increase in permissible compressive stress permitted as per **5.4.1.4** applies	**5.4.2** 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** *applies*
	c) Reinforced Masonry Wall (RMW)			
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 1.25F_a.	**5.4.2** applies.	*For Flexural members* *(i) without web reinforcement:* $F_v = 0.083\sqrt{f_m}$ $\leq 0.75MPa$ *(ii) with web reinforcement:* $F_v = 0.25\sqrt{f_m}$ $\leq 0.75MPa$ *For walls, shall be as per* **5.4.3**
	d) Confined Masonry Wall (CMW)			
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 1.25f_a	**5.4.2** applies.	*Allowable shear stress shall be according to* **5.4.3**

8.4.3 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.

8.4.4 Building Category

Masonry buildings shall be categorized into four types (Table 15), namely Types B, C, D and E, based on the earthquake zone and the category of the building [mentioned in Table 5 of IS1893 (Part 2)]. Type E buildings shall not be more than 3 storeys tall.

8.4.4.1 *Slenderness of Walls*

The masonry walls shall meet the slenderness limit specified in Table 16.

8.4.5 Footings

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

Table 15 Building Types depending on category of building and earthquake zone
(*Clause* 8.4.4)

<table>
<tr><th rowspan="2">Sl No.</th><th rowspan="2">Category of Building</th><th colspan="5">Earthquake Zone</th></tr>
<tr><th>II</th><th>III</th><th>IV</th><th>V</th><th>VI</th></tr>
<tr><td>(1)</td><td>(2)</td><td>(3)</td><td>(4)</td><td>(5)</td><td>(6)</td><td>(7)</td></tr>
<tr><td>i)</td><td>Normal Building</td><td>*B*</td><td>*C*</td><td>*D*</td><td>*E*</td><td>*E*</td></tr>
<tr><td>ii)</td><td>Important Building</td><td>C</td><td>D</td><td colspan="3">Not Admissible as per IS 1893 (Part 2)</td></tr>
<tr><td>iii)</td><td>Critical Building</td><td colspan="5" rowspan="2">MWB and MWBR are not admissible as per IS 1893 (Part 2)</td></tr>
<tr><td>iv)</td><td>Special Building</td></tr>
</table>

Table 16 Allowable Height-to-Thickness Ratios of Unreinforced Masonry Walls
(*Clause* 8.4.4.1)

Sl No.	Storey	Building Type			
		B	C	D	E
(1)	(2)	(3)	(4)	(5)	(6)
i)	Parapet	2.5	2.5	2.0	1.5
ii)	Top Storey	14	14	14	9
iii)	Other Storeys	16	16	16	12
iv)	First Storey	18	18	18	15

8.5 Masonry Walls with Prescriptive Bands (MWB), and Masonry Walls with Prescriptive Bands and Vertical Reinforcement (MWBR)

Masonry buildings shall have the features specified hereunder.

MWB and MWBR are not admissible structural systems for critical buildings and special buildings as per IS 1893 (Part 2) in all zones, and for important building category in earthquake zones IV, V and VI.

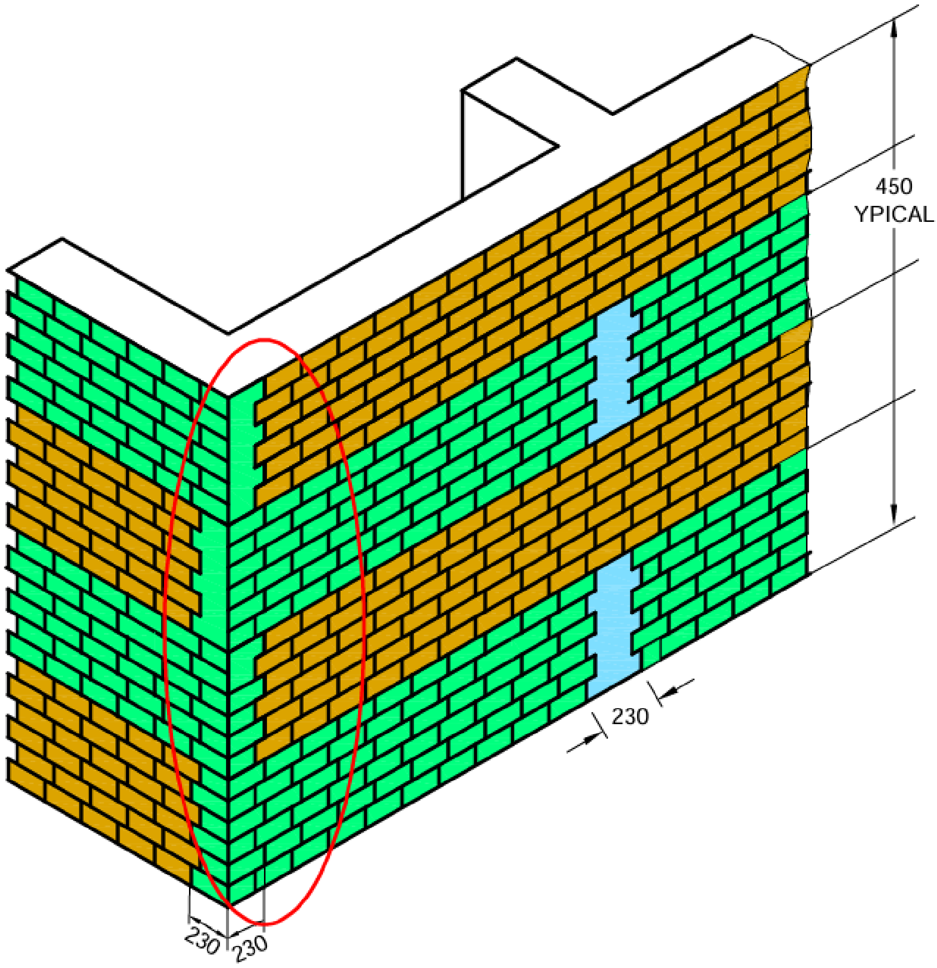
8.5.1 Masonry Walls

8.5.1.1 The bearing walls in both directions shall be straight and symmetrical in plan.

8.5.1.2 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 mm or larger thickness and of storey height less than 3.5 m

8.5.1.3 *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 (Fig. 23).



All dimensions are in mm.

FIG. 23 ALTERNATING TOOTHED JOINTS IN WALLS AT CORNER AND T-JUNCTION

8.5.2 Openings in Bearing Walls

8.5.2.1 Door and window openings reduce the lateral load resistance of walls, and hence, should preferably be small and more centrally located in the wall panels. The guidelines on the size and position of opening are given in Table 17 and Fig. 24.

8.5.2.2 Openings in any storey shall have their top at the same level to accommodate a continuous band over them, including the lintels throughout the building. In addition, openings in different storeys in a masonry wall shall be vertically aligned, and not staggered.

8.5.2.3 Where openings do not comply with the guidelines of Table 17, they should be strengthened by providing reinforced concrete or reinforcing the brickwork (Fig. 25) with high strength deformed bars of 8 mm diameter but the quantity of steel shall be increased at the jambs to comply with **8.5.3.9**, if so required.

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

8.5.2.5 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.

8.5.2.6 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.

Table 17 Size and Position of Openings in Bearing Walls
(*Clause* 8.5.2.1)

S.No.	**Position of Opening**	**Details of Opening**		
		B	**C**	**D, E**
(1)	(2)	(3)	(4)	(5)
i)	Minimum Distance b_5 (mm) from inside corner of outside wall	0	230	450
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 b_4 (mm) of pier between consecutive openings	340	450	560
iv)	Minimum vertical distance h_3 (mm) between two openings (one above the other)	600	600	600
v)	Maximum Width b_8 (mm) of opening of ventilator	900	900	900

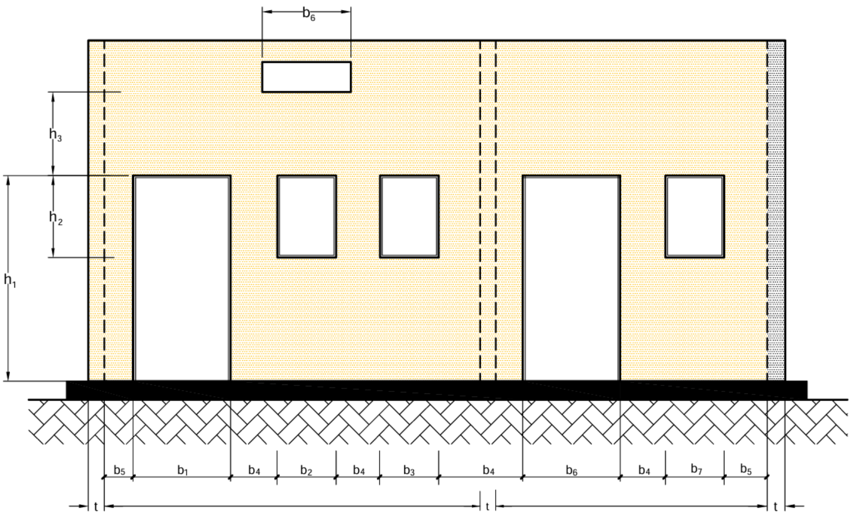


FIG. 24 DIMENSIONS OF OPENINGS AND PIERS FOR RECOMMENDATIONS IN TABLE 4

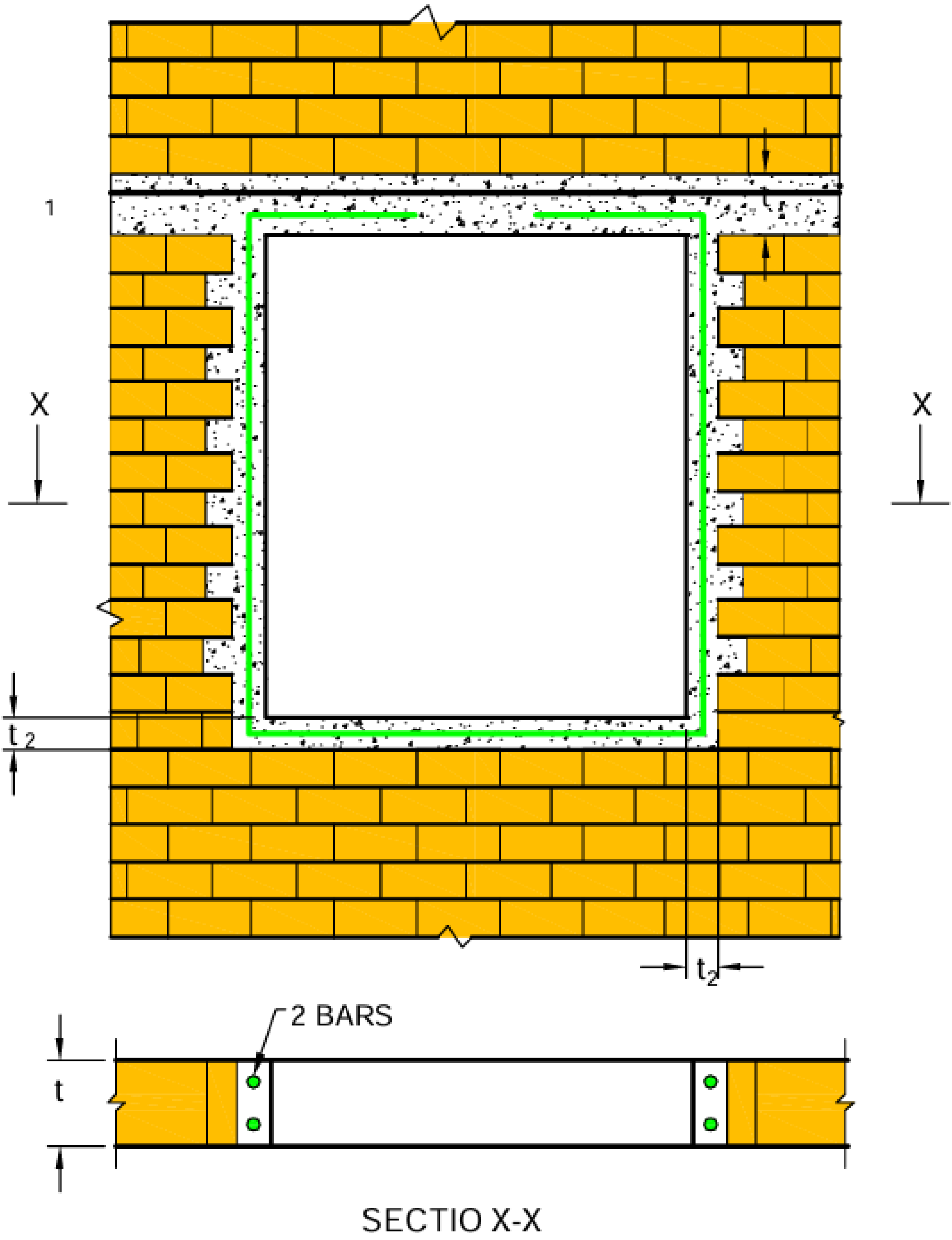


FIG. 25 STRENGTHENING MASONRY AROUND AN OPENING

8.5.3 Earthquake Strengthening Arrangements

8.5.3.1 All masonry buildings shall be strengthened by the methods specified for different types of buildings, as listed in Table 18, and detailed in subsequent clauses. Fig. 26 and Fig. 27 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 IS 1893 (Part 1). If this analysis shows that vertical steel is not required, the designer may take the decision accordingly.

Table 18 Measures recommended in masonry buildings made with rectangular masonry units
(*Clause* 8.5.3.1)

Sl No.	**Type of Building**	**Number of Storeys**	**Measures 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, b, c, d, e, f, g
iii)	D	1 and 2	a, b, c, d, e, f, g
		3 and 4	a, b, c, d, e, f, g, h
iv)	E	1 to 3	a, b, c, d, e, f, g, h

where
- a Masonry mortar (*as per* **8.3.2**);
- b Lintel band (*as per* **8.5.3.2**);
- c Roof band and gable band where necessary (*as per* **8.5.3.3** and **8.5.3.4**);
- d Vertical steel at corners and junctions of walls (*as per* **8.5.3.8**);
- e Vertical steel at jambs of openings (*as per* **8.5.3.9**);
- f Bracing in plan at tie level of roofs;
- g Plinth band where necessary (*as per* **8.5.3.6**); and
- h Dowel bars (*as per* **8.5.3.7**).

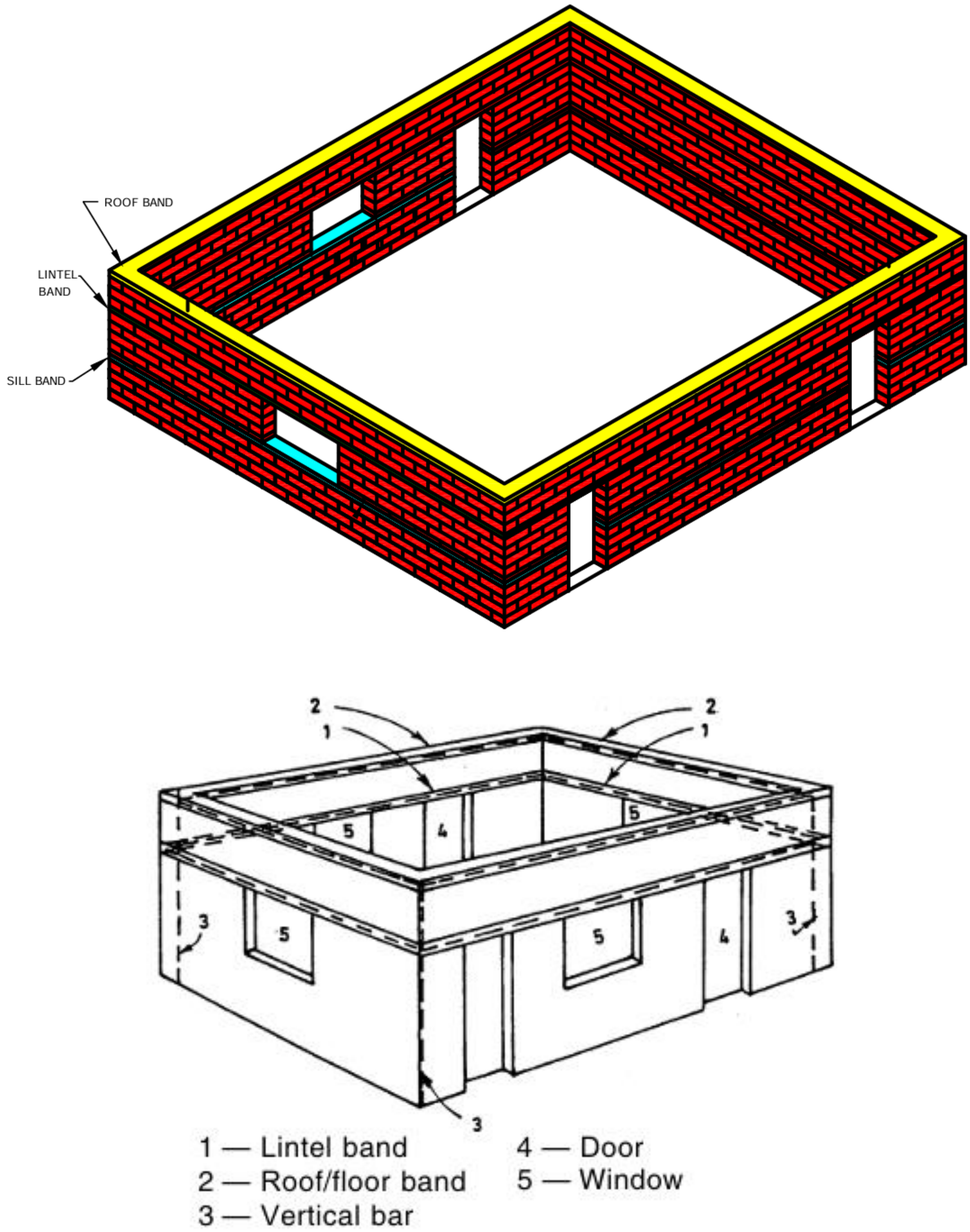
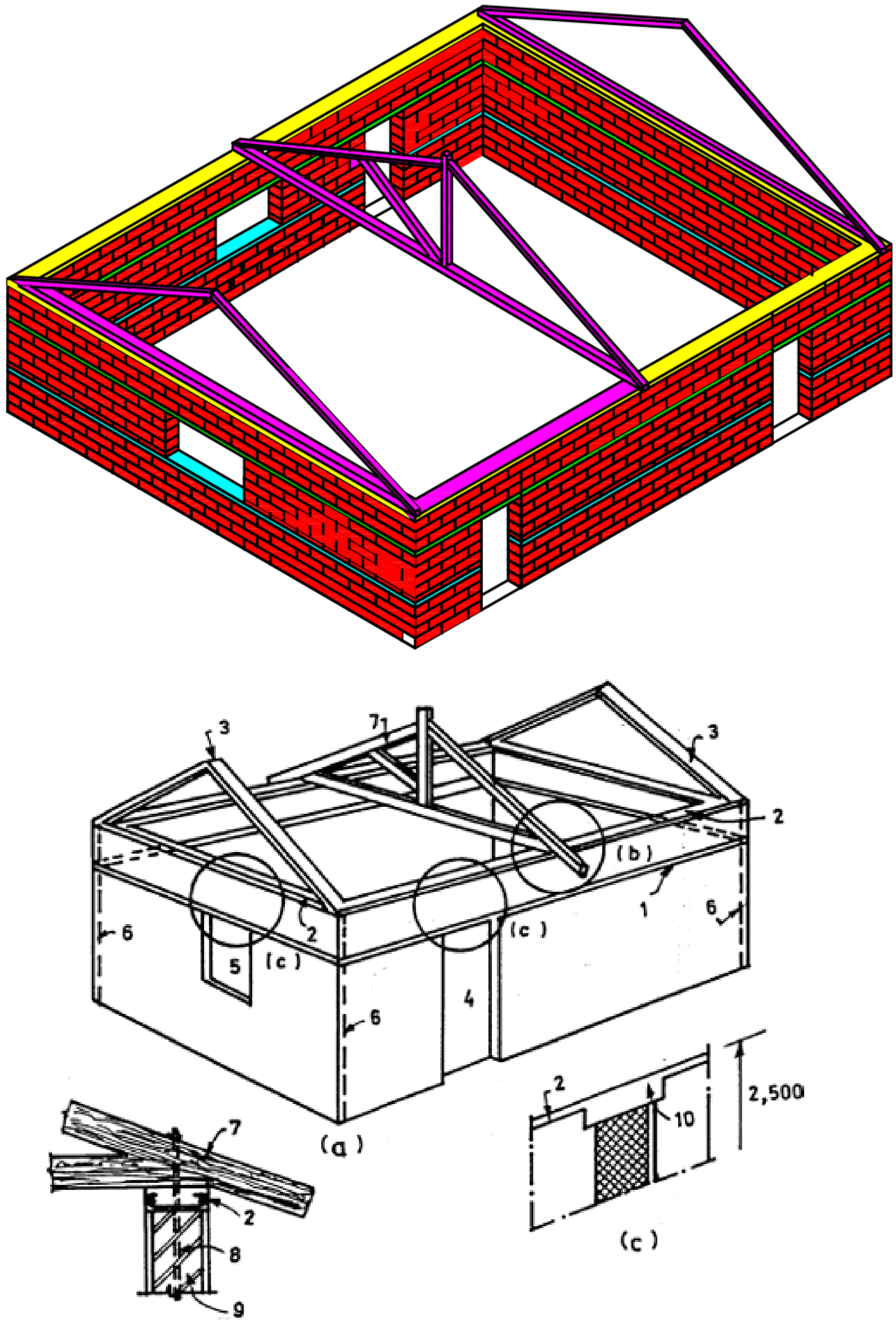


FIG. 26 OVERALL ARRANGEMENT OF REINFORCING MASONRY BUILDINGS



1 — Lintel band
2 — Eaves 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 integrated with roof band:

a) Perspective view
b) Details of truss connection with wall
c) Detail of integrating door lintel with roof band

FIG. 27 OVERALL ARRANGEMENT OF REINFORCING MASONRY BUILDING HAVING PITCHED ROOF

8.5.3.2 Lintel band shall be provided at the lintel level on all load bearing internal, external longitudinal and cross walls, as per **8.5.3.5**. This band provided in panel or partition walls improves their stability during strong earthquake shaking.

8.5.3.3 Roof band shall be provided immediately below the roof or floors. The specifications of the band are given in **8.5.3.5**. 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.

8.5.3.4 Gable band is provided at the top of gable masonry below the purlins. The specifications of the band are given in **8.5.3.5**. This band shall be made continuous with the roof band at the eaves level.

8.5.3.5 *Section and Reinforcement of Band*

8.5.3.5.1 The band shall be as per **8.4.2**.

8.5.3.5.2 The bands shall be of the full width of the wall not less than 75 mm in depth and reinforced with longitudinal steel (Table 19). 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 19.

a) Span of wall shall be the distance between centerlines of its cross walls or buttresses. For spans greater than 8 m intermediate pilasters or buttresses shall be provided or the adequacy of such long-span walls shall be ascertained through detailed analysis and full-scale test.

b) The numbers and diameter of bars given above pertain to high strength deformed bars.

c) The geometry of the bands shall be as per **8.4.2.2**.

d) Concrete used in the bands shall be as per **8.3.4**.

e) The longitudinal steel bars shall be held in position by steel links or stirrups 6 mm diameter spaced at maximum 150 mm apart.

8.5.3.5.3 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 at least equal to that specified in Table 19.

Table 19 Recommended Longitudinal Steel in Reinforced Concrete Bands
(*Clause* 8.5.3.5.2 and 8.5.3.5.3)

Sl No.	Span	Type of Building							
		B		C		D		E	
		Number of Bars	Diameter	Number of Bars	Diameter	Number of Bars	Diameter	Number of Bars	Diameter
	(m)		(mm)		(mm)		(mm)		(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

8.5.3.5.4 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. 28 are recommended.

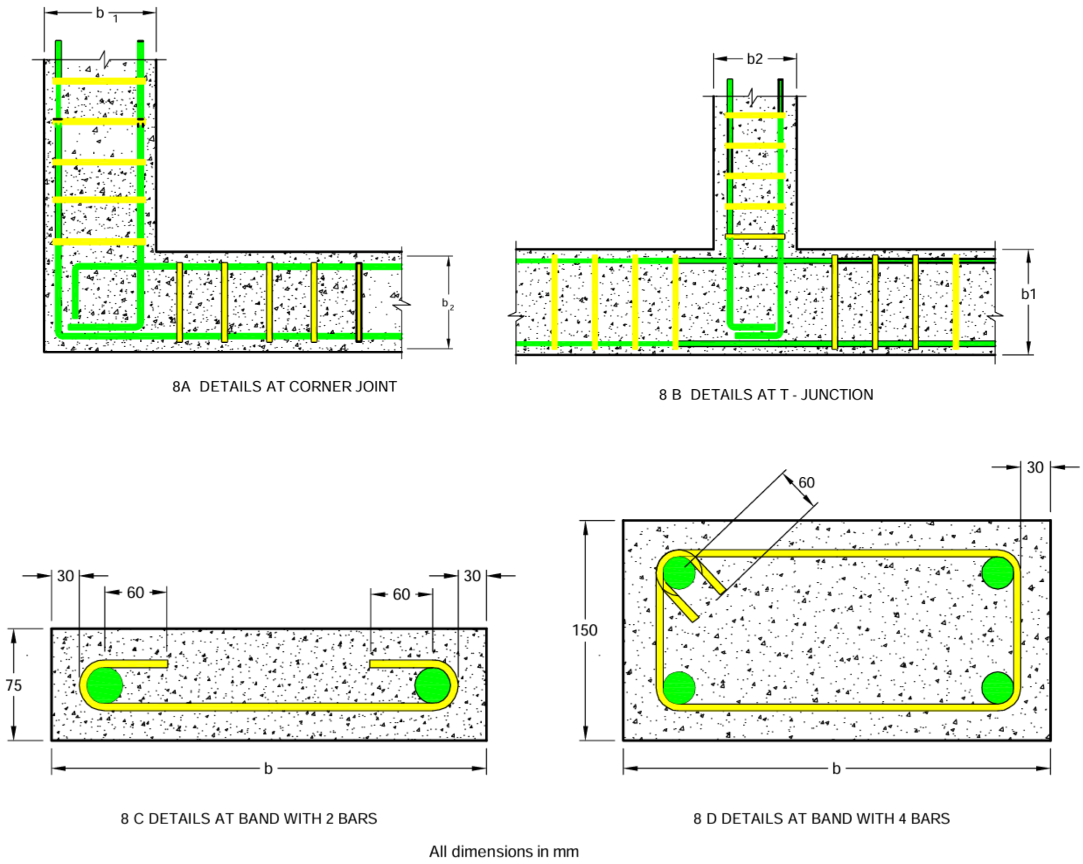


FIG. 28 REINFORCEMENT AND BENDING DETAIL IN RC BAND

8.5.3.6 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. Where used, its section may be kept the same as in **8.5.3.5.** This band shall serve as damp proof course as well.

8.5.3.7 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.

8.5.3.8 *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 20. For walls thicker than 340 mm, the area of the bars shall be proportionately increased.

With respect to Table 20,

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. 29. The vertical bars shall be covered with concrete of grade M20 or 1:3 mortar in suitably created pockets around the bars (Fig. 29). This shall ensure their safety from corrosion and good bond with masonry.

Table 20 Vertical Steel Reinforcement in Masonry Walls with Rectangular Masonry Units
(*Clause* 8.5.3.9)

<table>
<tr><th rowspan="2">Sl No.</th><th rowspan="2">Number of Storeys</th><th rowspan="2">Storey</th><th colspan="4">Diameter (in mm) of HSD Single Bar at each Critical Section</th></tr>
<tr><th>Type B</th><th>Type C</th><th>Type D</th><th>Type E</th></tr>
<tr><td>(1)</td><td>(2)</td><td>(3)</td><td>(4)</td><td>(5)</td><td>(6)</td><td>(7)</td></tr>
<tr><td>i)</td><td>1</td><td>-</td><td>Nil</td><td>Nil</td><td>10</td><td>12</td></tr>
<tr><td rowspan="2">ii)</td><td rowspan="2">2</td><td>Top</td><td>Nil</td><td>Nil</td><td>10</td><td>12</td></tr>
<tr><td>Bottom</td><td>Nil</td><td>Nil</td><td>12</td><td>16</td></tr>
<tr><td rowspan="3">iii)</td><td rowspan="3">3</td><td>Top</td><td>Nil</td><td>10</td><td>10</td><td>12</td></tr>
<tr><td>Middle</td><td>Nil</td><td>10</td><td>12</td><td>16</td></tr>
<tr><td>Bottom</td><td>Nil</td><td>12</td><td>12</td><td>16</td></tr>
<tr><td rowspan="4">iv)</td><td rowspan="4">4</td><td>Top</td><td>10</td><td>10</td><td>10</td><td rowspan="4">Buildings with 4 storeys not permitted.</td></tr>
<tr><td>Third</td><td>10</td><td>10</td><td>12</td></tr>
<tr><td>Second</td><td>10</td><td>12</td><td>16</td></tr>
<tr><td>Bottom</td><td>12</td><td>12</td><td>20</td></tr>
</table>

8.5.3.8.1 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.

8.5.3.8.2 Bars in different storeys shall be lapped as per the requirements of IS 456.

8.5.3.9 Vertical reinforcement of jambs of window and door openings shall be provided as per Table 20. It may start from the foundation of floor and terminate in lintel band (Fig. 9).

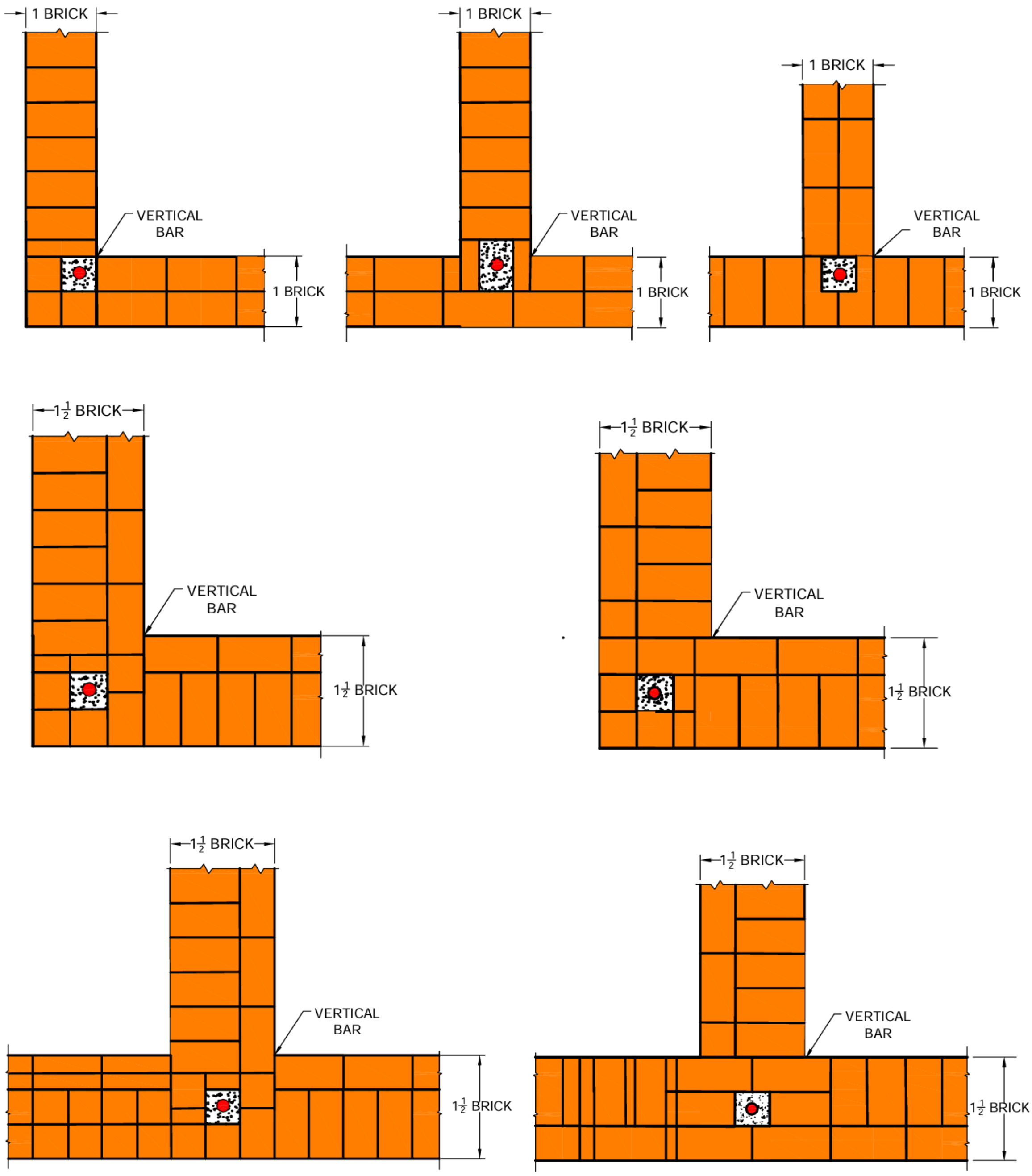


FIG. 29 TYPICAL DETAILS OF PROVIDING VERTICAL STEEL BARS IN BRICK MASONRY

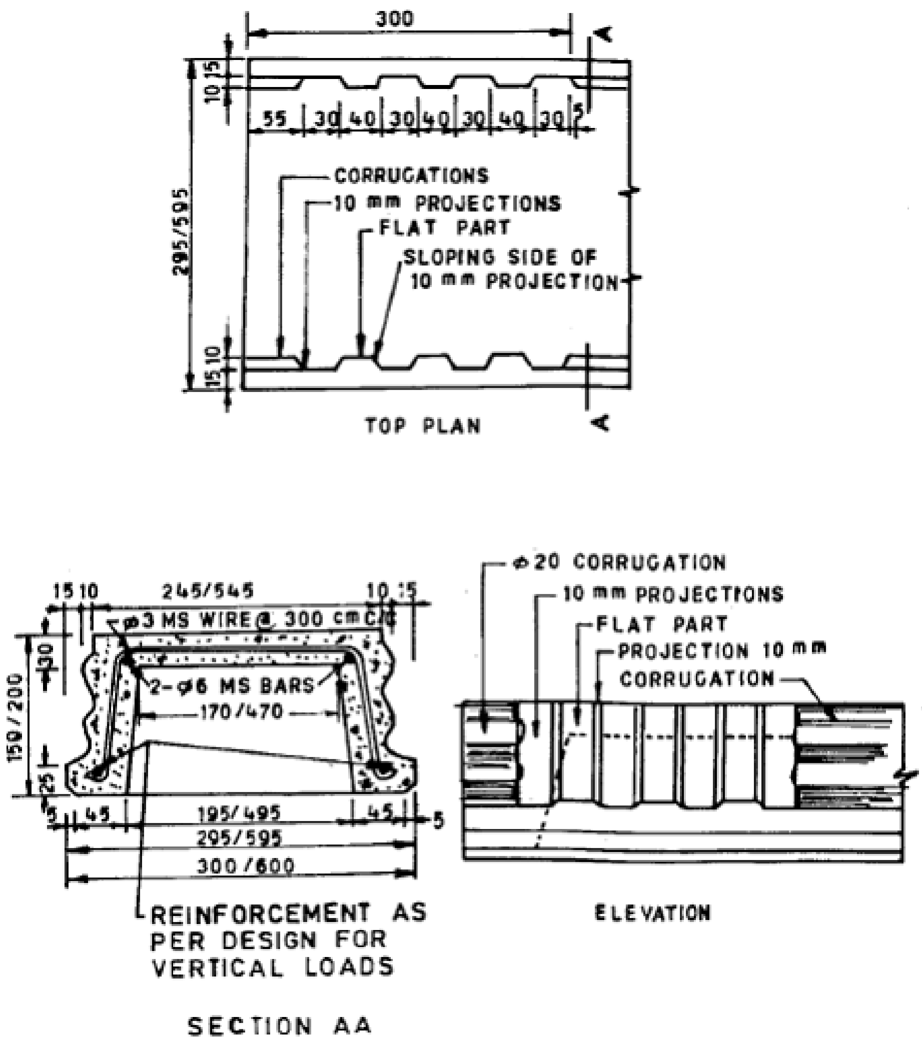
8.7 Floors/ Roofs with Small Precast Components

8.7.1 *Types of Precast Floors/Roofs*

Earthquake resistance measures for floors and roofs with small precast components, as covered in this Section, have been dealt with as typical examples.

8.7.1.1 *Precast reinforced concrete unit roof/floor*

The unit is a precast reinforced concrete component, channel (inverted trough) shaped in section (*see* Fig. 30). The nominal width of the unit varies from 300 to 600 mm, its height from 150 to 200 mm and a minimum flange thickness of 30 mm. Length of unit shall vary according to room dimensions, but the maximum length is restricted to 4.2 m from stiffness considerations. Horizontal corrugations are provided on the two longitudinal faces of the units so that the structural roof/floor acts monolithic after concrete grouted in the joints between the units attains strength (*see* Fig. 31).



All dimensions in millimetres.

FIG. 30 CHANNEL UNITS

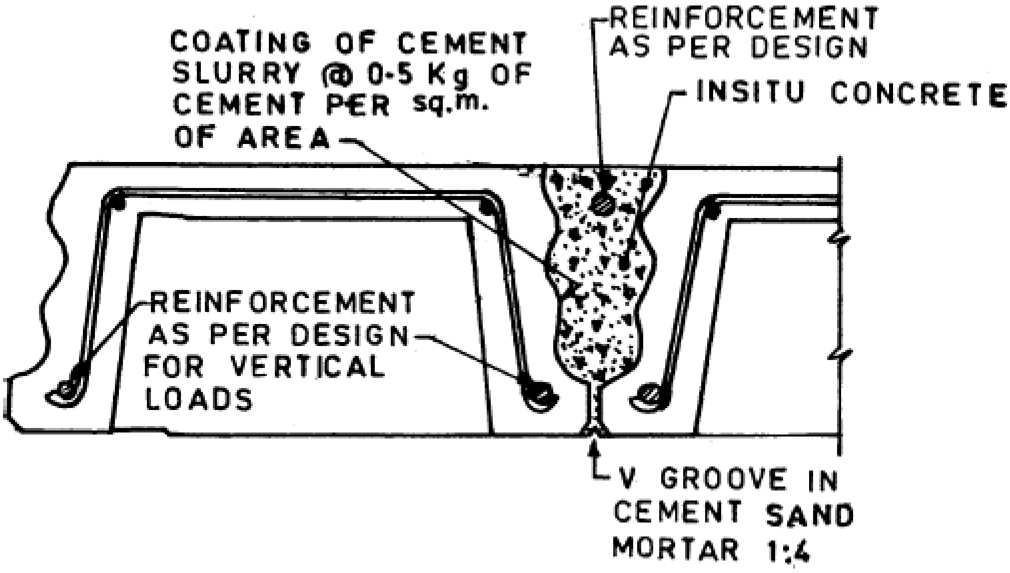
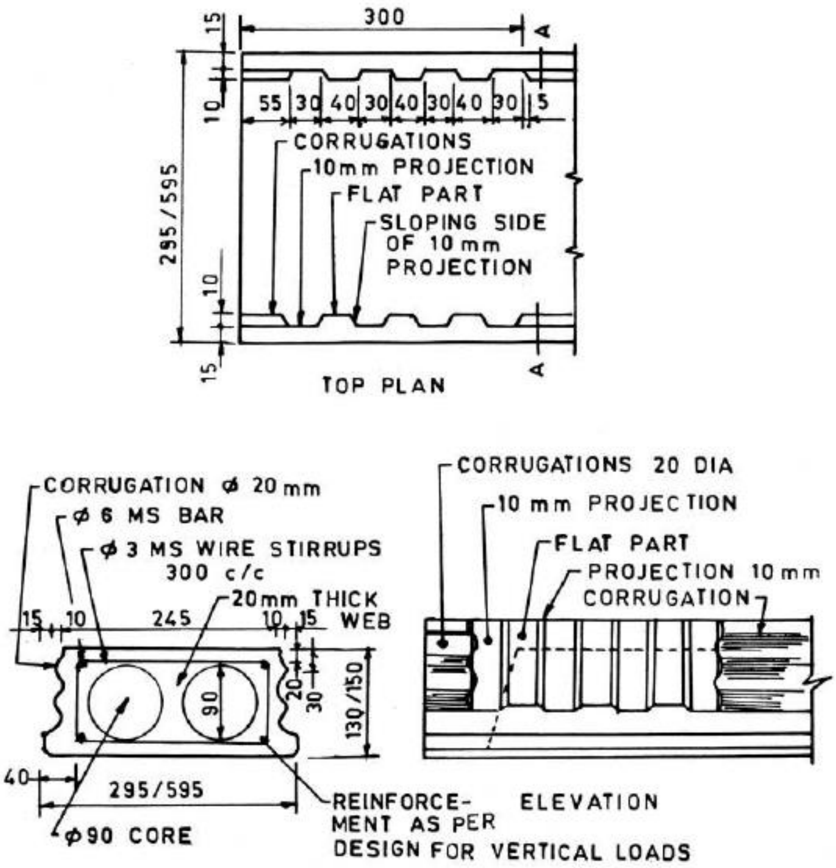


FIG. 31 CHANNEL UNITS

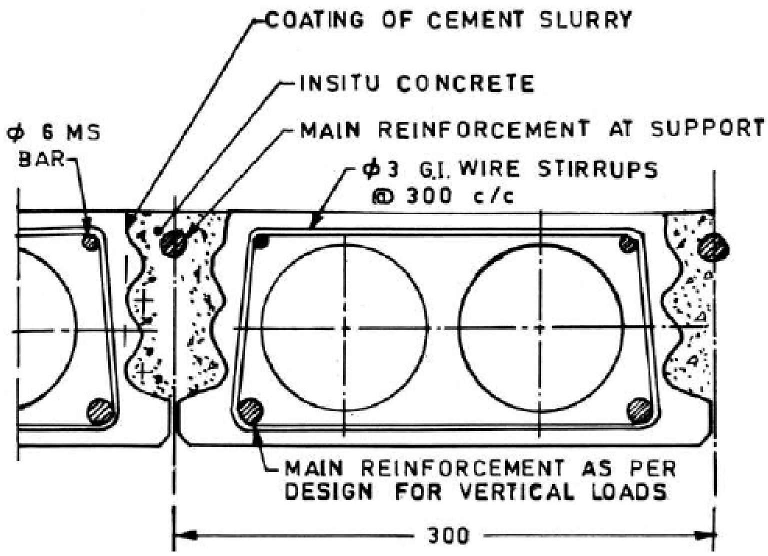
8.7.1.2 *Precast reinforced concrete cored unit roof/floor*

The unit is a reinforced concrete component having a nominal width of 300 to 600 mm and thickness of 130 to 150 mm having two circular hollows 90 mm diameter, throughout the length of the unit (*see* Fig. 32). The minimum flange/web thickness of the unit shall be 20 mm. Length of unit varies according to room dimensions, but the maximum length shall be restricted to 4.2 m from stiffness considerations. Horizontal corrugations are provided on the two longitudinal faces of the units so that the structural roof/floor acts monolithic after concrete grouted in the joints between the units attains strength (*see* Fig. 33).



All dimensions in millimetres.

FIG. 32 CORE UNITS

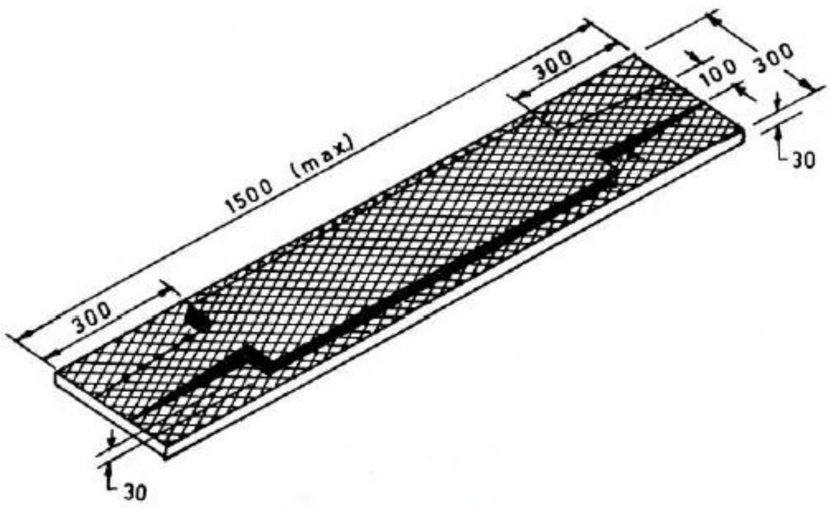


All dimensions in millimetres.

FIG. 33 CORED, UNIT FLOOR

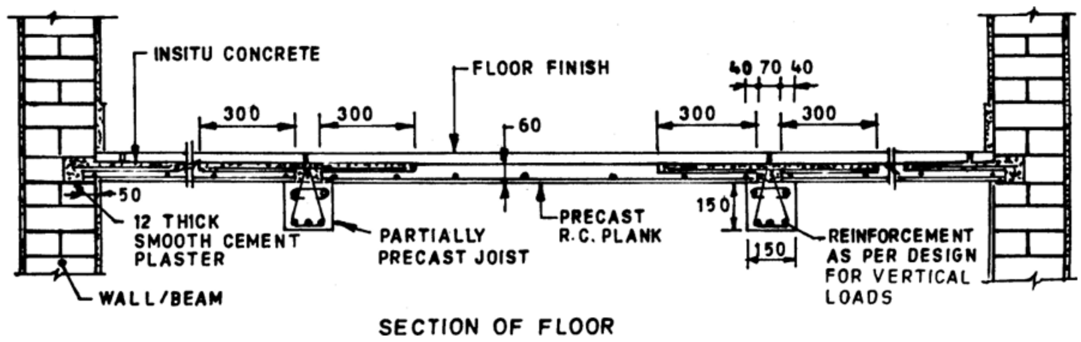
8.7.1.3 *Precast reinforced concrete plank and joist scheme for roof/floor*

The scheme consists of precast reinforced concrete planks supported on partially precast reinforced concrete joists. The reinforced concrete planks are 300 mm wide and the length varies according to the spacing of the joists, but it shall not exceed 1.5 m (*see* Fig. 34). To provide monolithic property to the roof/floor and to have T-beam effect with the joists, the planks shall be made partially 30 mm thick and the partially 60 mm thick and *in-situ* concrete shall be filled in the depressed portions to complete the roof/floor structurally (*see* Fig. 35).



All dimensions in millimetres.

FIG. 34 PRECAST REINFORCED PLANK



All dimensions in millimeteres

FIG. 35 PRECAST REINFORCED PLANK FLOOR

8.7.1.4 *Prefabricated brick panel system for roof/floor*

It consists of prefabricated reinforced brick panels (*see* Fig. 36) supported on Precast reinforced concrete joists with nominal reinforced 35 mm thick structural deck concrete over the brick panels and joists (*see* Fig. 37). The width of the brick panels shall be 530 mm for panels made of bricks of conventional size and 450 mm for panels made of bricks of modular size. The thickness of the panels shall be 75 mm or 90 mm, respectively depending upon whether conventional or modular bricks are used. The length of the panels shall vary depending upon the spacing of the joists, but the maximum length shall not exceed 1.2 m.

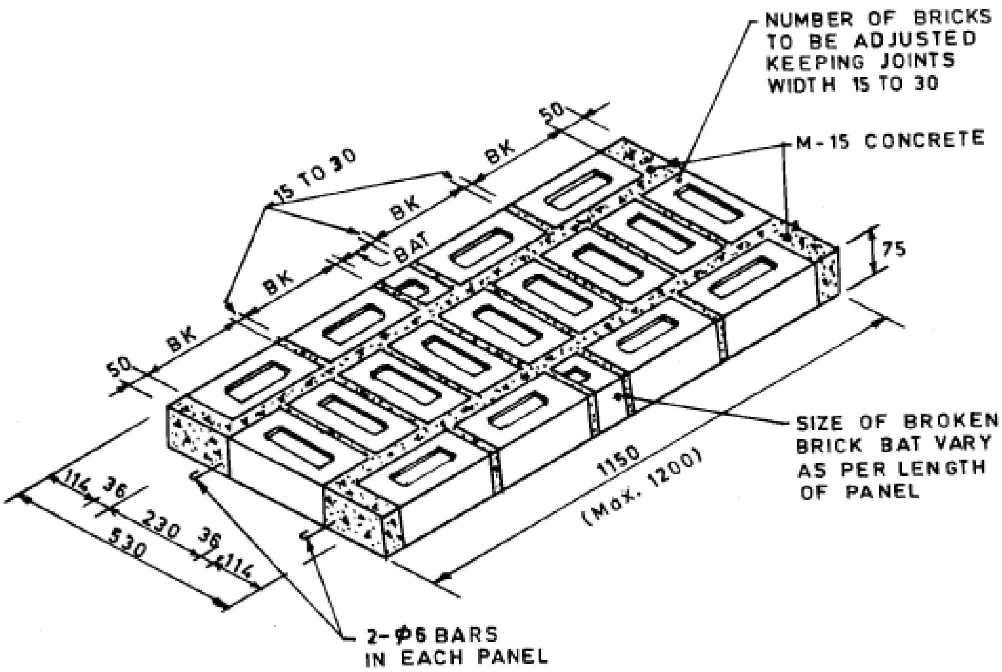


FIG. 36 PREFAB BRICK PANEL

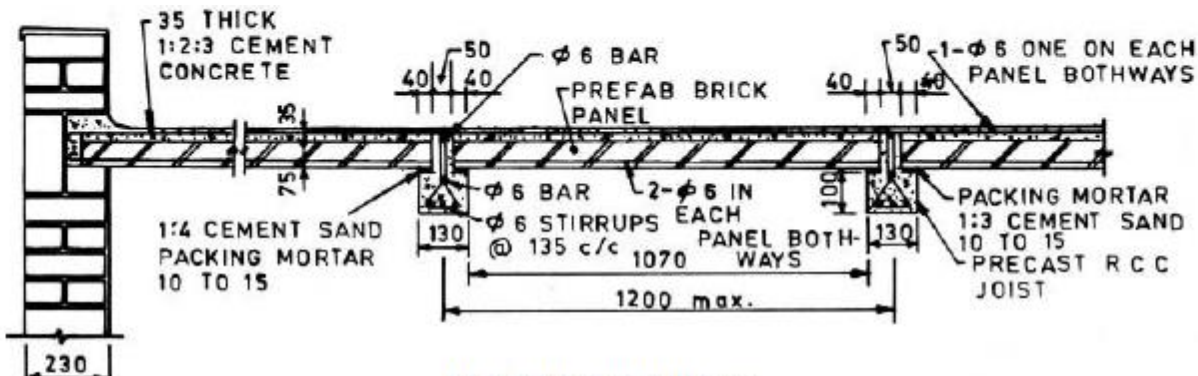
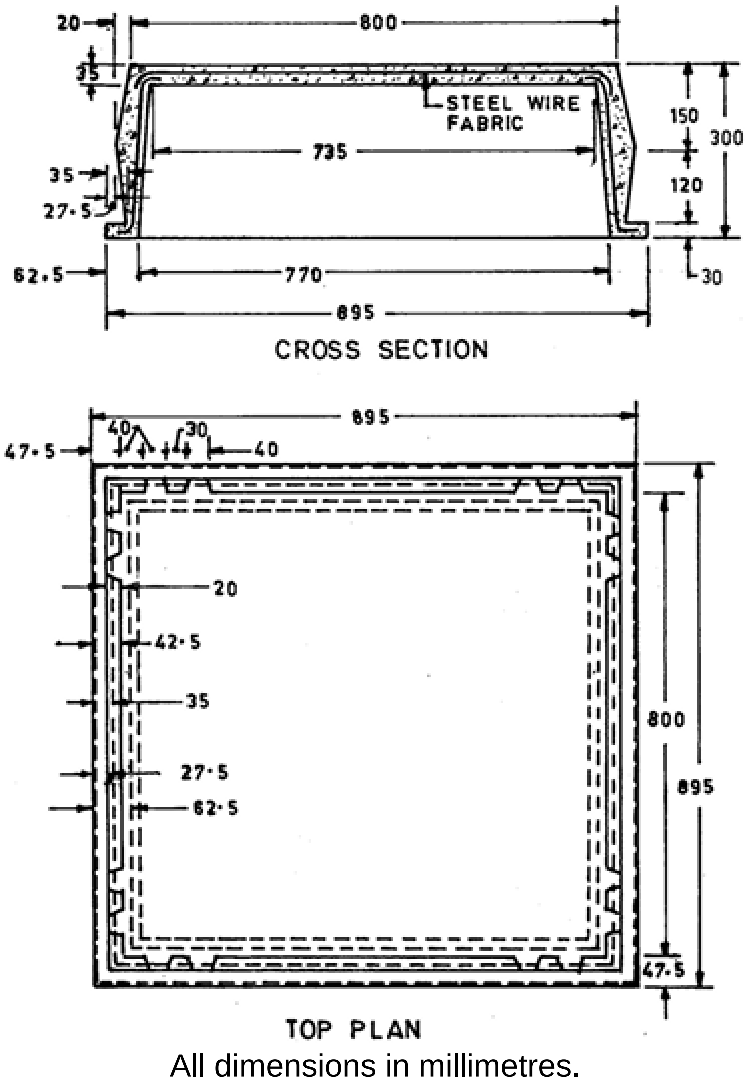


FIG. 37 BRICK PANEL FLOOR

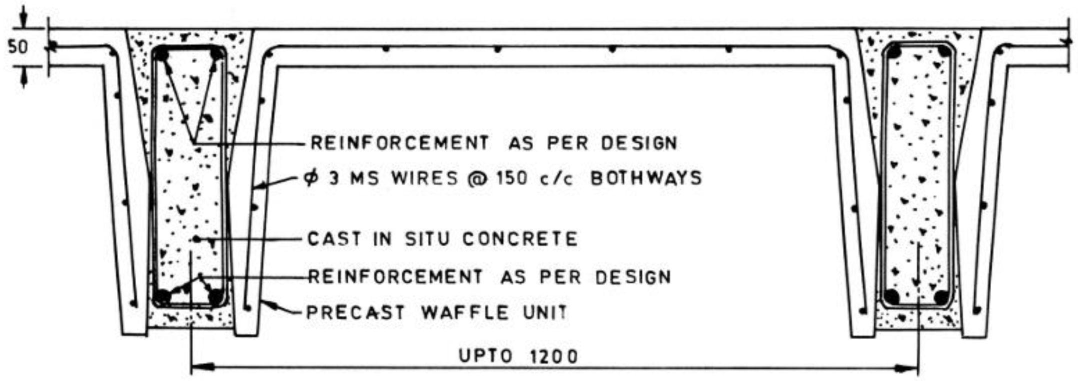
8.7.1.5 *Precast reinforced concrete waffle unit roof/floor*

Waffle units are of the shape of inverted troughs, square or rectangular in plan, having lateral dimensions up to 1.2 m and depth depending upon the span of the roof/floor to be covered (*see* Fig. 38 and Fig. 39). The minimum thickness of flange/web shall be 35 mm. Horizontal projections may be provided on all the four external faces of the unit and the unit shall be so shaped that it shall act monolithic with *in-situ* concrete to ensure load transfer. Vertical castellation, called shear keys, shall be provided on all the four external faces of the precast units to enable them to transfer horizontal shear force from one unit to adjacent unit through *in-situ* concrete filled in the joints between the units. The waffle units shall be laid in a grid pattern with gaps between two adjacent units, and reinforcement, as per design, and structural concrete shall be provided in the gaps between the units in both the directions. The scheme is suitable for two way spanning roofs and floors of buildings having large spans.



All dimensions in millimetres.

FIG. 38 WAFFLE UNITS



All dimensions in millimetres.

FIG. 39 WAFFLE UNIT FLOOR

8.7.2 *Seismic Resistance Measures*

8.7.2.1 All floors and roofs to be constructed with small precast components shall be strengthened as specified for various categories of buildings in Table 21. The strengthening measures are detailed in **8.7.2.3** and **8.7.2.8**.

8.7.2.2 Vertical castellation, called shear keys, shall be provided on the longitudinal faces of the channel, cored and waffle units to enable them to transfer horizontal shear force from one unit to the adjacent unit through the *in-situ* concrete filled in the joints between the units. The minimum percentage of area of shear keys as calculated below, on each face of the unit, shall be 15.

Shear keys shall have a minimum width of 40 mm at its root with the body of the component and shall be to the full height of the component and preferably at uniform spacing. Percentage of area of shear keys shall be calculated as:

$$\frac{\text{No. of shear keys on one face of the component x 40}}{\text{Length of the face of the component, in mm}} \text{ x } 100$$

Table 21 Strengthening Measures for Floors/Roofs with Small Precast Components

(*Clauses* 8.7.2.1 *and* 8.7.2.8)

Sl No.	**Building Category**	**Number of Storeys**	**Strengthening to be Provided in Floor/Roof with**			
			Channel/Cored Unit	R.C. Planks and Joists	Brick Panels and Joists	Waffle Units
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	B	1 to 3	a	a	a	a
		4	a, c	a, c	a, d	a
ii)	C	1 and 2	a, b	a	a	a
		3 and 4	a, b, c	a, c	a, d	a, e

Sl No.	Building Category	Number of Storeys	Strengthening to be Provided in Floor/Roof with			
			Channel/Cored Unit	R.C. Planks and Joists	Brick Panels and Joists	Waffle Units
(1)	(2)	(3)	(4)	(5)	(6)	(7)
iii)	D	1 - 4	a, b c	a, c	a, d	a, c, e
iv)	E	1 - 3	a, b, c	a, c	a, d	a, c, e

NOTE

a - Tie beam as per **8.7.2.3**;
b - Reinforcing bars of precast unit and tied to tie beam reinforcement as per **8.7.2.4**;
c - Reinforced deck concrete as per **8.7.2.5**;
d - Reinforced deck concrete as per **8.7.2.6**; and
e - Reinforced bars in joint between precast waffle units tied to tie beam reinforcement as per **8.7.2.7**.

8.7.2.3 Tie beam (*see* Table 20) is a beam provided all round the floor or roof to bind together all the precast components to make it a diaphragm. The beams shall be to the full width of the supporting wall or beam less the bearing of the precast components. The depth of the beam shall be equal to the depth of the precast components plus the thickness of structural deck concrete, where used over the components. The beam shall be made of cement concrete of grade not leaner than M15 and shall be reinforced as indicated in Table 18. If depth of tie is more than 75 mm, equivalent reinforcement shall be provided with one bar of minimum diameter 8 mm at each corner. Tie beams shall be provided on all longitudinal and cross walls. Typical details of the beams are shown in Fig. 40 to Fig. 44.

NOTE – Adequate edge support say 60 mm, shall be provided to precast element on the wall so as to avert its slippage during seismic ground motion.

8.7.2.4 Top reinforcement in the channel or cored units (*see* Table 21) shall be projected out at both the ends for anchorage length and tied to tie beam reinforcement.

8.7.2.5 Structural deck concrete (*see* Table 21) of grade not leaner than M15 shall be provided over precast components or act monolithic. Wherever, deck concrete is to be provided, the top surface of the components shall be finished rough. Cement slurry with 0.5 kg of cement/m^2 of the surface area shall be applied over the components immediately before laying the deck concrete and the concrete shall be compacted using plate vibrators. The minimum thickness of deck concrete shall be 35 mm or 40 mm reinforced with 6 mm diameter bars and 150 mm apart both ways and anchored into the tie beam placed all round. The maximum size of coarse aggregate used in deck concrete shall not exceed 12 mm.

NOTE – Under conditions of economic constraints, the deck concrete itself could serve as floor finish. The concrete is laid in one operation (*see* Fig. 44) without joints.

8.7.2.6 The deck concrete normally used over the brick panel with joist floor shall be reinforced with 6 mm diameter bars spaced 150 mm apart both ways (*see* Table 21)

8.7.2.7 For floors/roofs with precast waffle units, two 16 mm diameter high strength deformed bars shall be provided as top reinforcement in the joints between waffle units, in addition to reinforcement required for taking bending moment for vertical loads. This reinforcement (*see* Table 21) shall be fixed to tie beam reinforcement.

8.7.2.8 In case of floors/roofs with precast components other than those indicated in Table 21, the buildings shall be analysed for maximum expected seismic forces and the floor/roof shall be designed to act as diaphragm and take care of the resulting forces.

8.8 Timber Construction

8.8.1 Timber has higher strength per unit weight and is, therefore, very suitable for earthquake resistant construction. Materials, design and construction in timber shall generally conform to good practice [6-4(17)].

8.8.2 Timber (with masonry in-fill) construction shall generally be restricted to two storeys with or without the attic floor.

8.8.3 In timber (with masonry in-fill) construction attention shall be paid to fire safety against electric short circuiting, kitchen fire, etc.

8.8.4 The superstructure of timber buildings shall be made rigid against deformations by adopting suitable construction details at the junctions of the framing members and in wall panels as given in **8.8.6** to **8.8.10** so that the construction as a whole behaves as one unit against earthquake forces.

8.8.5 *Foundations*

8.8.5.1 Timber construction shall preferably start above the plinth level, the portion below shall be in masonry or concrete.

8.8.5.2 The superstructure may be connected with the foundation in one of the two ways as given below:

a) The superstructure may simply rest on the plinth masonry, or in the case of small buildings of one storey having plan area less than about 50 m^2, it may rest on firm plane ground so that the building is free to slide laterally during ground motion.

NOTES

1 Past experience has shown that superstructure of the buildings not fixed with the foundation escaped collapse even in a severe earthquake although they were shifted sideways.

2 Where fittings for water supply or water borne sanitation from the house are to be installed, proper attention should be given to permit movement so as to avoid fracture of damage to pipes.

b) The superstructure may be rigidly fixed into the plinth masonry or concrete foundation as given in Fig. 45 or in case of small building having plan area less than 50 m^2, it may be fixed to vertical poles embedded into the ground. In each case the building is likely to move along with its foundation. Therefore, the superstructure shall be designed to carry the resulting earthquake shears.

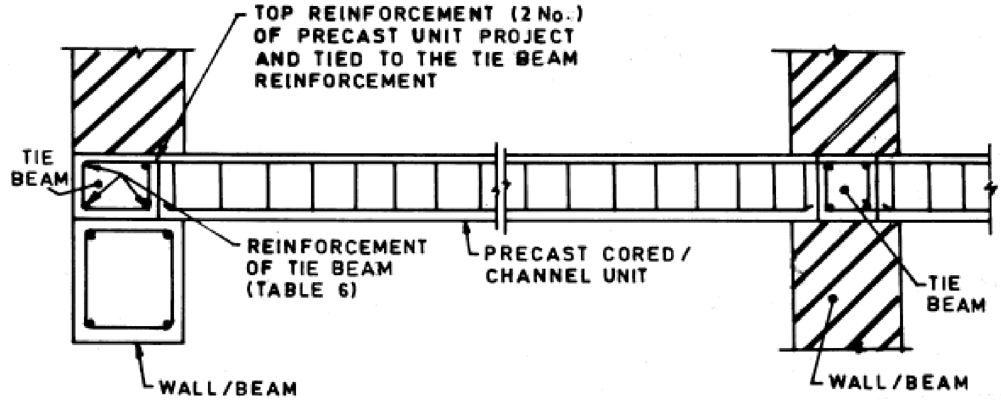


Fig. 40 CONNECTION OF PRECAST CORED/CHANNEL UNIT WITH TIE BEAM

8.8.6 *Types of Framing*

The types of construction usually adopted in timber building are as follows:

a) Stud wall construction; and
b) Brick nogged timber frame construction.

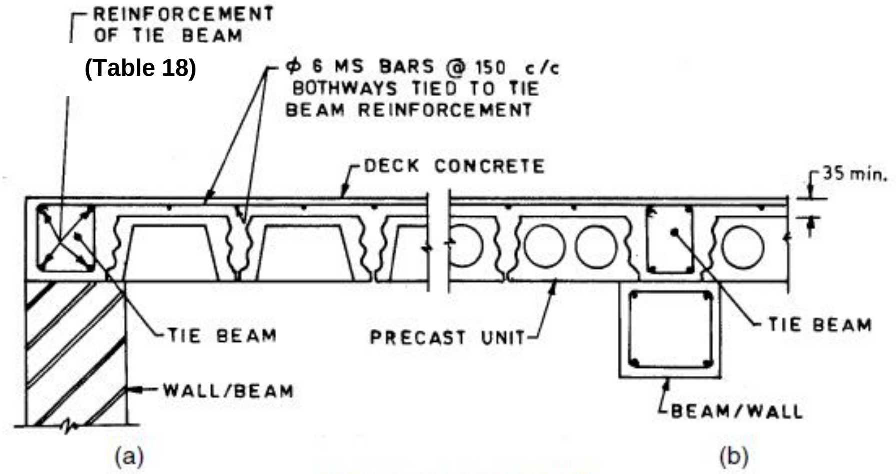
8.8.7 *Stud Wall Construction*

8.8.7.1 The stud wall construction consists of timber studs and corner posts framed into sills, top plates and wall plates. Horizontal struts and diagonal braces are used to stiffen the frame against lateral loads. The wall covering may consist of *EKRA*, timber or like. Typical details of stud walls are shown in Fig. 46. Minimum sizes and spacing of various members used are specified in **8.8.7.2** to **8.8.7.10**.

8.8.7.2 The timber studs for use in load bearing walls shall have a minimum finished size of 40 x 90 mm and their spacing shall not exceed those given in Table 22.

8.8.7.3 The timber studs in non-load bearing walls shall not be less than 40 x 70 mm in finished cross section. Their spacing shall not exceed 1 m.

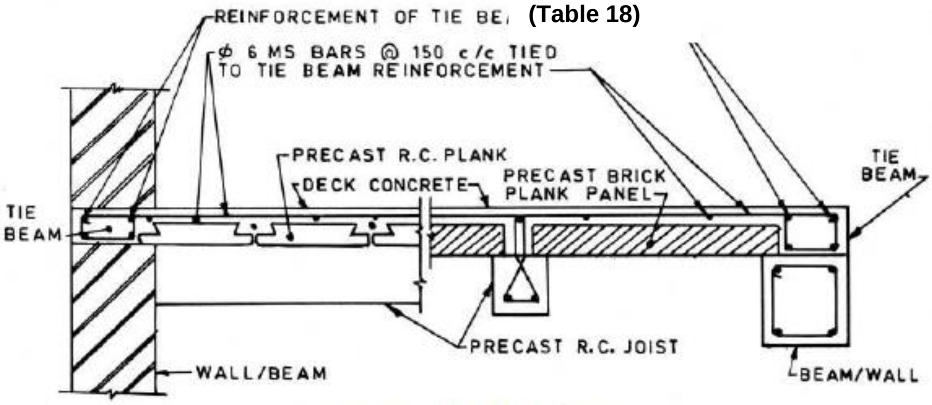
8.8.7.4 There shall be at least one diagonal brace for every 1.6 m x 1 m area of load bearing walls. Their minimum finished sizes shall be in accordance with Table 23.



a) Channel unit floor/roof.
b) Cored unit floor/roof.

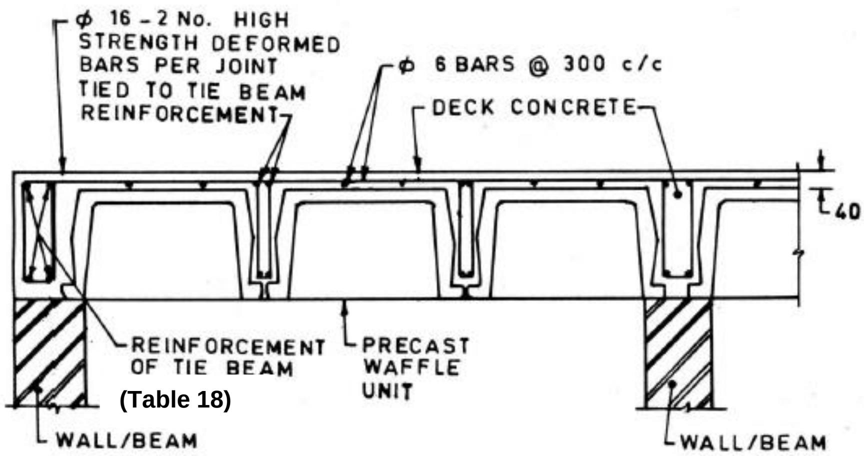
All dimensions in millimetres.

FIG. 41 CONNECTION OF CHANNEL/CORED UNIT FLOOR/ROOF (WITH DECK CONCRETE) WITH TIE BEAM



All dimensions in millimetres.

FIG. 42 CONNECTION OF PRECAST REINFORCED CONCRETE PLANK AND PRECAST BRICK PANEL FLOOR/ROOF (WITH DECK CONCRETE) WITH TIE BEAM



All dimensions in millimetres.

FIG.43 CONNECTION OF PRECAST WAFFLE UNIT FLOOR/ROOF (WITH DECK CONCRETE) WITH TIE BEAM

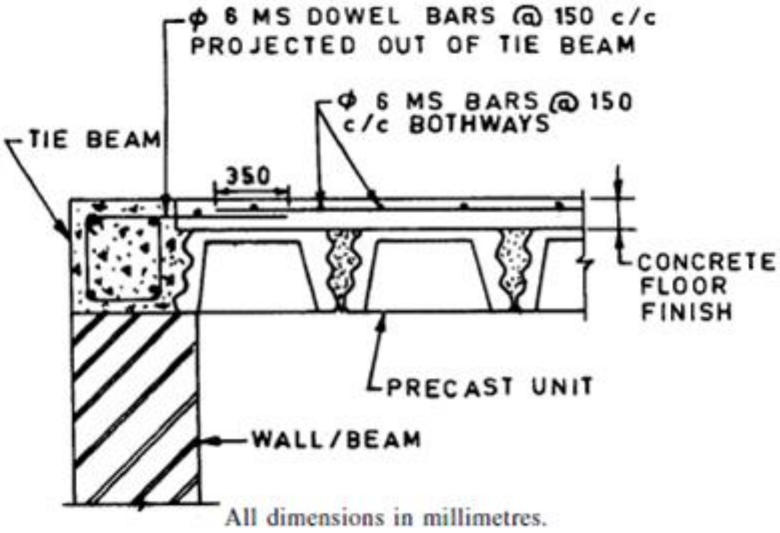
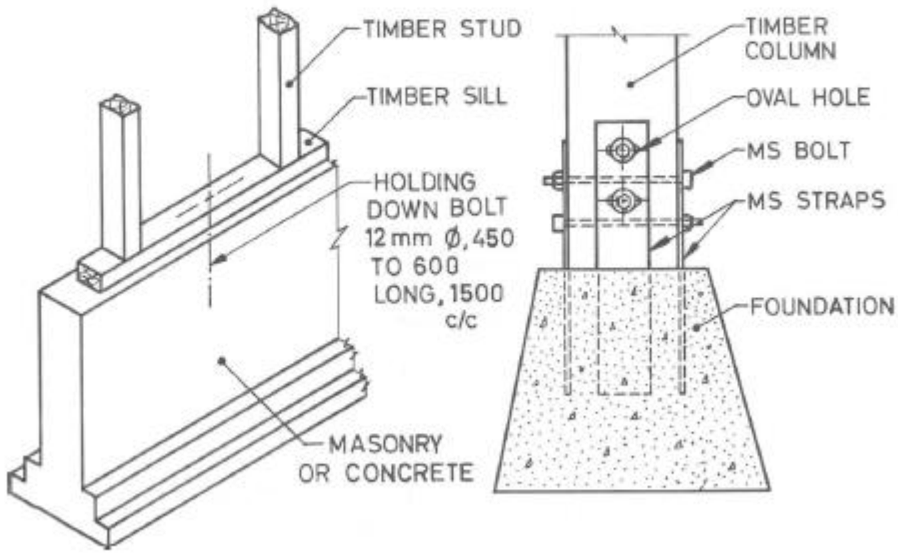


FIG. 44 PROVISION OF REINFORCEMENT IN CONCRETE FLOOR FINISH



45 A Suitable for Strip Foundation 45 B Suitable for Isolated Column Footings

All dimensions in millimetres.

FIG. 45 DETAILS OF CONNECTION OF COLUMN WITH FOUNDATION

Table 22 Maximum Spacing of 40 mm x 90 mm Finished Size Studs in Stud Wall Construction
(*Clause* 8.8.7.2)

Sl No.	Group of Timber (Grade I[1)])	Single Storeyed or First Floor of the Double Storeyed Buildings		Ground Floor of Double Storeyed Buildings	
		Exterior Wall mm	Interior Wall mm	Exterior Wall mm	Interior Wall mm
(1)	(2)	(3)	(4)	(5)	(6)
i)	Group A, B	1000	800	500	400
ii)	Group C	1000	1000	500	500

[1)]Grade I timbers as defined in Table 5 of [6-4(17)].

Table 23 Minimum Finished Sizes of Diagonal Braces
(*Clauses* 8.8.7.4, 8.8.8.2 *and* 8.8.8.4)

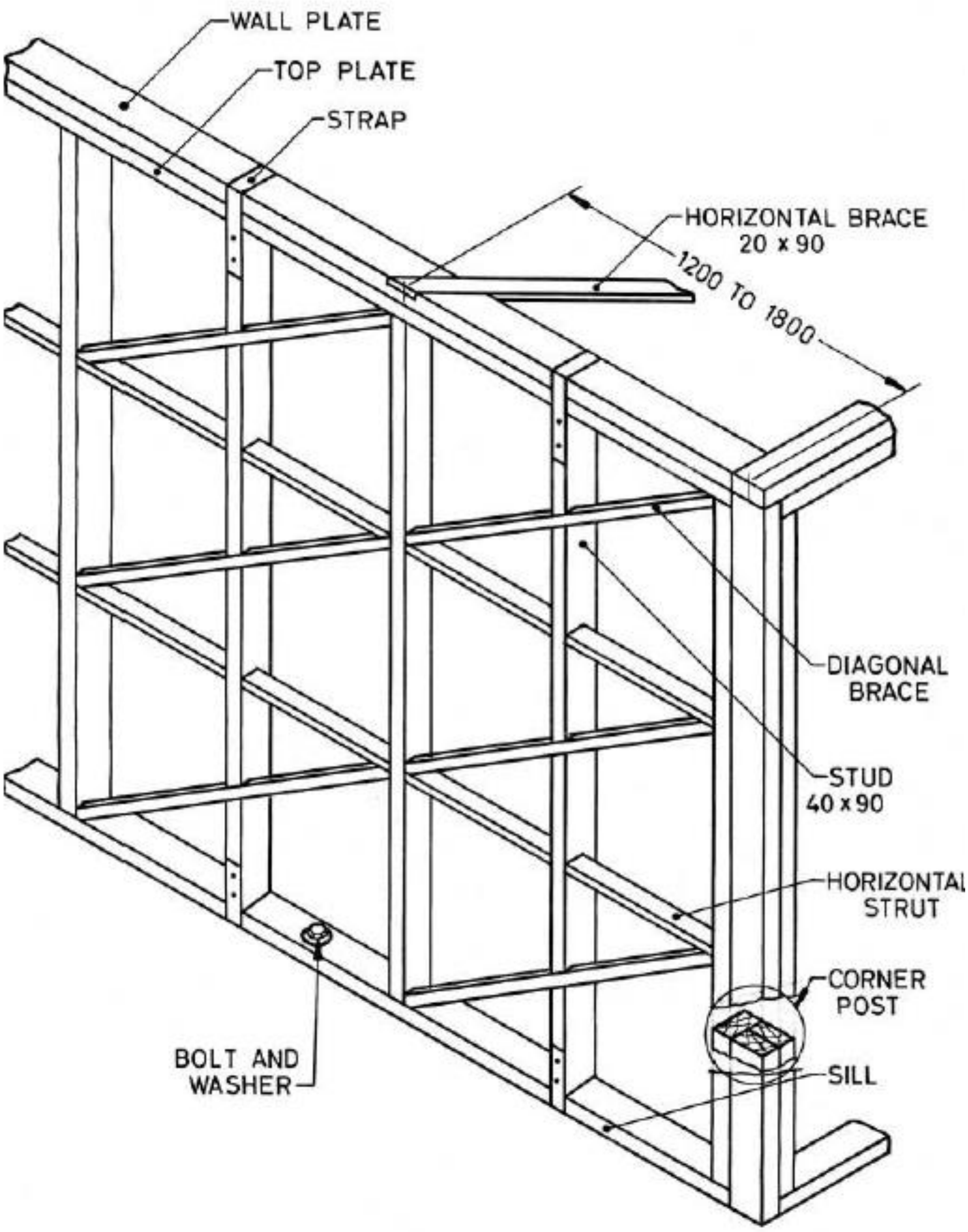
Sl No.	Building Category (*see* Table 2)	Group of Timber (Grade I[1)])	Single Storeyed or First Floor of Double Storeyed Buildings		Ground Floor of Double Storeyed Buildings	
			Exterior Wall mm x mm	Interior Wall mm x mm	Exterior Wall mm x mm	Interior Wall mm x mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)
i)	B, C	All	20 x 40	20 x 40	20 x 40	20 x 40
ii)	D and E	Group A and Group B	20 x 40	20 x 40	20 x 40	30 x 40
		Group C	20 x 40	30 x 40	30 x 40	30 x 40

[1)]Grade I timber as defined in Table 5 of [6-4(17)].

8.8.7.5 The horizontal struts shall be spaced not more than 1 m apart. They shall have a minimum size of 30 x 40 mm for all locations.

8.8.7.6 The finished sizes of the sill, the wall plate and top plate shall not be less than the size of the studs used in the wall.

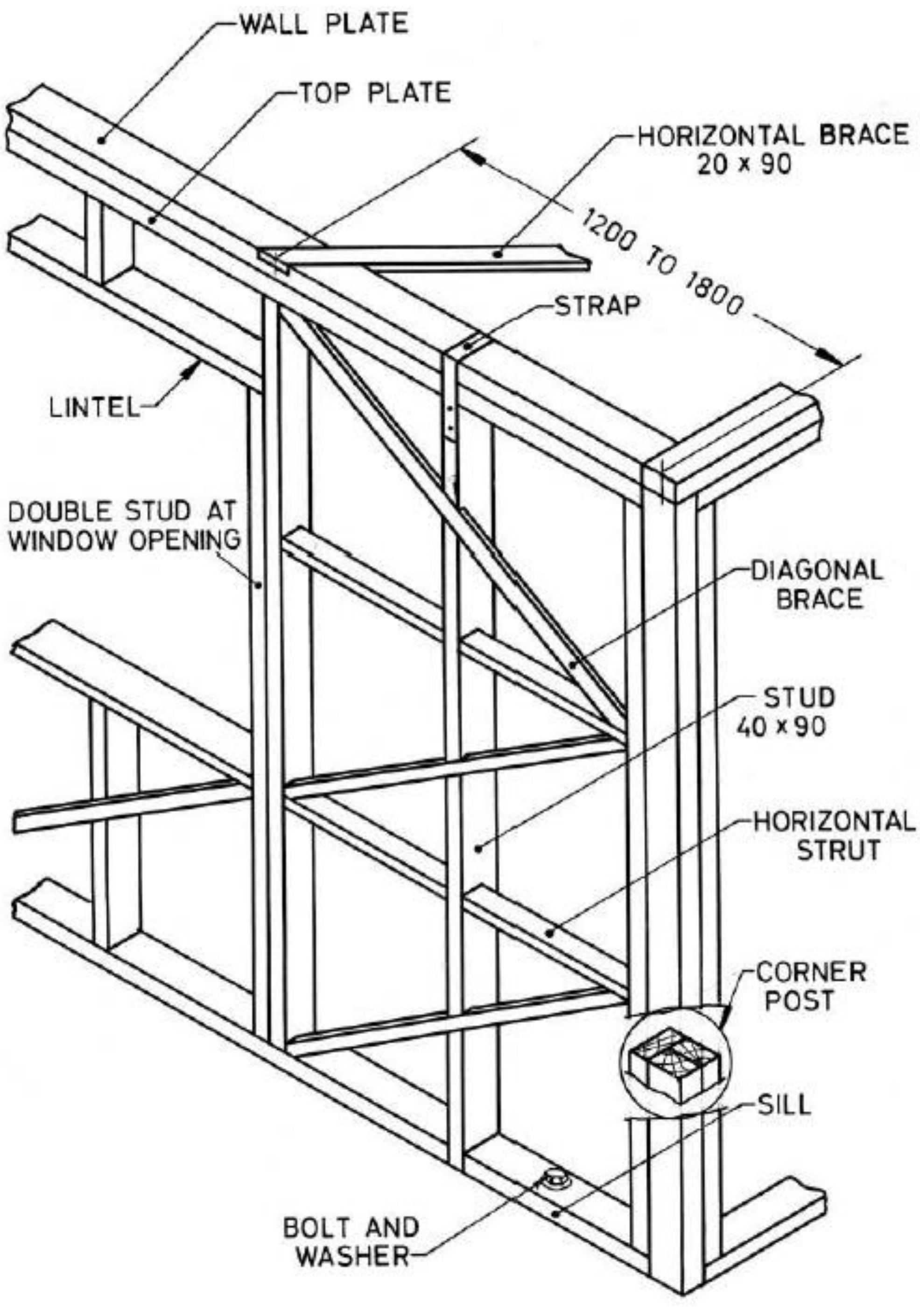
8.8.7.7 The corner posts shall consists of three timbers, two being equal in size to the studs used in the walls meeting at the corner and the third timber being of a size to fit so as to make a rectangular section (*see* Fig. 46).



Timber Framing in Stud Wall Construction without Opening in Wall

All dimensions in millimetres.

FIG. 46 STUD WALL CONSTRUCTION (*Continued*)



46 B Timber Framing in Stud Wall Construction with Opening in Wall

All dimensions in millimetres.

FIG. 46 STUD WALL CONSTRUCTION

8.8.7.8 The diagonal braces shall be connected at their ends with the stud wall members by means of wire nails having 6 gauge (4.88 mm diameter) and 100 mm length. Their minimum number shall be 4 nails for 20 mm x 40 mm braces and 6 nails for 30 mm x 40 mm braces. The far end of nails may be clutched as far as possible.

8.8.7.9 Horizontal bracing shall be provided at corners of T-junctions of walls at sill, first floor and eaves levels. The bracing members shall have a minimum finished size of 20 mm x 90 mm and shall be connected by means of wire nails to the wall plates at a distance between 1.2 m and 1.8 m measured from the junction of the walls. There shall be a minimum number of six nails of 6 gauge (4.88 mm diameter) and 10 cm length with clutching as far ends.

8.8.7.10 Unsheathed studding shall not be used adjacent to the wall of another building. The studding shall be sheathed with close jointed 20 mm or thicker boards.

8.8.8 *Brick Nogged Timber Frame Construction*

8.8.8.1 The brick nogged timber frame consists of intermediate verticals, columns, sills, wall plates, horizontal nogging members and diagonal braces framed into each other and the space between framing members filled with tight-fitting brick masonry in stretcher bond. Typical details of brick nogged timber frame construction are shown in Fig. 47. Minimum sizes and spacing of various elements used are specified in **8.8.8.2** to **8.8.2.9**.

8.8.8.2 The vertical framing members in brick nogged load bearing walls shall have minimum finished sizes as specified in Table 23.

8.8.8.3 The minimum finished size of the vertical members in non-load bearing walls shall be 40 mm x 100 mm spaced not more than 1.5 m apart.

8.8.8.4 The sizes of diagonal bracing members shall be the same as in Table 23.

8.8.8.5 The horizontal framing members in brick-nogged construction shall be spaced not more than 1 m apart. Their minimum finished sizes shall be in accordance with Table 24 and Table 25.

8.8.8.6 The finished sizes of the sill, wall plate and top plate shall be not less than the size of the vertical members used in the wall.

8.8.8.7 Corner posts shall consist of three vertical timbers as described in **8.8.7.7**.

8.8.8.8 The diagonal braces shall be connected of their ends with the other members of the wall by means of wire nails as specified in **8.8.7.8**.

8.8.8.9 Horizontal bracing members of corners of T-junctions of wall shall be as specified in **8.8.7.9**.

Table 24 Minimum Finished Sizes of Vertical in Brick Nogged Timber Frame Construction
(*Clauses* 8.8.8.5)

Sl No.	**Spacing**	**Group of Timber (Grade I[1])**	**Single Storeyed or First Floor of Double Storeyed Buildings**		**Ground Floor of Double Storeyed Buildings**	
			Exterior Wall	Interior Wall	Exterior Wall	Interior Wall
(1)	(2) m	(3)	(4) mm x mm	(5) mm x mm	(6) mm x mm	(7) mm x mm
i)	1	Group A, B	50 x 100	50 x 100	50 x 100	50 x 100
		Group C	50 x 100	70 x 100	70 x 100	90 x 100
ii)	1.5	Group A, B	50 x 100	70 x 100	70 x 100	80 x 100
		Group C	70 x 100	80 x 100	80 x 100	100 x 100

[1]Grade I timber as defined in Table 5 of [6-4(17)].

Table 25 Minimum Finished Size of Horizontal Nogging Members
(*Clause* 8.8.8.5)

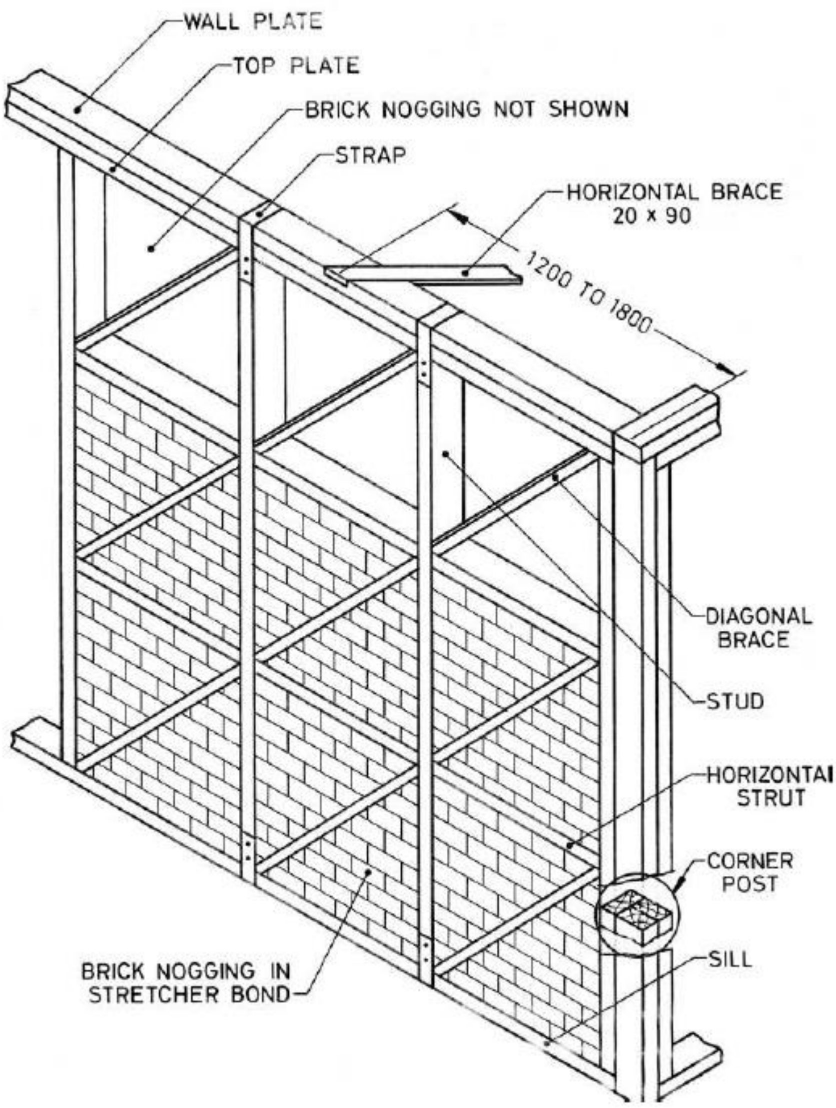
Sl No.	**Spacing of Verticals** m	**Size** mm
(1)	(2)	(3)
i)	1.5	70 x 100
ii)	1	50 x 100
iii)	0.5	25 x 100

8.8.9 *Notching and Cutting*

8.8.9.1 Timber framing frequently requires notching and cutting of the vertical members. The notching or cutting should in general be limited to 20 mm in depth unless steel strips are provided to strengthen the notched face of the members. Such steel strips, where necessary shall be at least 1.5 mm thick and 35 mm wide extending at least 150 mm beyond each side of the notch or cut and attached to the vertical member by means of bolts or screws at each end.

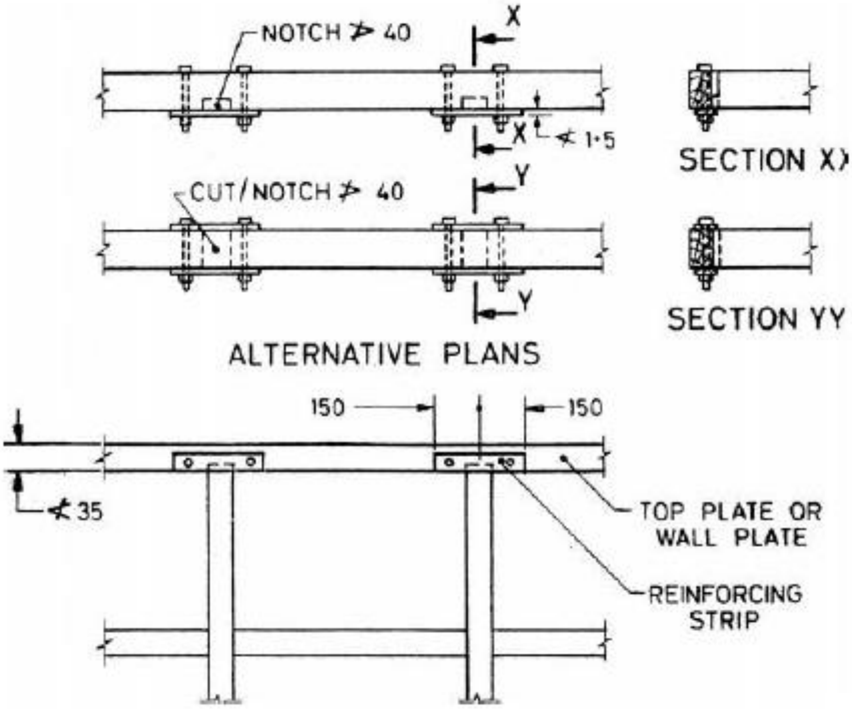
8.8.9.2 The top plate, the wall plate or the sill of a wall may be notched or cut, if reinforcing strip of iron is provided as specified in **8.8.9.1**. In case the member is notched or cut not to exceed 40 mm in depth, such reinforcing strip may be placed along the notched edge only. Where the notch or cut is more than 40 mm in depth or the member is completely cut through, such reinforcing strips shall be placed on both edges of the member. The details of notching and cutting are shown in Fig. 48.

8.8.9.3 Joints in timber shall preferably be bound by metallic fasteners.



All dimensions in millimetres.

FIG. 47 BRICK NOGGED TIMBER FRAME CONSTRUCTION



All dimensions in millimetres.
FIG. 48 NOTCHING AND CUTTING

8.8.10 *Bridging and Blocking*

8.8.10.1 All wooden joists shall have at least one row of cross bridging for every 3.5 m length of span. The cross-section of the bridging member shall be a minimum of 40 x 70 mm and the member shall be screwed or nailed to the joists.

8.8.10.2 All spaces between joists shall be blocked at all bearing with solid blocks not less than 40 mm thick and the full depth of the joists. The block shall be screwed or nailed to the joists as well as to the bearings.

9 CONFINED MASONRY WALLS (CMW)

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 **5**.

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 during an earthquake.

d) There shall be at least two lines of walls in each orthogonal direction of the building plan, and the walls along each line shall extend over at least 50 percent of the building dimension in the direction of analysis at each storey level (Fig. 49).

e) The walls should always be continuous up to the building height – there shall be no vertical offsets along the building height.

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.

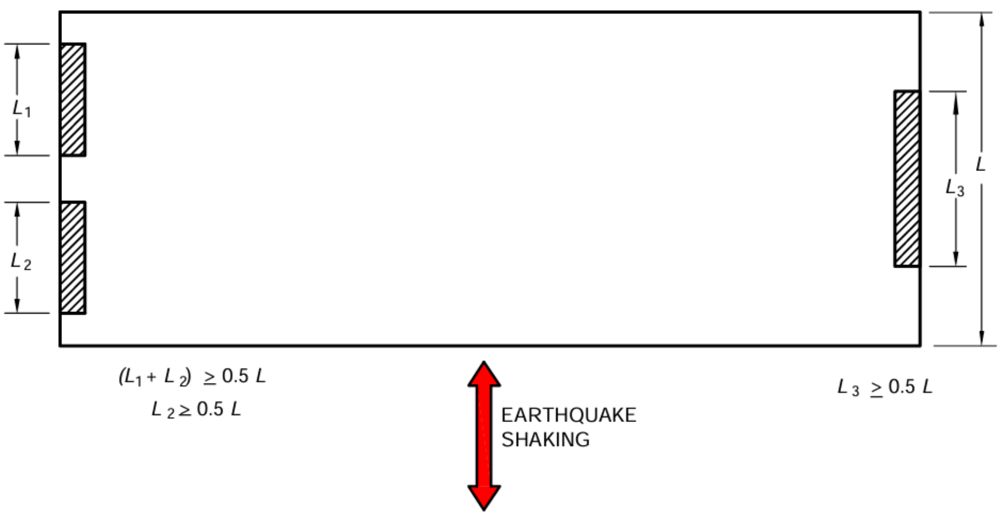


FIG. 49 AT LEAST TWO PARALLEL LINES OF WALLS ARE REQUIRED IN EACH PLAN DIRECTION

9.1.4 *Minimum Design Dimensions and Placements of Confining Elements*

Requirements regarding spacing of tie-columns and tie-beams are presented in Fig. 50. Minimum dimension of tie-column and tie-beam 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. 51).

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. A continuous horizontal band is not essential for $h < 2.5$ m, but a lintel beam shall be provided above the openings.

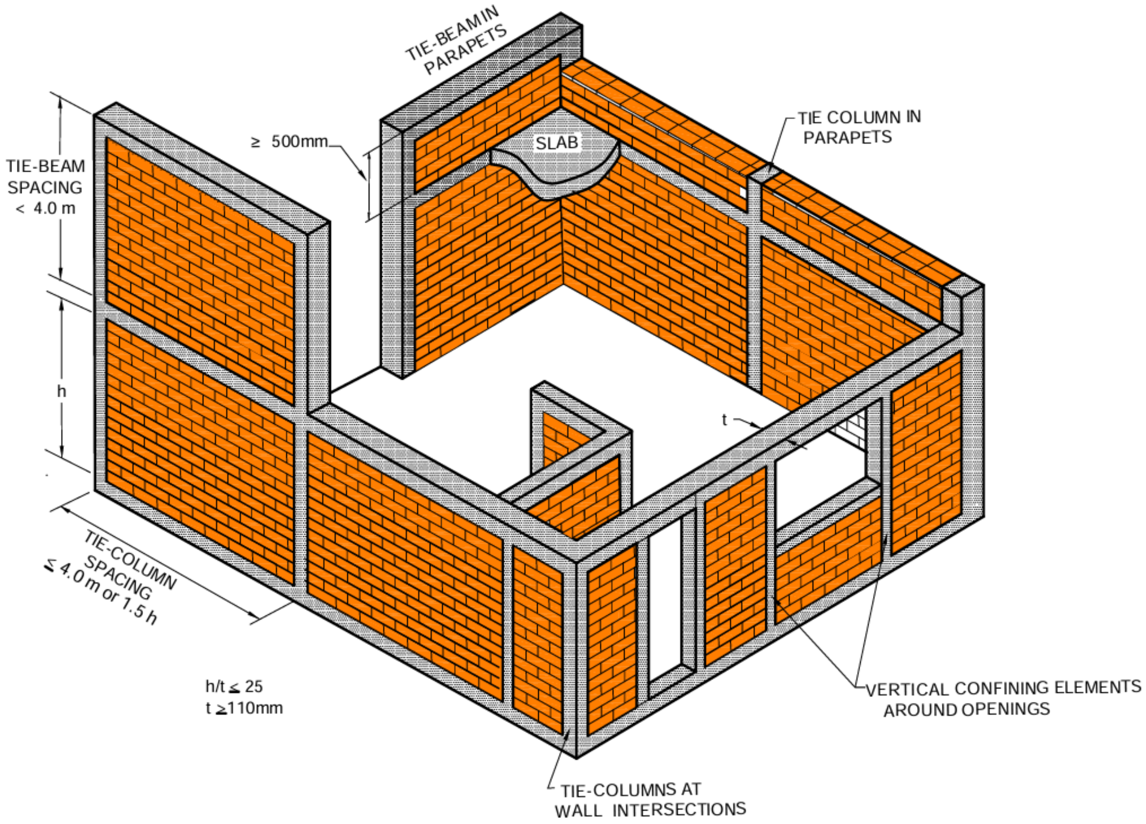


FIG. 50 REQUIREMENTS REGARDING CONFINING ELEMENTS

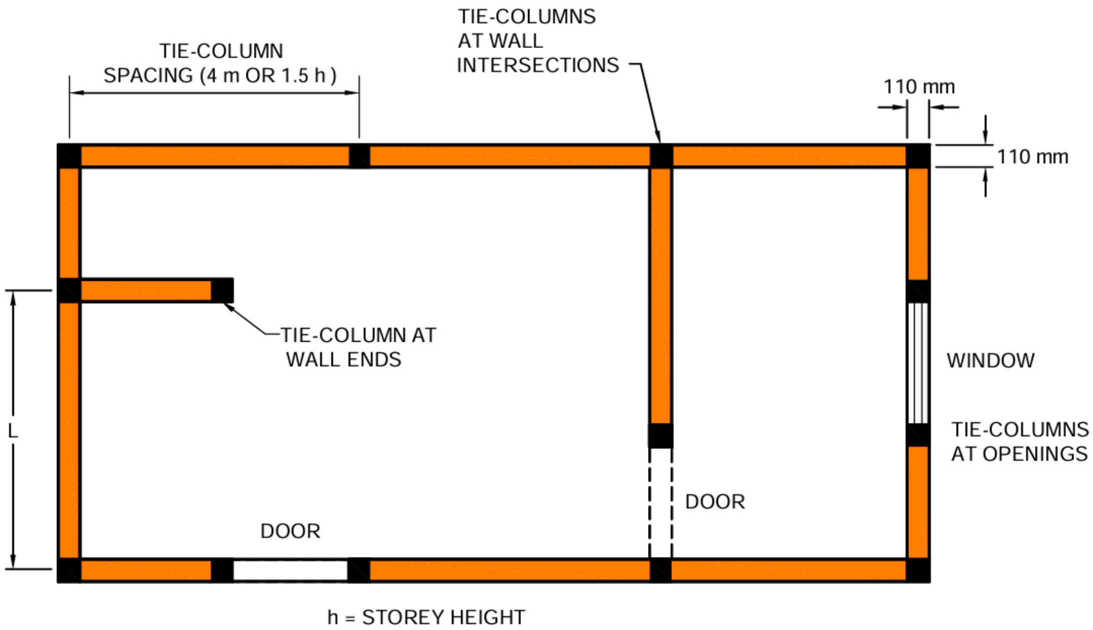


FIG. 51 TYPICAL PLAN ILLUSTRATING THE PLACEMENT OF RC TIE COLUMNS

9.1.5 *Minimum Dimensions of Masonry Walls*

The following shall be applicable:

a) Wall thickness (*t*) should not be less than 110 mm; and

b) The maximum wall height-to-thickness (*h*/*t*) ratio shall not exceed 25, where *h* is the height between floors.

9.1.5.1 *Aspect Ratio of Confined Masonry Walls*

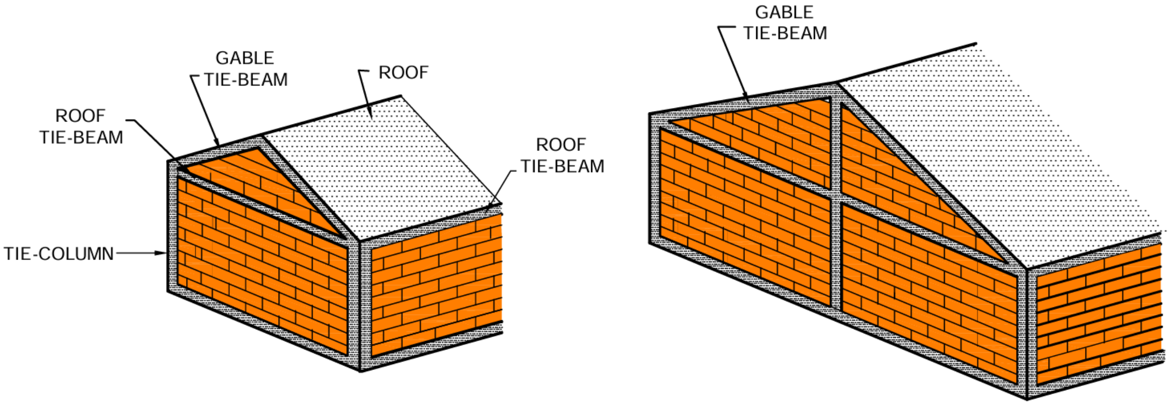
The height-to-width ratio of a wall which is a part of the lateral load resisting system shall be less than or equal to 2.

9.1.5.2 *Parapets*

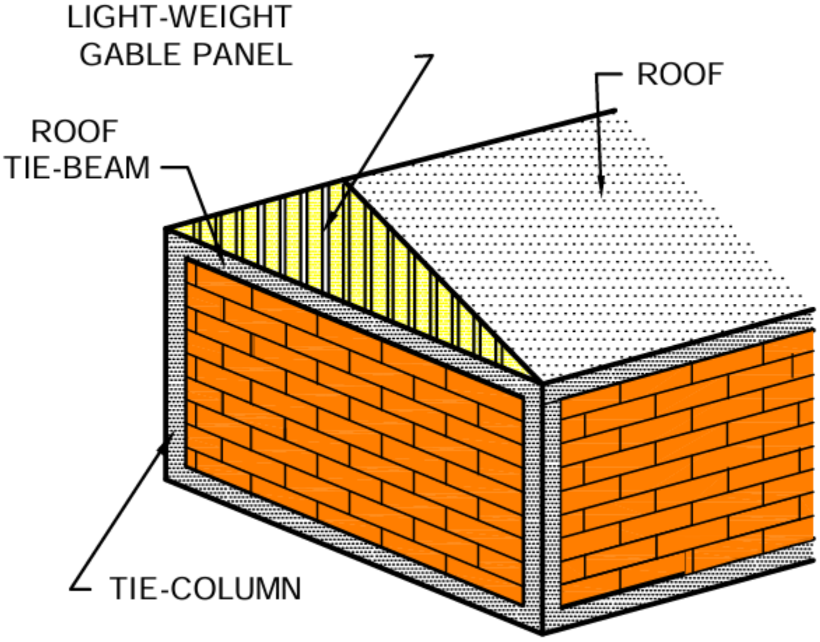
RC tie-columns and tie-beams should extend to the top of the parapet (Fig. 50). When a parapet is not confined by tie-beams, its height should not be more than 3 times its thickness. For parapet walls with top tie-beams, the height shall not be more than 1.2 m.

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. 52(a). Alternatively, a gable portion of the wall can be made of timber or other light-weight material that is continuous Fig. 52(b).



52A CONFINING ELEMENTS



52B LIGHT-WEIGHT GABLE PANELS

FIG. 52 TYPES OF 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 earthquake 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.

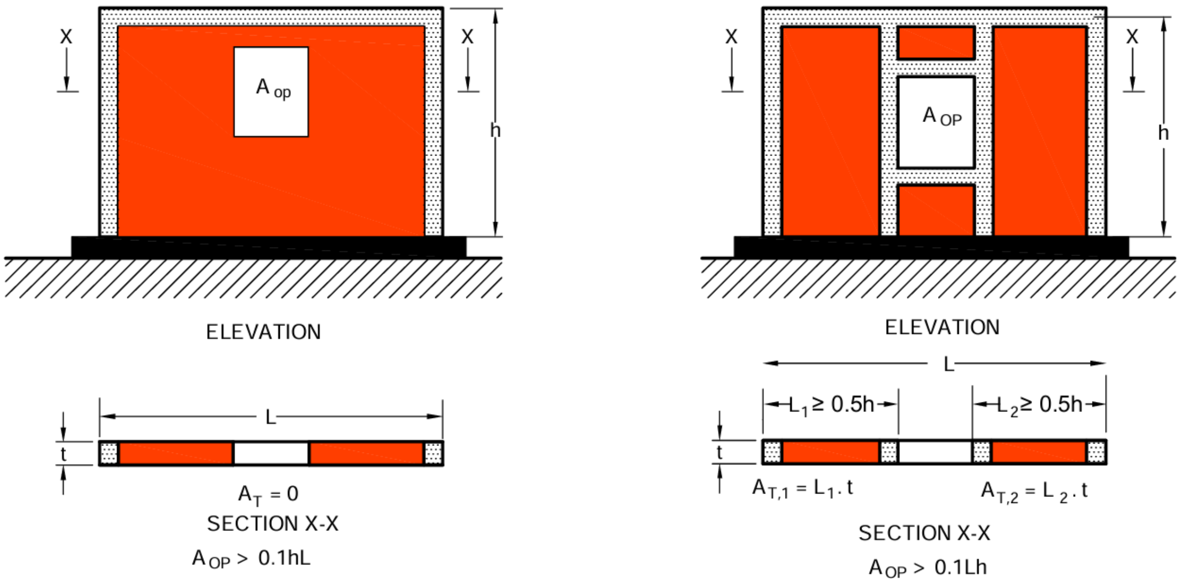
9.1.6.2 *Walls with large openings*

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 earthquake resistance of the building should be disregarded but should be strengthened [*see* Fig. 53(a)].

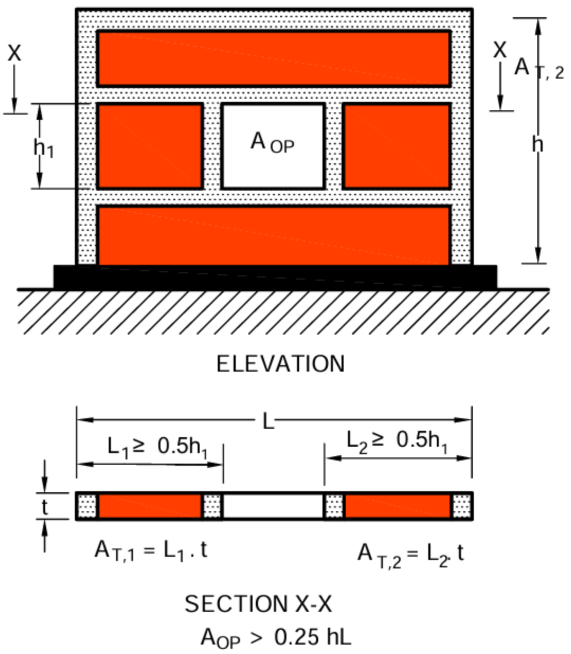
b) When reinforced concrete tie-columns are provided at the opening, and the confined masonry panels are considered to contribute to earthquake resistance of the building, the aspect ratio *H*/*L* of these panels shall be less than 2.0 Fig. 53(b). Better performance can be achieved by providing both 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. 53(c)].



53A UNCONFINED PANEL

53B PANEL WITH TIE COLUMNS



53C PANEL WITH HORIZONTAL BANDS

FIG. 53 MASONRY WALLS WITH LARGE OPENING

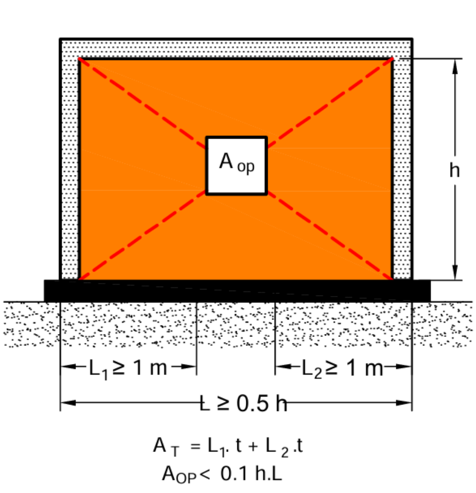
9.1.6.3 *Walls with Small and Single Openings*

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

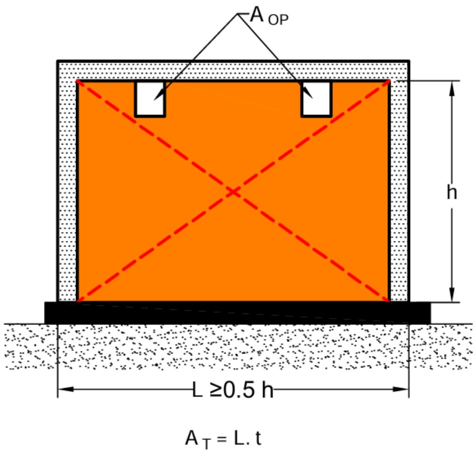
a) When the opening is located outside the diagonals Fig. 54(a), it can be ignored, and the entire wall cross-sectional area considered for earthquake resistance.

b) When an opening is located at the intersection of the panel diagonals [Fig. 54(b)], the panel cross-sectional area (A_T) considered for earthquake resistance should exclude the opening length.

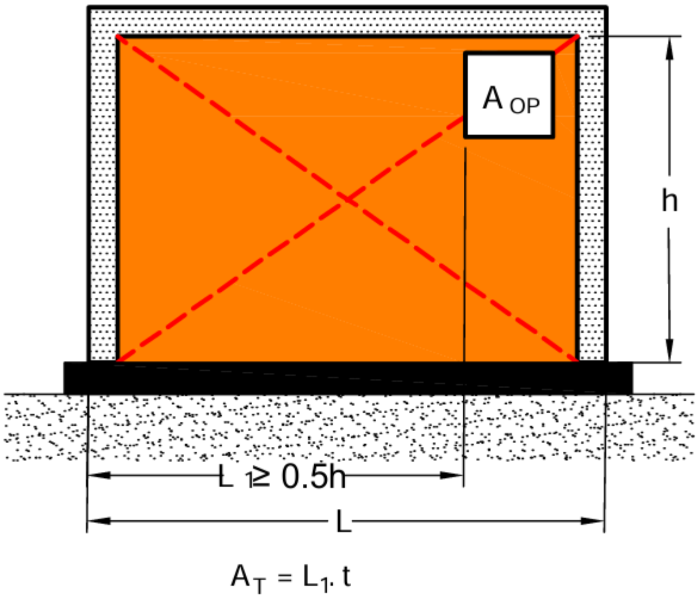
c) When an opening is located close to one end of the panel, the panel cross-sectional area (A_T) considered in earthquake resistance should use the larger pier length [Fig. 54(c)].



54A OPENING ALONG DIAGONAL



54B OPENING OFF THE DIAGONAL



54C OPENING ALONG THE DIAGONAL

FIG. 54 CONFINED MASONRY WALL PANEL WITH 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 for load cases (b), (c), and (d) may be increased by one-third.

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 be taken as per IS 456 and modulus of elasticity of masonry as per **8.3.2**. Transformed section properties of a confined masonry panel used in design are illustrated in Fig. 55.

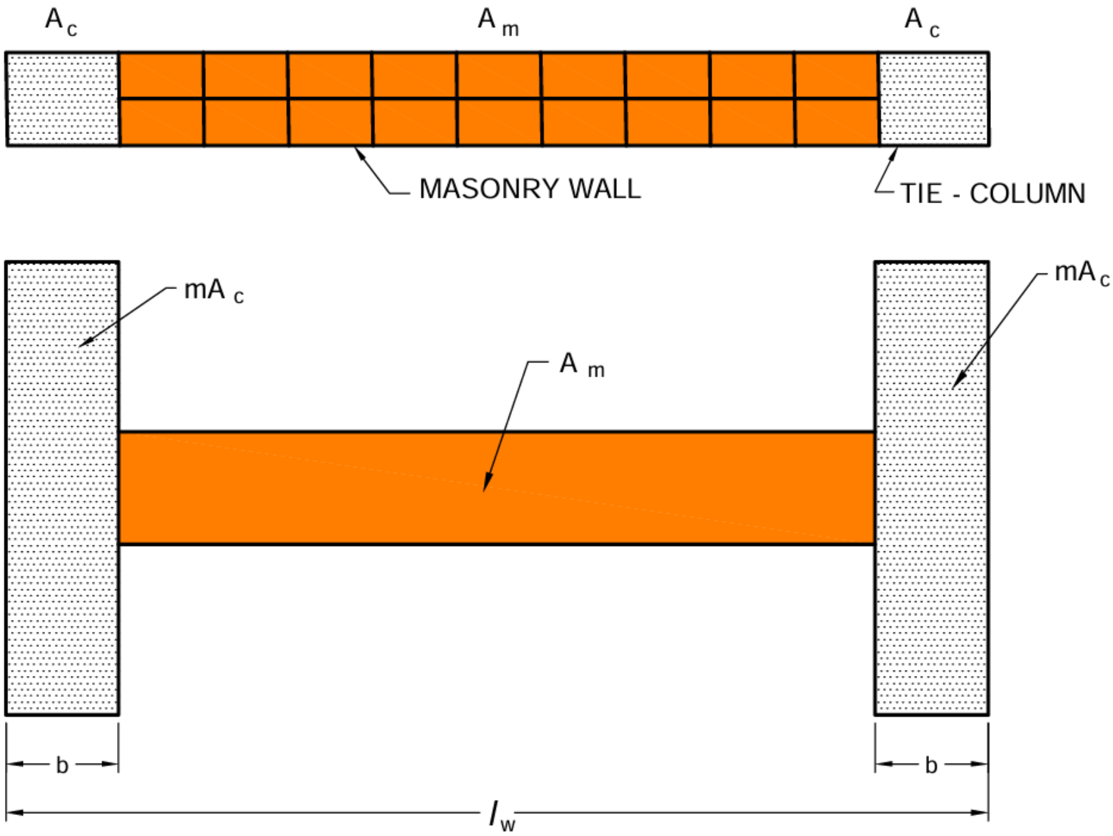


FIG. 55 TRANSFORMED SECTION PROPERTIES OF A CONFINED MASONRY PANEL

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_m = Compressive strength of masonry (**8.1.3**),
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 per Table 10.

Table 26 Shear Stress τ_{cm} for Confined Masonry
(*Clause* 9.2.2.6)

M/V$_d$	**f$_t$** (MPa)	**Maximum Allowable Shear Strength** (MPa)
(1)	(2)	(3)
< 1	$\frac{1}{36}\left(4-\frac{M}{Vl_w}\right)$	$\left(0.4-0.2\frac{M}{Vl_w}\right)$
> 1	$0.083\sqrt{f_m}$	0.4

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** shall apply.

9.2.2.6 *Permissible Shear Stress*

The allowable shear stress for confined masonry walls shall be as per Table 26. If there is tension in any part of a section of masonry, the area under tension shall be ignored while working out the shear stress on the section.

When designing buildings as per the Working Stress Method, the load combinations given in Table 27 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 27 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

9.2.2.7 *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) 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) Bond is perfect between steel reinforcement and adjacent concrete.

e) Maximum compressive strains in masonry (ε_m) and concrete (ε_c) shall be taken equal to 0.004 0 and 0.003 0, respectively.

f) Linear stress-strain relation of masonry shall be considered in design unless a more accurate relation is determined by experimental study of masonry prisms.

9.2.2.8 *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 0.003 0 in place of 0.003 5. 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. 56. For design purposes the partial safety factor γ_m of 2.0 shall be applied.

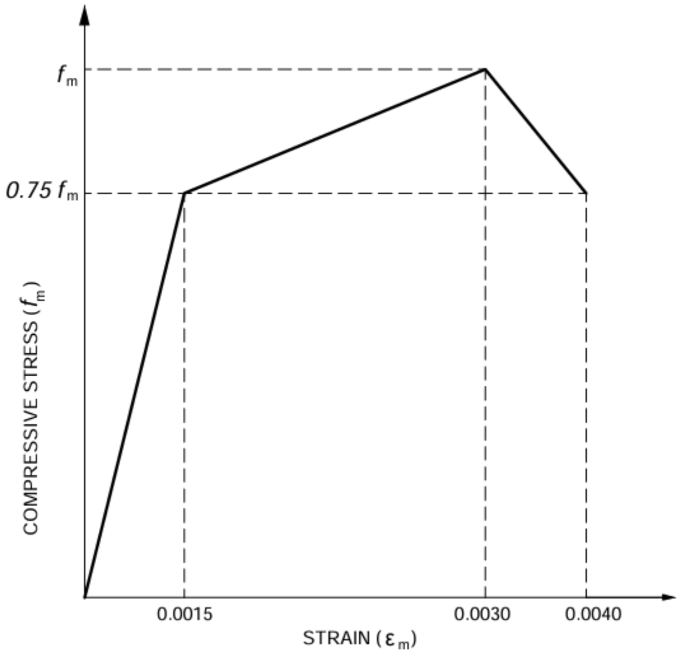


FIG. 56 ACCEPTABLE STRESS-STRAIN CURVE OF MASONRY

9.2.2.9 *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 10.

9.2.2.10 *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 against earthquake force acting in out-of-plane direction if slenderness limits in **9.4.1** 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. 57.

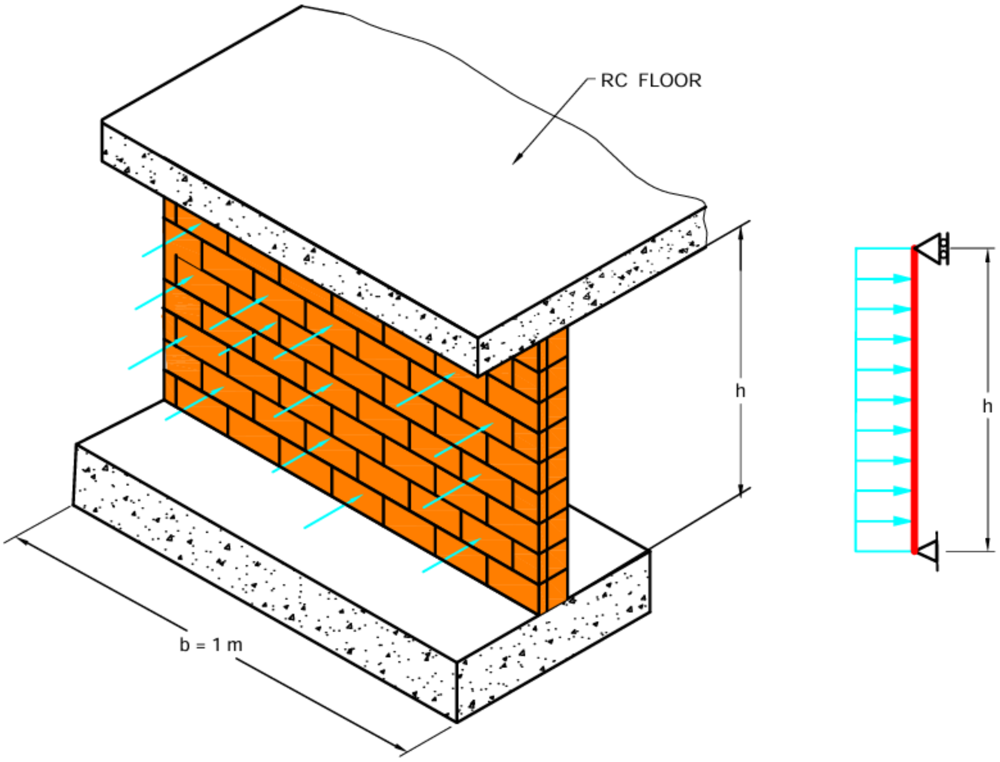


FIG. 57 UNIT LENGTH B OF MASONRY WALL CONSIDERED FOR OUT-OF-PLANE BENDING IN VERTICAL DIRECTION

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

$$\frac{P}{P_u} + \frac{M}{M_u} \leq 1.0$$

Design bending moment demand *M* due to design 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 per **9.2.2.9** 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.

9.2.2.9 *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 **9.2.2.7**.

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. 58):

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

$$M_u = \left[0.30Pd + M_{uf}\right]$$

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

ii) When $P = \left(P_u/3\right)$

Point B: $M_u = \left[0.10Pd + M_{uf}\right]$, and Point C:

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

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

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

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.

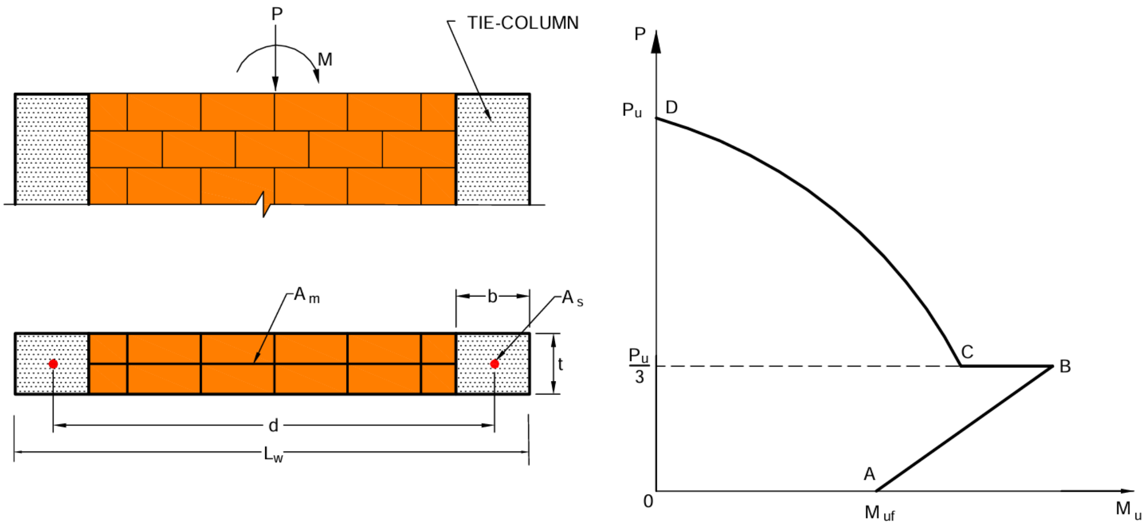


FIG. 58 SIMPLIFIED INTERACTION DIAGRAM FOR DESIGN OF CONFINED MASONRY WALLS

9.2.2.12 *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:

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

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. 59) 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. 59) 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.

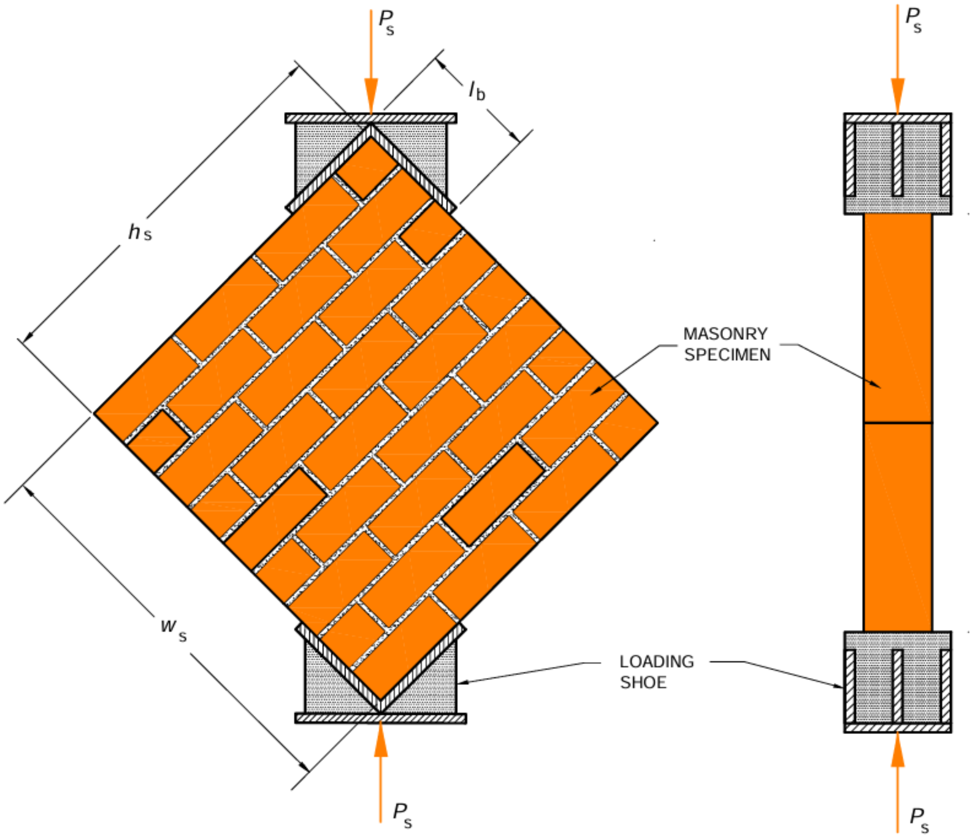


FIG. 59 WALL SPECIMEN FOR DIAGONAL COMPRESSION TEST

(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. 60; 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:

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

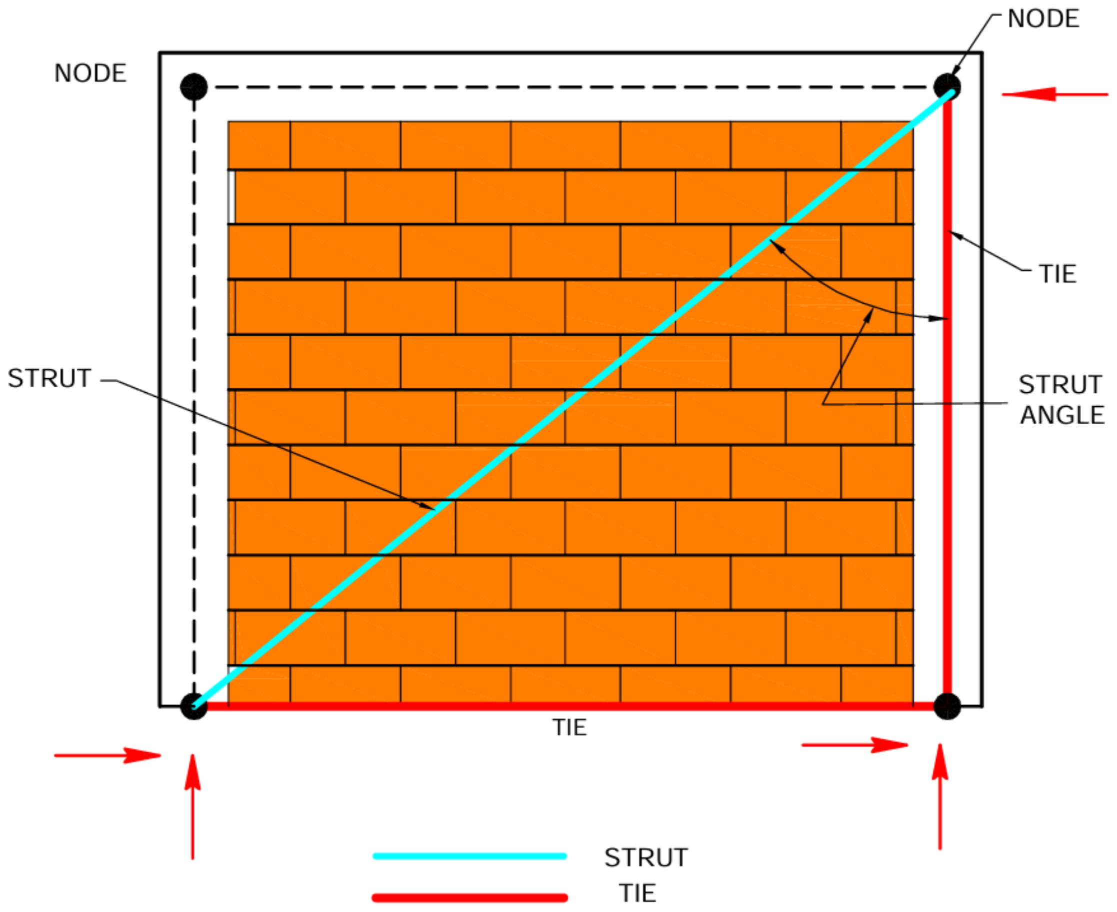


FIG. 60 STRUT-AND-TIE MODEL OF CONFINED MASONRY WALLS

9.2.2.13 *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*.

9.2.3 *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 earthquake forces determined as per IS 1893 (Part 1). Also, the load combinations shall be as per **7.5** of IS 1893 (Part 1).

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 earthquake analysis, or advanced analysis methods (static and/or dynamic analysis). The simplified method can be used for buildings up to three storeys high, 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 earthquake 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 IS 1893 (Part 1). 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.

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.

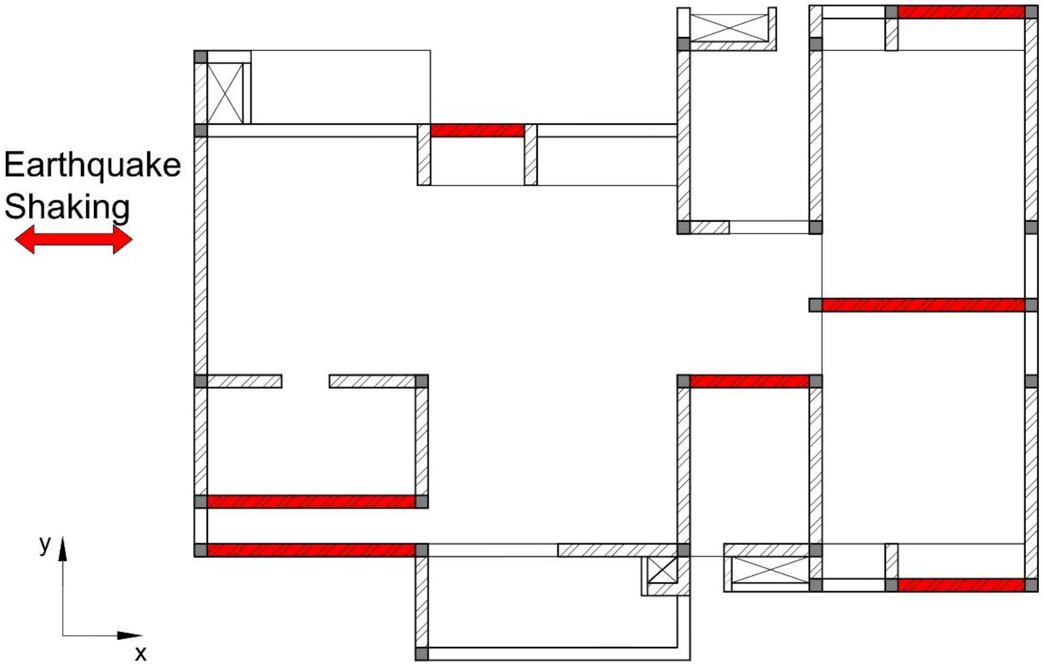
9.3.2.3 Wall Index (*WI*), also known as wall density, shall be used as an indicator of earthquake load-resisting capacity of earthquake resistant design of low-rise buildings with regular plan shape and elevation. The design according to the simplified method shall be deemed satisfactory provided that 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 shall be taken as ratio of the sum of cross-sectional areas of all confined masonry walls in the considered direction of earthquake shaking, given by:

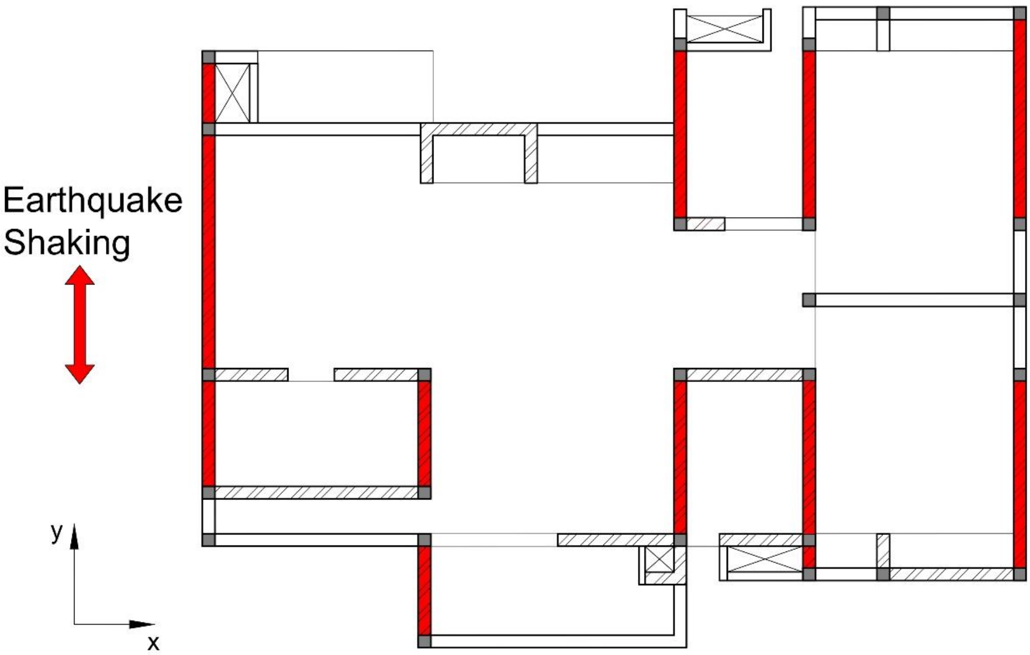
$$WI_{Floor} = \frac{A_W}{A_p}$$

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. 61. The wall index shall be estimated along each principal plan direction (X and Y) of the building. Only confined masonry walls aligned in the direction under consideration (shown shaded in Fig. 61) should be considered in the A_W calculations for the specific direction.



61A X-DIRECTION



61B Y-DIRECTION

FIG. 61 WALLS TO BE CONSIDERED IN ESTIMATION OF WALL INDEX ALONG PRINCIPAL PLAN DIRECTIONS OF A BUILDING: (A) X-DIRECTION, AND (B) Y-DIRECTION

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 3 storey 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 IS 1893 (Part 1);
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 28; these were obtained based on the Limit State Method of Design, and assuming masonry unit compressive strength of 5.0 MPa, mortar type M1, and average floor weight *w of* 12 kN/m^2.

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

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

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 IS 1893 (Part 1), γ_m the partial safety factor for masonry as given in **9.2.2.8**.

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 28 WI_{floor} for different Earthquake Zones
(Clause 9.3.2.3.1)

Earthquake Zone	**II**	**III**	**IV**	**V**	**VI**
WI_{floor} (percent)	0.7	1.1	1.6	2.4	3.2

9.3.2.3 *Advanced analysis methods*

Earthquake 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 IS 1893 (Part 1). 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 Earthquake 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*

In earthquake zones IV, V and VI, reduced tie spacing (s/2) is required at the ends of tie-columns, as shown in fig. 62. 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.

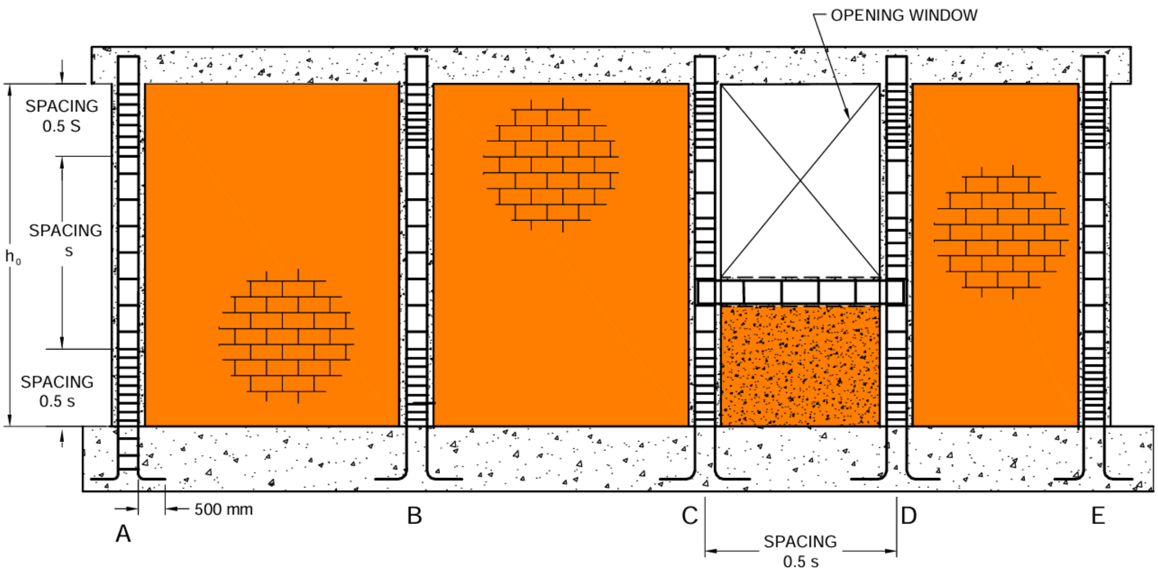


FIG. 62 TIE-COLUMN TRANSVERSE REINFORCEMENT REQUIREMENTS IN EARTHQUAKE ZONES IV, V AND VI

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.

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. 63). Ties shall have 135° hooked ends. Hooks shall be staggered as shown in Fig. 63.

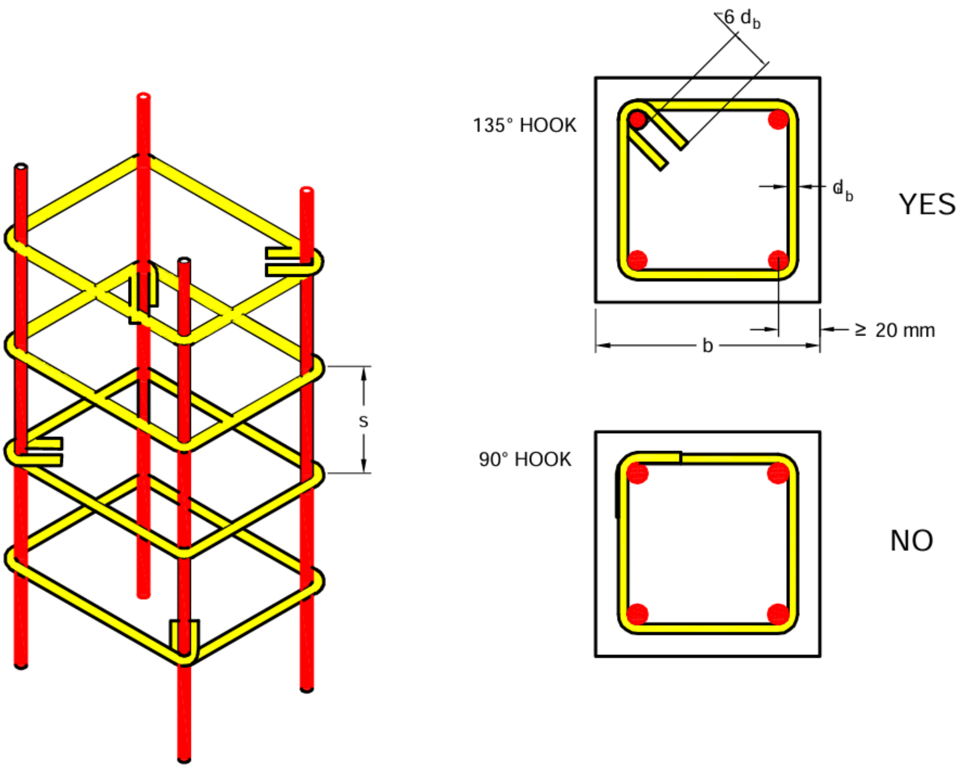
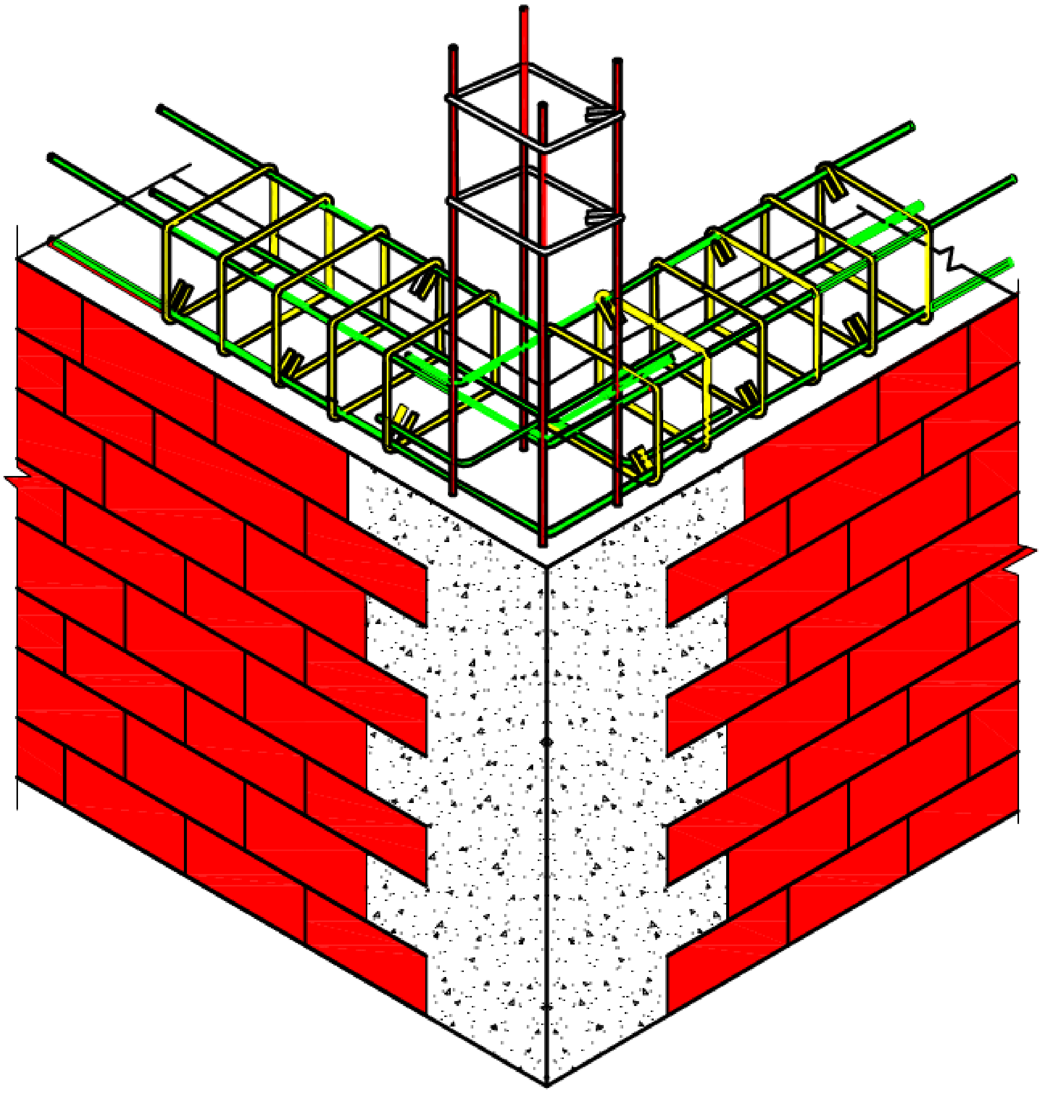


FIG. 63 LAYOUT AND DETAILING OF TIE-COLUMNS AND TIE-BEAMS

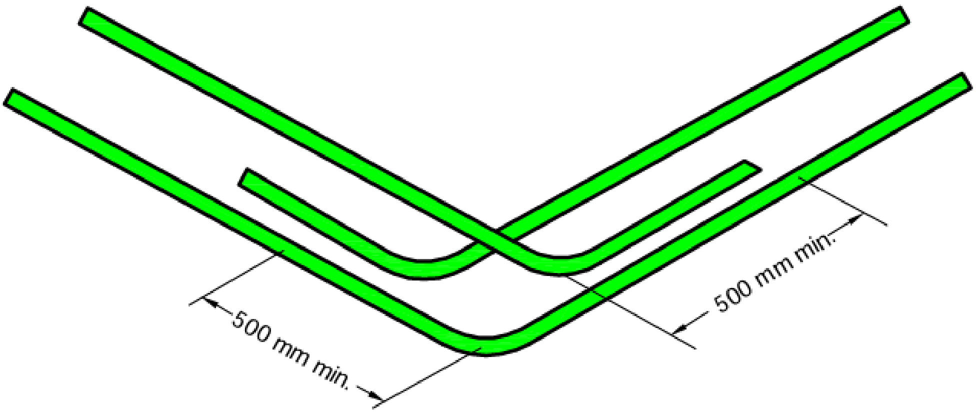
9.4.1.4 *Anchorage of longitudinal bars*

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. 64).

b) Tie-column longitudinal bars at the roof level shall be bent by 90 degrees and lapped with the tie-beam longitudinal reinforcement (Fig. 65).



64A WALL INTERSECTION



64B HOOKED ANCHORAGE OF LONGITUDINAL REINFORCEMENT AT WALL INTERSECTIONS

FIG. 64 TIE-BEAM CONSTRUCTION: (A) WALL INTERSECTION, AND (B) HOOKED ANCHORAGE OF LONGITUDINAL REINFORCEMENT AT WALL INTERSECTIONS

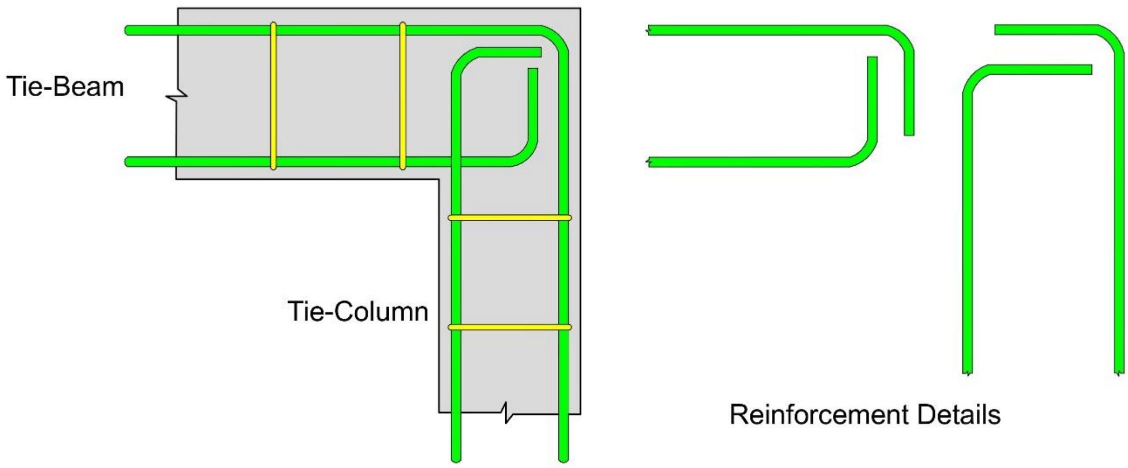


FIG. 65 ANCHORAGE OF LONGITUDINAL REINFORCEMENT OF TIE-COLUMN AND TIE-BEAM

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.

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. 66. 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. 67).

Reinforced concrete tie-columns shall be provided at the openings as shown in Fig. 54B and Fig. 54C. A nominal reinforcement around openings as detailed in Fig. 53 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.

9.4.2 *Construction*

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

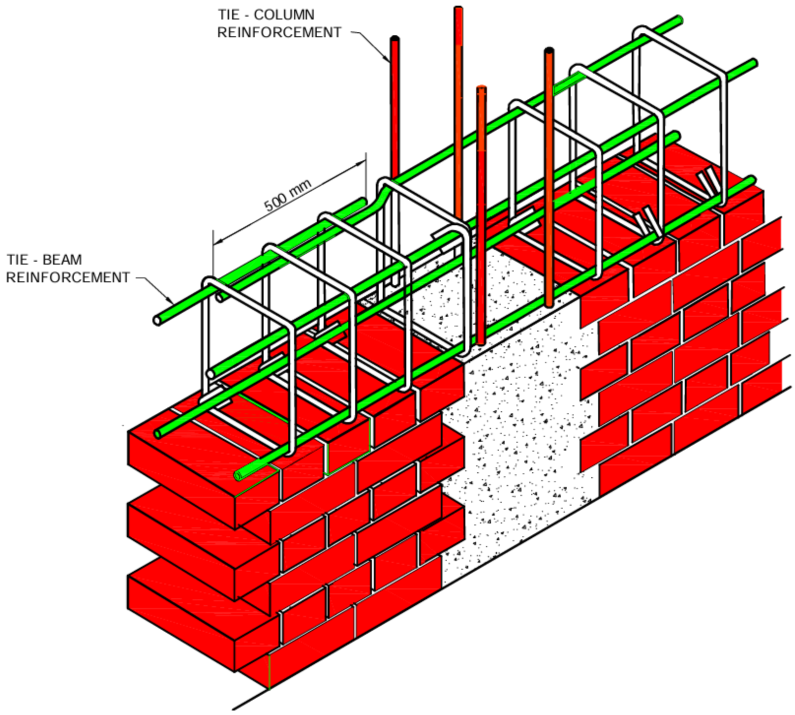


FIG. 66 CONTINUITY OF TIE-BEAM REINFORCEMENT THROUGH JOINT

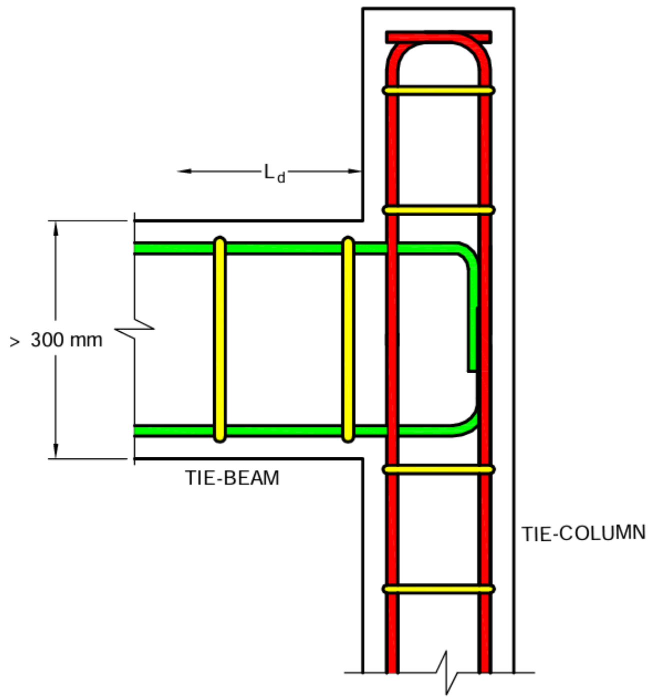


FIG. 67 ADDITIONAL CONFINEMENT OF VERTICAL REINFORCEMENT IN TIE-BEAMS AND TIE-COLUMNS IN EXTERIOR JOINT

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:

1) All intersecting walls shall be connected, unless measures to assure stability and good performance are taken.
2) Surfaces of construction joints shall be clean and rough. The joint shall be moistened before construction when clay units are used.
3) To fulfill the requirements of excellent masonry work, Flemish bond or English bond in brickwork shall be adopted.
4) 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.

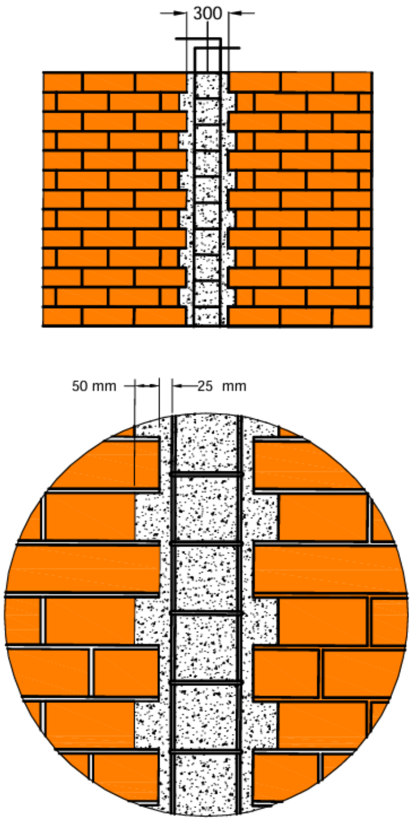
c) *Mortar joint thickness*

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.

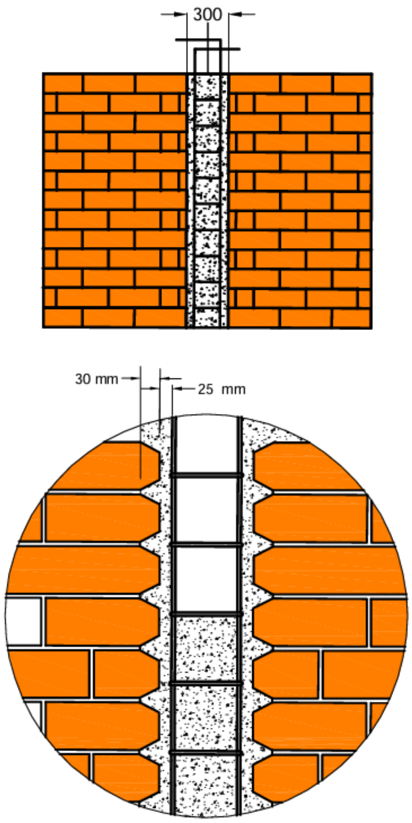
d) *Toothing*

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. 68.

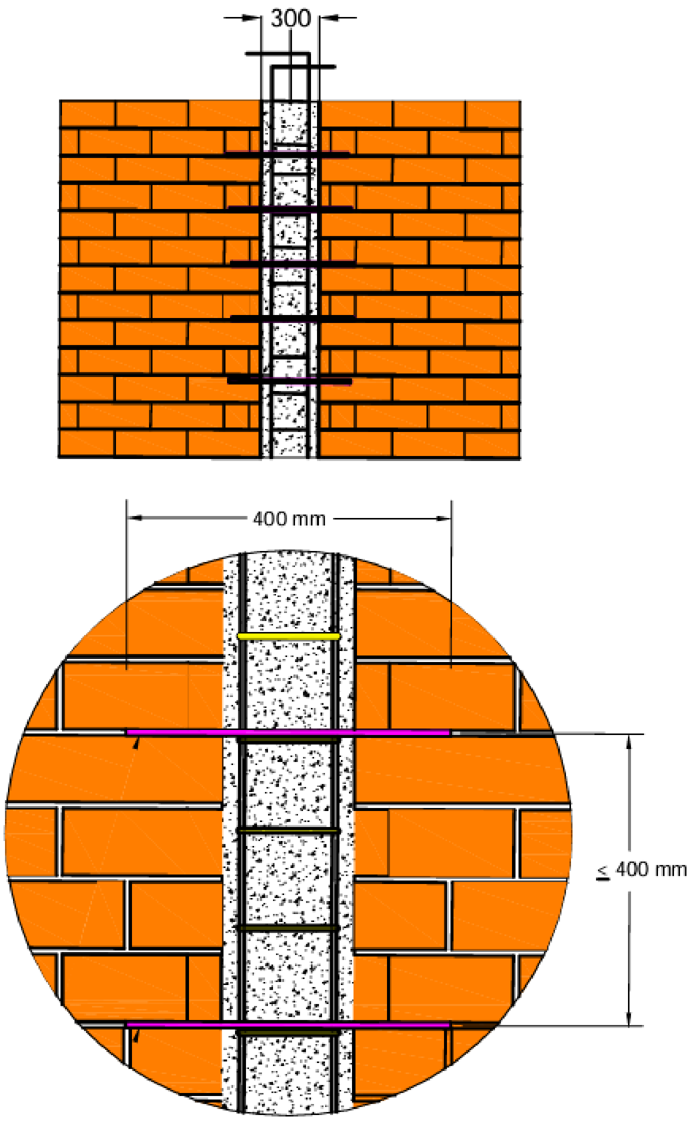
Toothed edges shall be left on each side of the wall at the interface with the tie-columns. Toothing length shall be equal to one-quarter of the masonry unit length, but not less than 50 mm, as shown in Fig. 68A. When hand-made bricks are used, it is desirable to cut the brick edges, as shown in Fig. 68B. 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 no toothing, as shown in Fig. 68C. The dowels are not necessary when toothed edges are used. 6 mm diameter bars shall be preferably used as dowels.



68A FULL TOOTHING



68B PARTIAL TOOTHING



68C NO TOOTHING

FIG. 68 REINFORCEMENT DETAILS OF TOOTHING IN CONFINED MASONRY WALLS

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.

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 WALLS (RMW)

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 **5** 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 Effective span of simply supported or 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.

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

a) Center of its support; and

b) Face of support plus half its effective depth.

10.2.4 *Slenderness Ratio*

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 spandrel masonry beams that are 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 29.

Table 29 Maximum Ratio of Effective Span to Effective Depth for walls subjected to out-of-plane bending
(*Clause* 10.2.4.1)

Sl No.	**Boundary Conditions**	**Maximum Permissible ratios**	
		Effective Span to Effective Thickness (l_{ef} /t_{ef}) of wall subjected to out-of-plane bending	**Effective Span to Effective Depth (l_{ef} /d) of Beam part of wall subjected to in-plane bending**
(1)	(2)	(3)	(4)
1	Simply supported	35	20
2	Continuous	45	26
3	Spanning in 2 directions	45	-
4	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. The tie-columns shall be extended into the parapet wall and terminated at the top of the parapet in a tie-beam.

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) In beams, 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 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, 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 (Fig. 69):

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, 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 reinforcement*

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 the requirements of IS 456, and the following anchorage values shall be used:

1) *Bend* — 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) *Hook* — The anchorage value of a standard U-type hook shall be equal to 16 times the diameter of the bar, and of the 135° hook end equal to 12 times diameter of the bar.

c) For transverse reinforcement 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.**

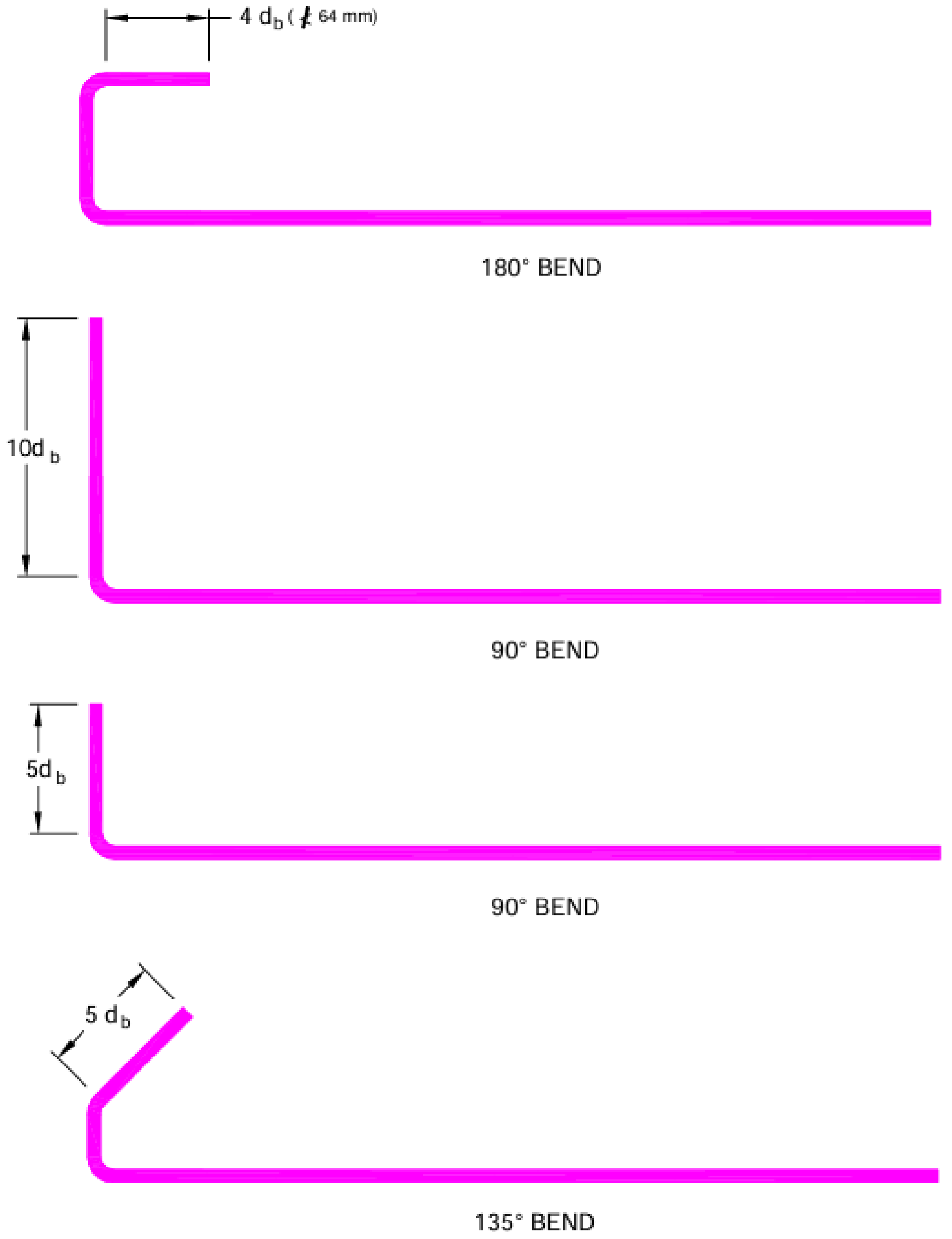


FIG. 69 STANDARD HOOK OF A REINFORCING BAR

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 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 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; and

c) Splices in tension members (which are not part of the lateral force resisting system) shall be enclosed in closed loop hoops made of bars not less than 8 mm diameter with spacing not more than 100 mm.

10.3.7 *Curtailment of Tension Reinforcement*

10.3.7.1 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. But, 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.

10.3.7.2 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:

a) 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
b) 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 8 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 135º.

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,

d = Distance from extreme compression fiber to centroid of tension reinforcement, and

F_s = Permissible stress in steel reinforcement as defined in **10.4.1.4**.

10.3.9.3 The maximum spacing of shear reinforcement shall not be greater than 0.5*d* or 120 mm, whichever is smaller.

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*

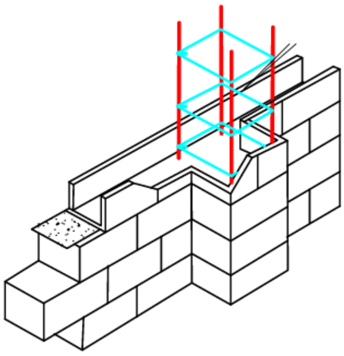
Reinforcement shall be located such that it acts compositely with the masonry and various ways in which it can be used in reinforced masonry are shown in Fig. 70.

10.3.10.2 *Protection of reinforcement*

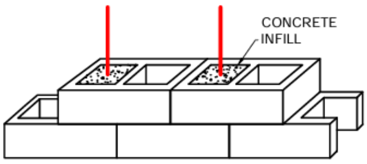
Where steel reinforcing bars are embedded in filled cavity (or pockets) or special bond construction, the bars shall have the minimum clear cover of 10 mm in mortar or a minimum clear cover of 15 mm or bar diameter, whichever is more in cement concrete (grout) so as to achieve good bond and corrosion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

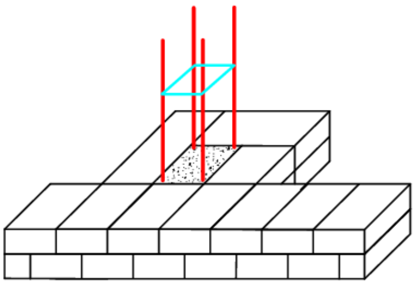
Reinforcing steel shall be corrosion resistant or protected adequately against corrosion. Reinforcement shall be galvanized reinforcing steel for protection against corrosion.



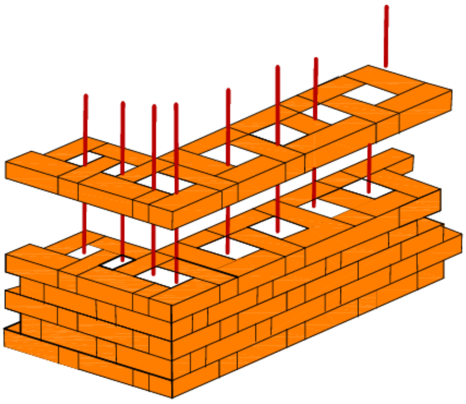
REINFORCED HOLLOW
BLOCKWORK WALL



WALL WITH VERTICAL AND BED



POCKET TYPE WALL



REINFORCING POCKETS FORMED BY BONDING
ARRANGEMENTS-QUETTA BOND

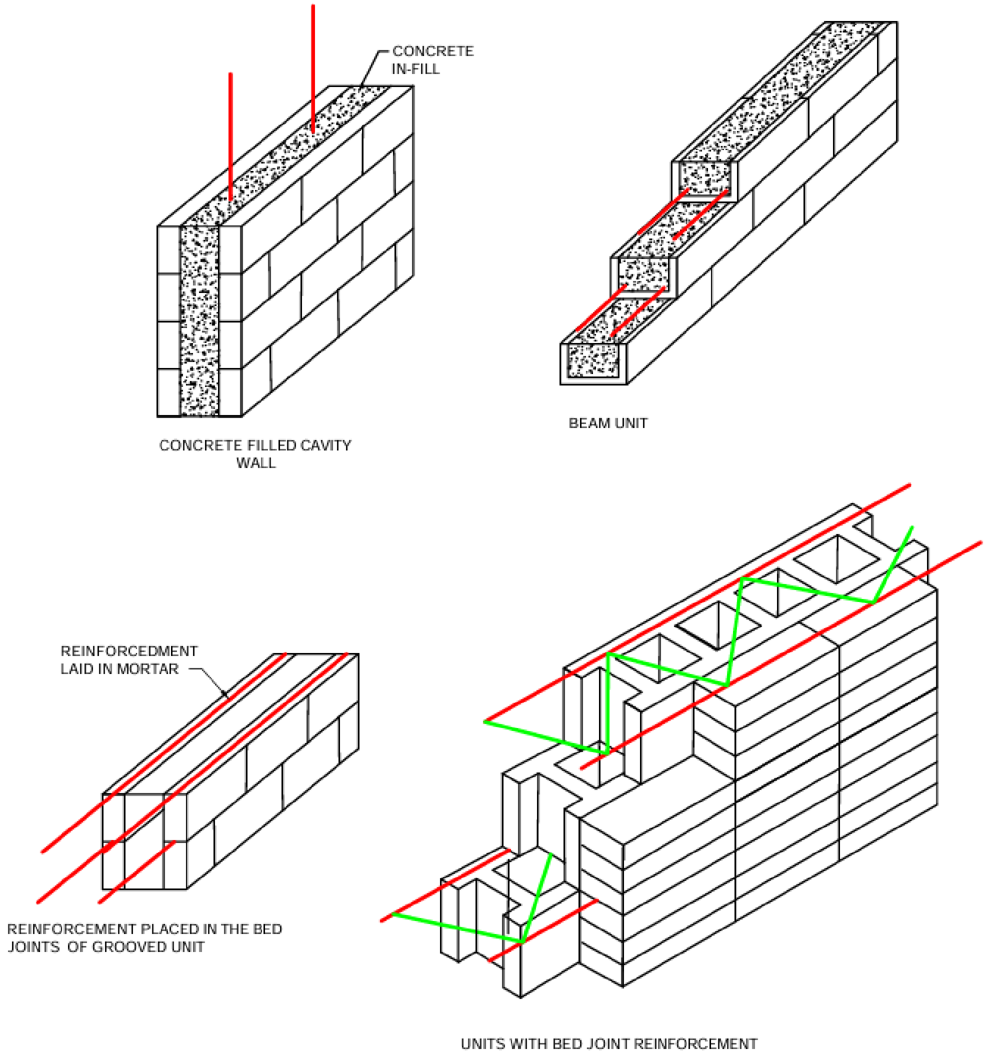


FIG. 70 REINFORCEMENT DETAILS

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_m = Permissible compressive stress in masonry,
F_s = Permissible steel tensile stress, and
k_s = Stress reduction factor as in Table 10.

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. 71), the reinforced section shall be considered as having a width of not more than:

a) centre-to-centre bar spacing, and

b) 6 times the wall thickness.

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** shall apply.

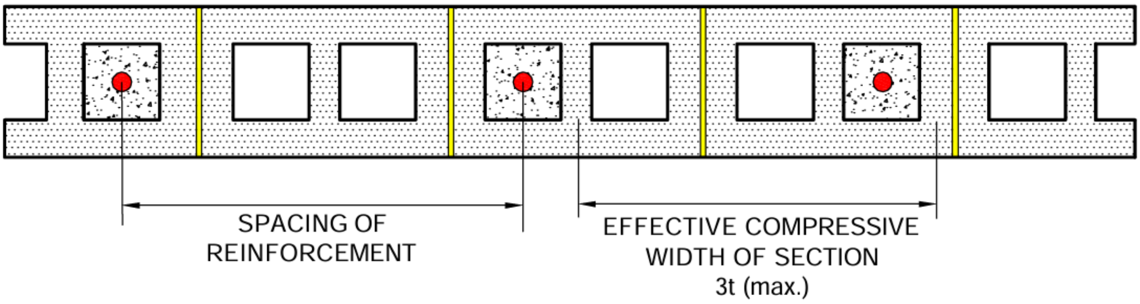


FIG. 71 EFFECTIVE COMPRESSIVE WIDTH

10.4.1.4 *Permissible shear stress*

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

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

For walls in compression and bending, the Permissible Shear Stress for reinforced masonry walls shall be according to Table 30.

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 30 Permissible Shear Stress in Reinforced Masonry Walls under Compression and Bending
(*Clause* 10.4.1.3)

Type of Wall	***M/V*d**	**Permissible Shear Stress, *F*v (MPa)**	**Maximum Allowable shear stress (MPa)**
(1)	(2)	(3)	(4)
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
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 Earthquake 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 IS 1893 (Part 1).

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 earthquake performance, which can be appropriately chosen for a building considering its importance, location, and acceptable degree of damage. Table 31 summarizes the requirement for these structural walls and *R* values recommended for use in IS 1893 (Part 1).

Table 31 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)
1	RMB 1	Reinforced Masonry Walls with minimum reinforcement	*As per* **10.5.2.1**	II and III	2.5
2	RMB 2	Reinforced Masonry Walls with design reinforcement	*As per* **10.5.2.2**	II, III, IV and V	3.0

3	RMB 3	Reinforced Masonry Walls with special design reinforcement	*As per* **10.5.2.2** and **10.5.2.3**	All	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 as given in **5** 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 Walls*.

b) *Minimum Reinforcement Requirements* (*Figure* 72)

   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 8 mm spaced not more than 400 mm; or bond beam reinforcement shall be provided of at least 100 mm^2 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 section and shall comply with the requirements of **10.5.2.1** (a) to (c).

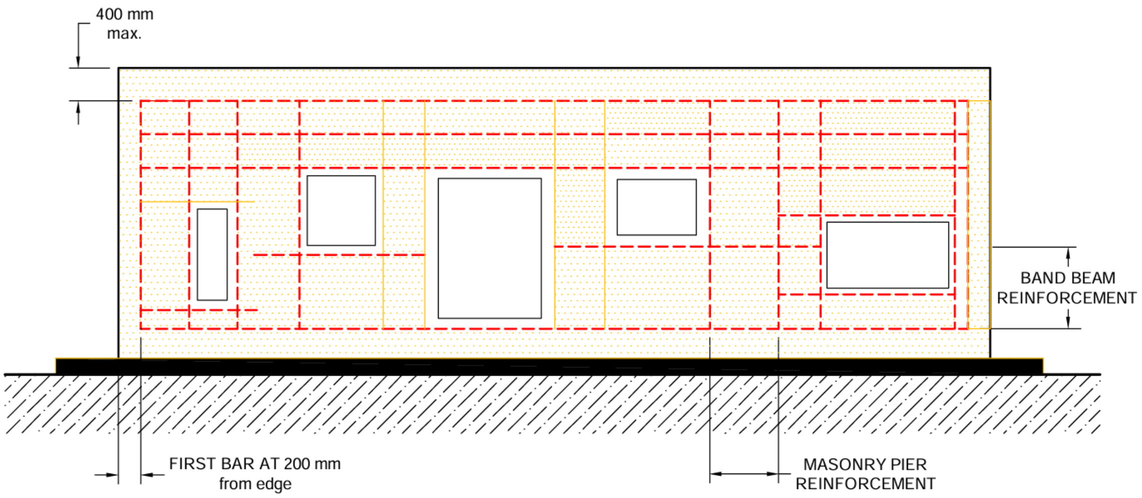


FIG. 72 REQUIREMENTS OF MINIMUM REINFORCEMENT IN MASONRY WALLS (RMB 1)

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 135º or 180º standard hook.

11 GUIDELINES FOR IMPROVING EARTHQUAKE RESISTANCE OF LOW STRENGTH MASONRY BUILDINGS

The term 'low strength masonry' includes fired brickwork laid in clay mud mortar and random rubble; uncoursed, undressed or semi-dressed stone masonry in weak mortars; such as cement sand, lime sand and clay mud. Special features of design and construction for improving earthquake resistance of buildings of low strength masonry are given in **11.1** to **11.6.5.7.**

11.1 For the purpose of this clause, the terminology given in **8.1** shall apply.

11.2 General Principles

The general principles given in **9.2.1** to **9.2.5** shall be observed in construction of earthquake resistant buildings.

11.2.1 *Lightness*

Since the earthquake force is a function of mass, the weight of the building shall be as minimum as possible, consistent with structural safety and functional requirements. Roofs and upper storeys of buildings, in particular, should be designed as light as possible.

11.2.2 *Continuity of Construction*

11.2.2.1 As far as possible, the parts of the building should be tied together in such a manner that the building acts as one unit.

11.2.2.2 For integral action of building, roof and floor slabs should be continuous throughout as far as possible.

11.2.2.3 Additions and alterations to the structures should be accompanied by the provision of positive measures to establish continuity between the existing and the new construction.

11.2.3 *Projecting and Suspended Parts*

11.2.3.1 Projecting parts should be avoided as far as possible. If the projecting parts cannot be avoided, they should be properly reinforced and firmly tied to the main structure, and their design should be in accordance with good practice [6-4(7)].

NOTE – In cases where stability of projecting parts against overturning is achieved by counterweight in the form of wall, slab, etc, the overturning should be checked by increasing the weight of projecting part and decreasing the weight of stabilizing mass simultaneously in accordance with the vertical seismic coefficient specified in **4.4.2** of good practice [6-4(7)].

11.2.3.2 Ceiling plaster shall preferably be avoided. When it is unavoidable, the plaster shall be as thin as possible.

11.2.3.3 Suspended ceiling shall be avoided as far as possible. Where provided they shall be light, adequately framed and secured.

11.2.4 *Building Configuration*

11.2.4.1 In order to minimise torsion, the building should have a simple rectangular plan and be symmetrical both with respect to mass and rigidity so that the centre of mass and rigidity of the building coincide with each other. It will be desirable to use separate blocks of rectangular shape particularly in seismic zone V and IV.

NOTE – For small buildings, minor asymmetry in plan and elevation may be ignored. Designing such buildings against torsion may be difficult and uncertain.

11.2.5 *Fire Safety*

Fire frequently follows an earthquake and therefore, buildings shall be constructed to make them fire resistant in accordance with the provisions of Part 4 'Fire and Life Safety' of the Code for fire safety, as relevant.

11.3 Special Construction Features

11.3.1 *Foundations*

11.3.1.1 For the design of foundations, the provisions of Part 6 ‘Structural Design: Section 2 Soils and Foundation’ in conjunction with **5** of Part 6 ‘Structural Design: Section 1 Loads, forces and Effects’ shall generally be followed.

11.3.1.2 The subgrade below the entire area of the building shall preferably be of the same type of the soil. Wherever this is not possible, the buildings should preferably be separated into units and then the units should be located separately.

11.3.1.3 Loose fine sand, soft silt and expansive clays should be avoided. If unavoidable, the following measures may be taken to improve the soil on which the foundation of the building may rest:

a) Sand piling/under-reamed piling/stone columns, etc; and
b) Soil stabilization.

11.3.2 *Roofs and Floors*

11.3.2.1 Flat roof or floor should not preferably be made of tiles or ordinary bricks supported on steel, timber or reinforced concrete joists, nor they shall be of a type which in the event of an earthquake is likely to be loosened and parts or all of which may fall. If this type of construction cannot be avoided, the joists should be blocked at ends and bridged at intervals such that their spacing is not altered during an earthquake.

11.3.2.1.1 For pitched roofs, corrugated iron or asbestos sheets should be used in preference to country, Allahabad or Mangalore tiles or other loose roofing units. All roofing materials shall be properly tied to the supporting members. Heavy roofing materials shall generally be avoided.

11.3.2.2 *Pent roofs*

11.3.2.2.1 All roof trusses should be supported on and fixed to timber band, reinforced concrete, or reinforced brick band. The holding down bolts should have adequate size and length as required for earthquake and wind forces.

Where a trussed roof adjoins a masonry gable, the ends of the purlins should be carried on and secured to a plate or bearer which shall be adequately bolted to reinforced concrete or reinforced brick band at the top of gable end masonry.

11.3.2.2.2 At tie level all the trusses and the gable end should be provided with diagonal braces in plan so as to transmit the lateral shear due to earthquake force to the gable walls acting as shear walls.

11.3.2.3 *Jack arches*

Jack arched roofs or floors, where used should be provided with mild steel ties in all spans along with diagonal braces in plan to ensure diaphragm actions.

11.3.3 *Staircases*

11.3.3.1 The inter-connection of the stairs with the adjacent floors should be appropriately treated by providing sliding joints at the stairs to eliminate their bracing effect on the floors. Ladders may be made fixed at one end and freely resting at the other.

11.3.3.2 *Built-in staircase*

When stairs are built monolithically with floors, they can be protected against damage by providing rigid walls at the stair opening. The walls enclosing the staircase, should extend through the entire height of the stairs and to the building foundations.

11.4 Box Type Construction

This type of construction consists of prefabricated or *in-situ* masonry walls along both the axes of the building. The walls support vertical loads and also act as shear walls for horizontal loads acting in any direction. All traditional masonry construction falls under this category. In prefabricated construction attention shall be paid to the connection between wall panels so that transfer of shear between them is ensured.

11.5 Categories of Buildings

For the purpose of specifying the earthquake resisting features in masonry and wooden buildings, the buildings have been categorized in four categories B to E based on the seismic zone and the importance of building I, where I – importance factor applicable to the building [*see* **6.4.2** and Table 6 of good practice [6-4(8)]., the classification as given in **8.5** shall be applicable. The building categories are given in Table 14.

11.6 Low Strength Masonry Construction

11.6.1 Two types of construction are included herein, namely:

a) Brick construction using weak mortar, and
b) Random rubble and half-dressed stone masonry construction using different mortars such as clay mud, lime-sand and cement sand.

11.6.1.1 These constructions should not be permitted for important buildings and should preferably be avoided for building category *D* and shall not be used for category *E* (*see* Table 14).

11.6.1.2 It will be useful to provide damp-proof course at plinth level to stop the rise of pore water into the superstructure.

11.6.1.3 Precautions should be taken to keep the rain water away from soaking into the wall so that the mortar is not softened due to wetness. An effective way is to take out roof projections beyond the walls by about 500 mm.

11.6.1.4 Use of a water-proof plaster on outside face of walls will enhance the life of the building and maintain its strength at the time of earthquake as well.

11.6.1.5 Ignoring tensile strength, free standing walls should be checked against overturning under the action of design seismic coefficient, α_h, allowing for a factor of safety of 1.5.

11.6.2 *Brickwork in Weak Mortars*

11.6.2.1 The fired bricks should have a compressive strength not less than 3.5 MPa. Strength of bricks and wall thickness should be selected for the total building height.

11.6.2.2 The mortar should be lime-sand (1:3) or clay mud of good quality. Where horizontal steel is used between courses, cement-sand mortar (1:3) should be used with thickness so as to cover the steel with 6 mm mortar above and below it. Where vertical steel is used, the surrounding brickwork of 1 x 1 or 1 ½ x 1 ½ brick size depending on wall thickness should preferably be built using 1:6 cement-sand mortar.

11.6.2.3 The minimum wall thickness shall be one brick in one storey construction, and one brick in top storey and 1 ½ brick in bottom storeys of up to three storey construction. It should also not be less than 1/16 of the length of wall between two consecutive perpendicular walls.

11.6.2.4 The height of the building shall be restricted to the following, where each storey height shall not exceed 3.0 m :

For Categories B and C : three storeys with flat roof; and two storeys plus attic for pitched roof.

For Categories D : two storeys with flat roof; and one storey plus attic for pitched roof.

11.6.2.5 *Special bond in brick walls*

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 sloping (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. 20).

11.6.3 *Stone Masonry (Random Rubble or Half-Dressed)*

11.6.3.1 The construction of stone masonry of random rubble or dressed stone type should generally follow good practice [6-4(14)].

11.6.3.2 The mortar should be cement-sand (1:6), lime sand (1:3) or clay mud of good quality.

11.6.3.3 The wall thickness '*t*' should not be larger than 450 mm. Preferably it should be about 350 mm, and the stones on the inner and outer wythes should be interlocked with each other.

NOTE – If the two wythes are not interlocked, they tend to delaminate during ground shaking, bulge apart (*see* Fig. 73) and buckle separately under vertical load leading to complete collapse of the wall and the building.

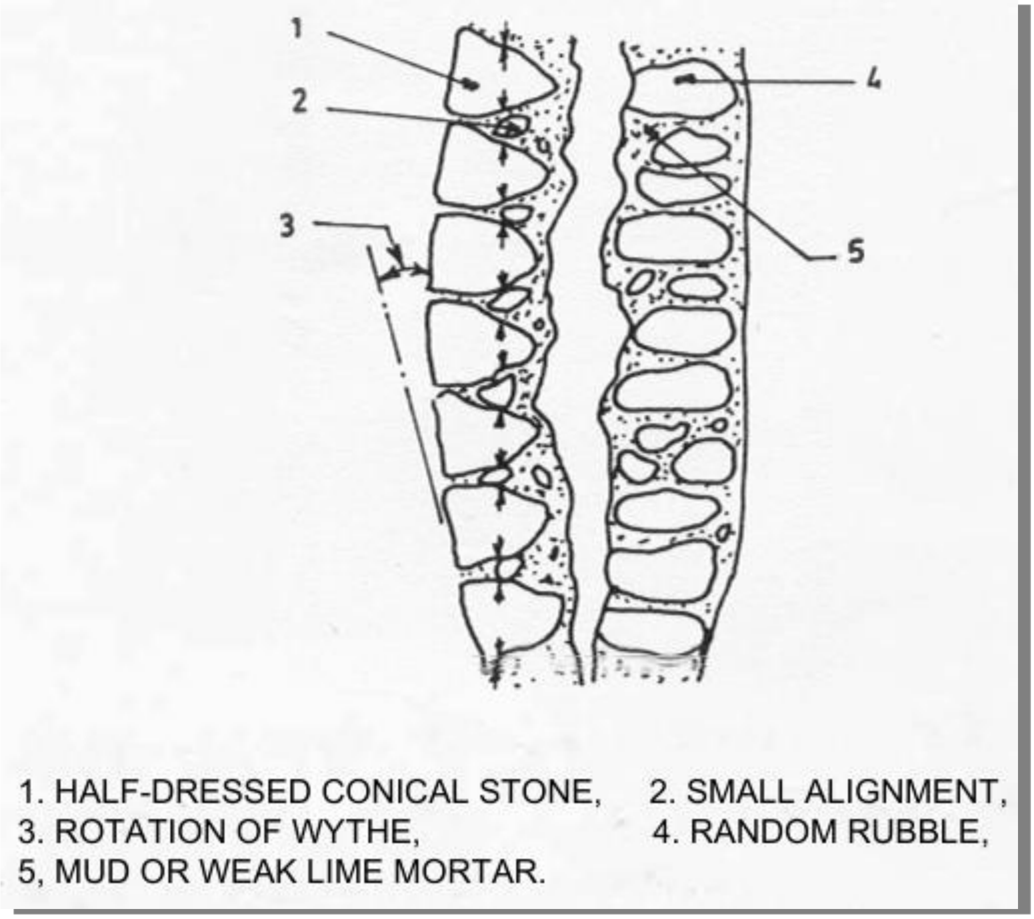
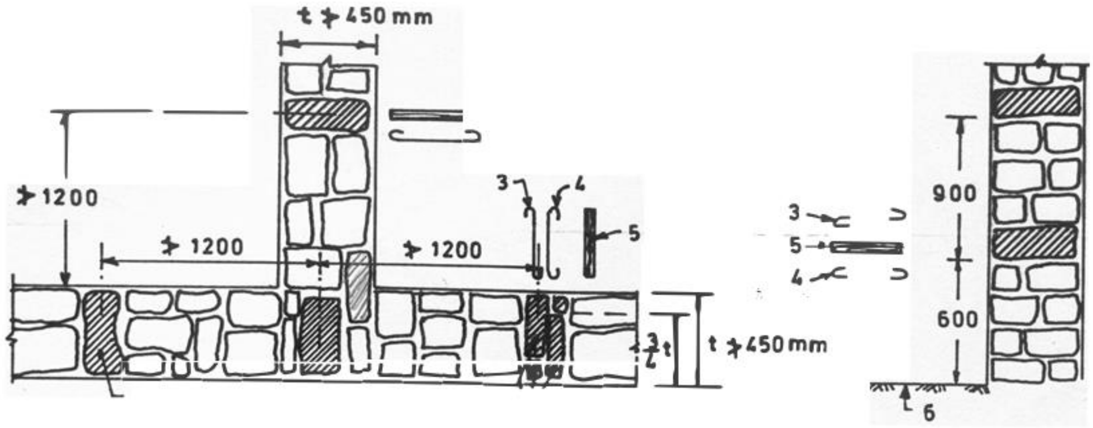


FIG. 73 WALL DELAMINATED WITH BUCKLED WYTHES

11.6.3.4 The masonry should preferably be brought to courses at not more than 600 mm lift.

11.6.3.5 `Through' stones of full length equal to wall thickness should be used in every 600 mm lift at not more than 1.2 m apart horizontally. If full length stones are not available, stones in pairs each of about ¾ of the wall thickness may be used in place of one full length stone so as to provide an overlap between them (*see* Fig. 74).



74A SECTIONAL PLAN OF WALL 74B CROSS-SECTION OF WALL

1. THROUGH STONE, 2, PAIR OF OVERLAPPING STONE,
3. S-SHAPE TIE, 4. HOOKED TIE,
5. WOOD PLANK, 6. FLOOR LEVEL.

All Dimensions in Millimetres

FIG. 74 THROUGH STONE AND BAND ELEMENTS

11.6.3.6 In place of 'through' stones, 'bonding elements' of steel bars 8 mm to 10 mm diameter bent to S-shape or as hooked links may be used with a cover of 25 mm from each face of the wall (*see* Fig. 74). Alternatively, wood bars of 38 mm x 38 mm cross section or concrete bars of 50 mm x 50 mm section with an 8 mm diameter rod placed centrally may be used in place of 'through' stones. The wood should be well treated with preservative so that it is durable against weathering and insect action.

11.6.3.7 Use of 'bonding' elements of adequate length should also be made at corners and junctions of walls to break the vertical joints and provide bonding between perpendicular walls.

11.6.3.8 Height of the stone masonry walls (random rubble or half-dressed) should be restricted as follows, with storey height to be kept 3.0 m maximum, and span of walls between cross walls to be limited to 5.0 m:

a) *For category B* – Two storeys with flat roof or one storey plus attic, if walls are built in lime-sand or mud mortar; and one storey higher if walls are built in cement-sand 1:6 mortar.
b) *For categories C and D* – Two storeys with flat roof or two storeys plus attic for pitched roof, if walls are built in 1:6 cement mortar; and one storey with flat roof or one storey plus attic, if walls are built in lime-sand or mud mortar, respectively.

11.6.3.9 If walls longer than 5 m are needed, buttresses may be used at intermediate points not farther apart than 4.0 m. The size of the buttress be kept of uniform thickness. Top width should be equal to the thickness of main wall, *t*, and the base width equal to one sixth of wall height.

11.6.4 *Opening in Bearing Walls*

11.6.4.1 Door and window openings in walls reduce their lateral load resistance and hence should preferably, be small and more centrally located. The size and position of openings shall be as given in Table 32 and Fig. 21.

11.6.4.2 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.

11.6.4.3 Where openings do not comply with the guidelines of Table 32, they should be strengthened by providing reinforced concrete lining as shown in Fig. 22 with 2 high strength deformed steel bars of 8 mm diameter.

Table 32 Size and Position of Openings in Bearing Walls (*see* Fig. 21)
(*Clauses* 11.6.4.1 *and* 11.6.4.3)

Sl No.	Description	Building Category	
		B and C	D
(1)	(2)	(3)	(4)
i)	Distance b_5 from the inside corner of outside wall, *Min,* mm	230	600
ii)	Total length of openings, ratio; *Max* : $(b_1 + b_2 + b_3)/l_1$ or $(b_6 + b_7)/l_2$ shall not exceed:		
	a) one storeyed building	0.46	0.42
	b) 2 and 3 storeyed building	0.37	0.33
iii)	Pier width between consecutive openings, b_4, mm	450	560
iv)	Vertical distance between two openings one above the other, h_3, *Min*, mm	600	600
v)	Width of opening of ventilator, b_8, *Max*, mm	750	750

11.6.4.4 The use of arches to span over the openings is a source of weakness and shall be avoided, otherwise, steel ties should be provided.

11.6.5 *Seismic Strengthening Arrangements*

11.6.5.1 All buildings to be constructed of masonry shall be strengthened by the methods as specified for various categories of buildings, listed in Table 33 and detailed in subsequent clauses. Fig. 23 and Fig. 24 show, schematically, the overall strengthening arrangements to be adopted for category *D* buildings, which consist of horizontal bands of reinforcement at critical levels and vertical reinforcing bars at corners and junctions of walls.

Table 33 Strengthening Arrangements Recommended for Low Strength Masonry Buildings
(*Clause* 11.6.5.1)

Sl No.	Building Category	Number of Storeys	Strengthening to be Provided
(1)	(2)	(3)	(4)
i)	*B*	1 and 2	*b*,*c*,*f*,*g*
		3	*b*,*c*,*d*,*f*,*g*
ii)	*C*	1	*b*,*c*,*f*,*g*
		2 and 3	*b*,*c*,*d*,*f*,*g*
iii)	*D*	1 and 2	*b*,*c*,*d*,*f*,*g*

where

b – Lintel band (*see* **11.6.5.2**).
c – Roof band and gable band where necessary (*see* **11.6.5.3** and **11.6.5.4**).
d – Vertical steel at corners and junctions of walls (*see* **11.6.5.7**).
f – Bracing in plan at tie level of pitched roofs (*see* **11.3.2.2.2**).
g – Plinth band, where necessary (*see* **11.6.5.6**).

NOTE – For building of category *B* in two storeys constructed with stone masonry in weak mortar, it will be desirable to provide vertical steel of 10 mm dia in both storeys.

11.6.5.2 Lintel band is a band provided at lintel level on all internal and external longitudinal as well as cross walls except partition walls. The details of the band are given in **11.6.5.5**.

11.6.5.3 Roof band is a band provided immediately below the roof or floors. The details of the band are given in **11.6.5.5**. Such a band need not be provided underneath reinforced concrete or reinforced brick slabs resting on bearing walls, provided that the slabs cover the width of end walls fully.

11.6.5.4 Gable band is a band provided at the top of gable masonry below the purlins. The details of the band are given in **11.6.5.5**. This band shall be made continuous with the roof band at the eaves level.

11.6.5.5 *Details of band*

11.6.5.5.1 *Reinforced band*

The band should be made of reinforced concrete of grade not leaner than M15 or reinforced brickwork in cement mortar not leaner than 1:3. The bands should be of full width of the wall, not less than 75 mm in depth and should be reinforced with 2 high strength deformed steel bars of 8 mm diameter and held in position by 6 mm diameter bar links, installed at 150 mm apart as shown in Fig. 25.

NOTES

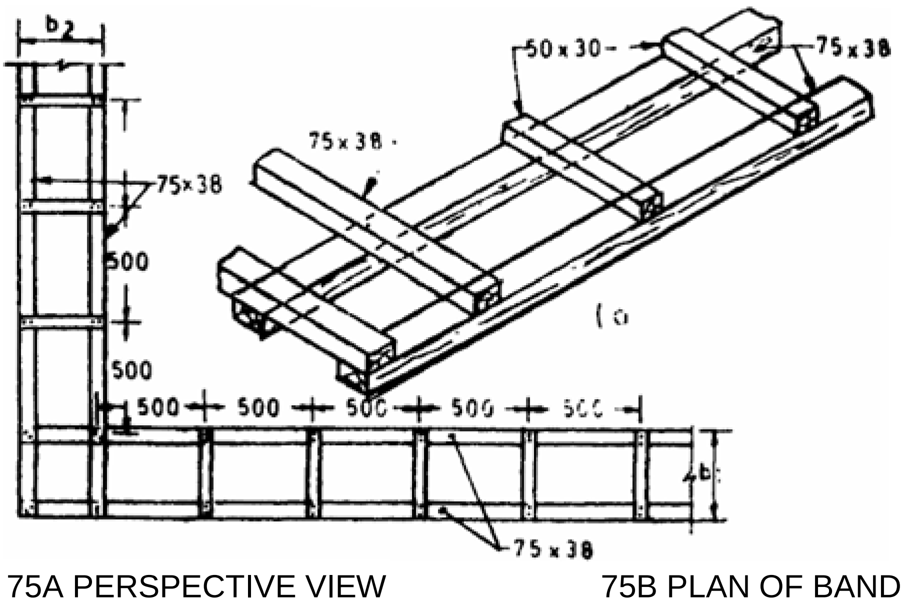
1 In coastal areas, the concrete grade shall be of grade in accordance with Part 6 'Structural design, Section 5 Concrete' of this Code and the filling mortar of 1:3 ratio (cement- sand) with water proofing admixture.

2 In case of reinforced brickwork, the thickness of joints containing steel bars should be increased to 20 mm so as to have a minimum mortar cover of 6 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.

3 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. 25 are recommended.

11.6.5.5.2 *Wooden band*

As an alternative to reinforced band, the lintel band could be provided using wood beams in one or two parallel pieces with cross elements as shown in Fig. 75.



75A PERSPECTIVE VIEW 75B PLAN OF BAND

All Dimensions in Millimetres

FIG. 75 WOODEN BAND FOR LOW-STREGNTH MASONRY BUILDINGS

11.6.5.6 Plinth band is a band 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 same as in **11.6.5.5.1**. This band serves as damp proof course as well.

11.6.5.7 *Vertical reinforcement*

Vertical steel at corners and junctions of walls which are up to 350 mm thick should be provided as specified in Table 34. For walls thicker than 350 mm, the area of the bars should be proportionately increased.

Table 34 Vertical Steel Reinforcement in Low Strength Masonry Walls
(*Clause* 11.6.5.7)

No. of Storeys	**Storey**	**Diameter of HSD Single Bar; in mm, at Each Critical Section for**		
		Category *B*	Category *C*	Category *D*
(1)	(2)	(3)	(4)	(5)
One	-	Nil	Nil	10
Two	a) Top	Nil	10	10
	b) Bottom	Nil	10	12
Three	a) Top	10	10	10
	b) Middle	10	10	12
	c) Bottom	12	12	12

NOTES

1 The diameters given above are for high strength deformed bars with yield strength 415 MPa. For mild-steel plain bars, use equivalent diameters.

2 The vertical bars should be covered with concrete of M15 grade or with mortar 1:3 (cement-sand) in suitably created pockets around the bars (*see* Fig. 26 and Fig. 76). This will ensure their safety from corrosion and good bond with masonry.

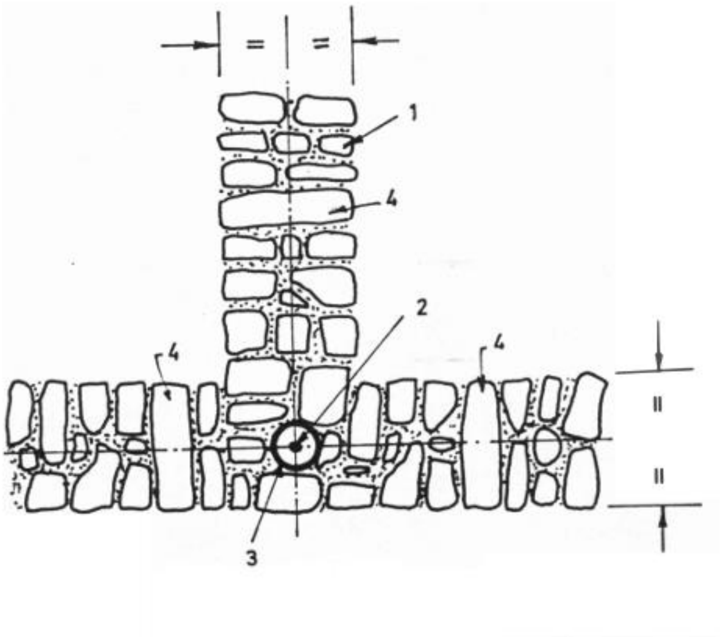
3 For category *B* two storey stone masonry buildings, *see* Note under Table 32.

11.6.5.7.1 The vertical reinforcement should 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 should pass through the lintel bands and floor slabs or floor level bands in all storeys. Bars in different storeys may be welded or suitably lapped.

NOTES

1 Typical details of providing vertical steel in brickwork at corners and T-junctions are shown in Fig. 26.

2 For providing vertical bar in stone masonry, use of a casing pipe is recommended around which masonry be built to height of 600 mm (*see* Fig. 53). The pipe is kept loose by rotating it during masonry construction. It is then raised and the cavity below is filled with M15 (or 1:2:4) grade of concrete mix and rodded to compact it.



1. STONE WALL, 2. VERTICAL STEEL BAR,
3. CASING PIPE, 4, THROUGH STONE OR BONDING ELEMENT.

FIG. 76 TYPICAL CONSTRUCTION DETAIL FOR INSTALLING VERTICAL STEEL BAR IN RANDOM RUBBLS STONE MASONRY

12 MASONRY WALLS USING RAT-TRAP BOND TECHNOLOGY

This type of masonry shall be applicable for masonry structures of not more than 2 storeys constructed in seismic zones II and III. For buildings in other seismic zones, this type of masonry wall shall be used only in non-load bearing applications such as partition walls. The provisions for this type of masonry shall be as given in Annex G.

13 SUSTAINABILITY IN MASONRY CONSTRUCTION

The sustainability of masonry structures is be addressed by quantifying embodied carbon in the structural system and the sources/types of materials used. The NBC deals with sustainability of buildings in Part 11 of the Code. This section gives guidelines for selection of materials for masonry and assessing the embodied carbon in the masonry systems. The guidelines on sustainability given in Part 11 of the Code shall be followed to address the sustainability masonry.

14 NOTATIONS AND SYMBOLS

The various notations and letter symbols used in the text of the Section shall have the meaning as given in Annex H.

ANNEX A
(*Clause* 4.7)

SOME GUIDELINES FOR ASSESSMENT OF ECCENTRICITY OF LOADING ON WALLS

A-1 Where a reinforced concrete roof and floor slab of normal span (not exceeding 30 times the thickness of wall) bear on external masonry walls, the point of application of the vertical loading shall be taken to be at the centre of the bearing on the wall. When the span is more than 30 times the thickness of wall, the point of application of the load shall be considered to be displaced from the centre of bearing towards the span of the floor to an extent of one-sixth the bearing width.

A-2 In case of a reinforced concrete slab of normal span (that is, less than 30 times the thickness of the wall), which does not bear on the full width of the wall and 'cover tiles or bricks' are provided on the external face, there is some eccentricity of load. The eccentricity may be assumed to be one-twelfth of the thickness of the wall.

A-3 Eccentricity of load from the roof/floor increases with the increase in flexibility and thus deflection of the slabs. Also, eccentricity of loading increases with the increase in fixity of slabs/beams at supports. Precast RCC slabs are better than *in-situ* slabs in this regard because of very little fixity. If supports are released before further construction on top, fixity is reduced.

A-4 Interior walls carrying continuous floors are assumed to be axially loaded except when carrying very flexible floor or roof systems. The assumption is valid also for interior walls carrying independent slabs spanning from both sides, provided the span of the floor on one side does not exceed that on the other by more than 15 percent. Where the difference is greater, the displacement of the point of application of each floor load shall be taken as one-sixth of its bearing width on the wall and the resultant eccentricity calculated therefrom.

A-5 For timber and other lightweight floors, even for full width bearing on wall, an eccentricity of about one-sixth may be assumed due to deflection. For timber floors with larger spans, that is, more than 30 times the thickness of the wall, eccentricity of one-third the thickness of the wall may be assumed.

A-6 In multi-storeyed buildings, fixity and eccentricity have normally purely local effect and are not cumulative. They just form a constant ripple on the downward increasing axial stress. If the ripple is large, it is likely to be more serious at upper levels where it can cause cracking of walls than lower down where it may or may not cause local over-stressing.

NOTE – The resultant eccentricity of the total loads on a wall at any level may be calculated on the assumption that immediately above a horizontal lateral support, the resultant eccentricity of all the vertical loads above that level is zero.

A-7 For a wall corbel to support some load, the point of application of the load shall be assumed to be at the centre of the bearing on the corbel.

ANNEX B
(*Clause* 5.4.1)

CALCULATION OF BASIC COMPRESSIVE STRESS OF MASONRY BY PRISM TEST

B-1 DETERMINATION OF COMPRESSIVE STRENGTH OF MASONRY BY PRISM TEST

When compressive strength of masonry (f_m) is to be established by tests, it shall be done in advance of the construction, using prisms built of similar materials under the same conditions with the same bonding arrangement as for the structure. In building the prisms, 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. Assembled specimen shall be at least 400 mm high and shall have a height to thickness ratio (h/t) of at least 2 but not more than 5. If the h/t ratio of the prisms tested is less than 5 in case of brickwork and more than 2 in case of blockwork, compressive strength values indicated by the tests shall be corrected by multiplying with the factor indicated in Table 35.

Table 35 Correction Factors for Different *h*/*t* Ratios
(*Clause* B-1)

Ratio of height to thickness (*h/t*)	2.0	2.5	3.0	3.5	4.0	5.0
Correction factors for brickwork[1)]	0.73	0.80	0.86	0.91	0.95	1.00
Correction factors for blockwork[1)]	1.00	-	1.20	-	1.30	1.37

[1)] Interpolation is valid for intermediate values.

Prisms shall be tested after 28 days between sheets of nominal 4 mm plywood, slightly longer than the bed area of the prism, in a testing machine, the upper platform of which is spherically seated. The load shall be evenly distributed over the whole top and bottom surfaces of the specimen and shall be applied at the rate of 350 kN/m to 700 kN/m. The load at failure should be recorded.

B-2 CALCULATION OF BASIC COMPRESSIVE STRESS

Basic compressive stress of masonry shall be taken to be equal to 0.25 f_m where f_m is the value of compressive strength of masonry as obtained from prism test.

ANNEX C

(*Clauses* 5.3.3 *and* 5.4.1.5)

GUIDELINES FOR DESIGN OF MASONRY SUBJECTED TO CONCENTRATED LOADS

C-1 EXTENT OF DISPERSAL OF CONCENTRATED LOAD

For concentric loading, maximum spread of a concentrated load on a wall may be taken to be equal to b+4t (b is width of bearing and t is thickness of wall), or stretch of wall supporting the load, or centre-to-centre distance between loads, whichever is less.

C-2 INCREASE IN PERMISSIBLE STRESS

C-2.1 When a concentrated load bears on a central strip of wall, not wider than half the thickness of the wall and is concentric, bearing stress in masonry may exceed the permissible compressive stress by 50 percent, provided the area of supporting wall is not less than three times the bearing area.

C-2.2 If the load bears on full thickness of wall and is concentric, 25 percent increase in stress may be allowed.

C-2.3 For loading on central strip wider than half the thickness of the wall but less than full thickness, increase in stress may be worked out by interpolation between values of increase in stresses as given in **C-2.1** and **C-2.2**.

C-2.4 In case concentrated load is from a lintel over an opening, an increase of 50 percent in permissible stress may be taken, provided the supporting area is not less than 3 times the bearing area.

C-3 CRITERIA OF PROVIDING BED BLOCK

C-3.1 If a concentrated load bears on one end of a wall, there is a possibility of masonry in the upper region developing tension. In such a situation, the load should be supported on an RCC bed block (of M20 Grade) capable of taking tension.

C-3.2 When any section of masonry wall is subjected to concentrated as well as uniformly distributed load and resultant stress, computed by making due allowance for increase in stress on account of concentrated load, exceeds the permissible stress in masonry, a concrete bed block (of M20 Grade) should be provided under the load in order to relieve stress in masonry. In concrete, angle of dispersion of concentrated load is taken to be 45° to the vertical.

C-3.3 In case of cantilevers and long span beams supported on masonry walls, indeterminate but very high edge stresses occur at the supports and in such cases it is necessary to relieve stress on masonry by providing concrete bed block of M20 Grade concrete. Similarly when a wall is subjected to a concentrated load from a beam which is not sensibly rigid (for example, a timber beam or an RS joist), a concrete bed block should be provided below the beam in order to avoid high edge stress in the wall because of excessive deflection of the beam.

ANNEX D
(*Clause* 5.5.5)

GUIDELINES FOR APPROXIMATE DESIGN OF NON-LOAD BEARING WALL

D-1 PANEL WALLS

A panel wall may be designed approximately as under, depending upon its support conditions and certain assumptions:

a) When there are narrow tall windows on either side of panel, the panel spans in the vertical direction. Such a panel may be designed for a bending moment of $PH/8$, where P is the total horizontal load on the panel and H is the height between the centres of supports. Panel wall is assumed to be simply supported in the vertical direction.

b) When there are long horizontal windows between top support and the panel, the top edge of the panel is free. In this case, the panel should be considered to be supported on sides and at the bottom, and the bending moment should depend upon height to length ratio of panel and flexural strength of masonry. Approximate values of bending moments in the horizontal direction for this support condition, when ratio (μ) of flexural strength of wall in the vertical direction to that in horizontal direction is assumed to be 0.5, are given in Table 36.

Table 36 Bending Moments in Laterally Loaded Panel Walls, Free at Top Edge and Supported on Other Three Edges
[*Clause* D-1 (b)]

$\frac{\text{Height of Panel, } H}{\text{Length of Panel, } L}$	0.30	0.50	0.75	1.00	1.25	1.50	1.75
Bending Moment	$\frac{P.L}{25}$	$\frac{P.L}{18}$	$\frac{P.L}{14}$	$\frac{P.L}{12}$	$\frac{P.L}{11}$	$\frac{P.L}{10.5}$	$\frac{P.L}{10}$

NOTE – For H/L ratio less than 0.30, the panel should be designed as a free-standing wall and for H/L ratio exceeding 1.75, it should be designed as a horizontally spanning member for a bending moment value of $PL/8$.

c) When either there are no window openings or windows are of 'hole-in-wall' type, the panel is considered to be simply supported on all four edges. In this case also, amount of maximum bending moment depends on height to length ratio of panel and ratio (μ) of flexural strength of masonry in vertical direction to that in the horizontal direction. Approximate values for maximum bending moment in the horizontal direction for masonry with μ = 0.50, are given in Table 37.

Table 37 Bending Moments in Laterally Loaded Panel Walls Supported on All Four Edges
[*Clause* D-1 (c)]

$\frac{\text{Height of Panel, } H}{\text{Length of Panel, } L}$	0.30	0.50	0.75	1.00	1.25	1.50	1.75
Bending Moment	$\frac{P.L}{72}$	$\frac{P.L}{36}$	$\frac{P.L}{24}$	$\frac{P.L}{18}$	$\frac{P.L}{15}$	$\frac{P.L}{13}$	$\frac{P.L}{12}$

NOTE – When *H*/*L* is less than 0.30, value of bending moment in the horizontal direction may be taken as nil and panel wall may be designed for a bending moment value of *PH*/8 in the vertical direction; when *H/L* exceeds 1.75, panel may be assumed to be spanning in the horizontal direction and designed for bending moment of *PL*/8.

D-2 CURTAIN WALLS

Curtain walls may be designed as panel walls taking into consideration the actual supporting conditions.

D-3 PARTITION WALLS

D-3.1 These are internal walls usually subjected to much smaller lateral forces. Behaviour of such wall is similar to that of panel wall and these could, therefore, be designed on similar lines. However, in view of smaller lateral loads, ordinarily these could be apportioned empirically as follows:

a) Walls with adequate lateral restraint at both ends but not at the top :

   1) The panel may be of any height, provided the length does not exceed 40 times the thickness; or
   2) The panel may be of any length, provided the height does not exceed 15 times the thickness (that is, it may be considered as a free-standing wall); or
   3) Where the length of the panel is over 40 times and less than 60 times the thickness, the height plus twice the length may not exceed 135 times the thickness;

b) Walls with adequate lateral restraint at both ends at the top :

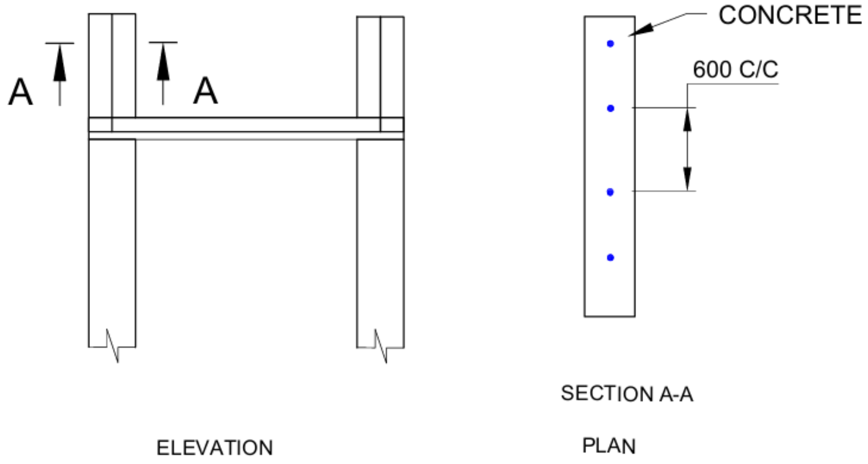
   1) The panel may be of any height, provided the length does not exceed 40 times the thickness; or
   2) The panel may be of any length, provided the height does not exceed 30 times the thickness; or
   3) Where the length of the panel is over 40 times and less than 110 times the thickness, the length plus three times the height should not exceed 200 times the thickness; and

c) When walls have adequate lateral resistant at the top but not at the ends, the panel may be of any length, provided the height does not exceed 30 times the thickness.

D-3.2 Strength of bricks used in partition walls should not be less than 3.5 N/mm^2 or the strength of masonry units used in adjoining masonry, whichever is less. Grade of mortar should not be leaner than M2.

D-4 CANTILEVER PROJECTIONS

D-4.1 The provisions given under **5** of Part 6 'Structural Design, Section 1 Loads, Forces and Effects' of the Code shall be applicable. As a general guide, the details of connection between parapets and slabs for existing structures shall be as given in Fig. 77.



NOTES
1. HOLE SHALL BE MADE IN BRICKS IN ADVANCE TO ACCOMMODATE THE REINFORCEMENT BARS OF SLAB.
2. BOTTOM BAR OF SLAB CONTINUED IN THE PARAPET AT SPACING OF 600 mm c/c.

Fig. 77 PARAPET WALLS

ANNEX E
(*Clause* 10.6.1.2)

TYPICAL ITERATIVE METHOD FOR WALLS SUBJECTED TO FLEXURE AND AXIAL LOAD

E-1 There is no closed form solution available for the computation of stress in walls due to combined action of bending and axial compression. Engineers normally prefer to use Design charts for computation of stresses in such conditions, but design charts for most masonry systems are not readily available. Therefore, iterative methods are used as most common tools for masonry structures. Two types of iterative procedures are given below, which can be used to analyse the adequacy of the section under specified load and determine the amount of reinforcement required, wherever necessary.

E-2 ITERATIVE PROCEDURE 1

Figure 78 shows a diagram of a wall subjected to an axial load and bending moment. The same diagram is applicable to both in-plane and out-of-plane bending of walls. There are three possible conditions for the wall:

a) Wall is uncracked,
b) Wall is cracked with the steel in compression (the extent of the crack has not reached the steel), or
c) Wall is cracked with the steel in tension.

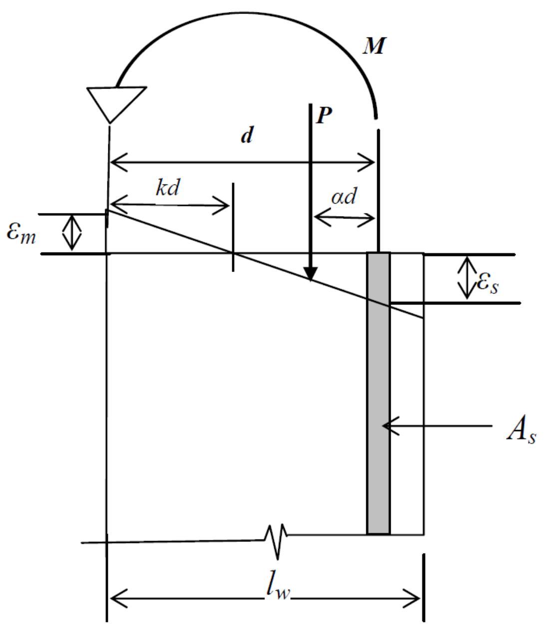


FIG. 78 FLEXURE AND AXIAL WALL LOADING

Figure 78 shows the interaction between bending moment and axial load. Loading conditions beyond the limits of the diagram are beyond the allowable stresses. The condition of the wall can be quickly diagnosed by using the non-dimensional parameter *M/Pd*. This parameter represents a straight line radiating from the diagram's origin as shown in Fig. 79. By summing moments, it can be shown that certain values of *M/Pd* divide the diagram into three regions. These values are given in Table 34. The allowable moments for regions 1 and 2 can be obtained in closed form with simple equations. But in region 3, it is complicated a bit. The process consists of initial assumption of thickness, length and other necessary parameters and determination of the amount of steel required.

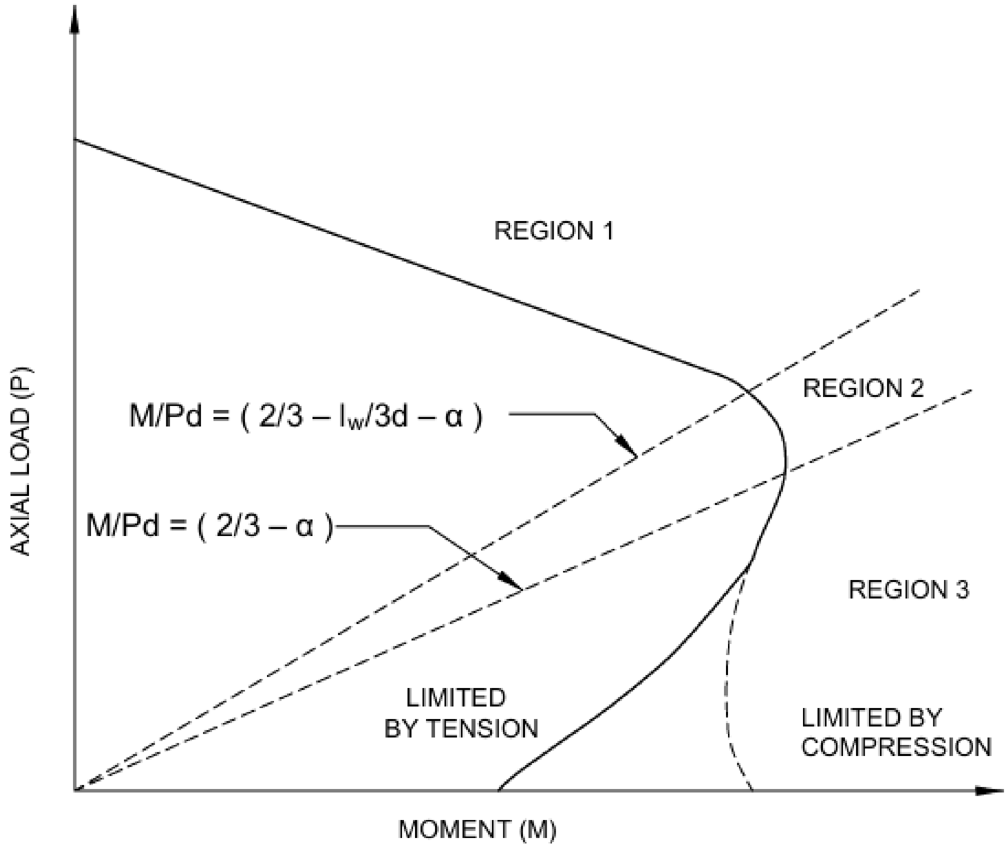


FIG. 79 INTERACTION DIAGRAM

For Region 3, the process starts with another assumption that the tension in steel controls. By making an initial guess about the location of the neutral axis, the expressions for M_p and a_2 provide an iterative process that quickly converges. The equations are derived from the summation of forces and moments and by using linear stress-strain relationships. The equations should always converge for all values of P, l_w, M, and n. If convergence is not obtained, it is likely that the assumed initial value of a has exceeded d. The initial value of a is to be reduced and same procedure is to be repeated. The iteration will often result in a negative steel area. This signifies that the element does not need reinforcement to resist the loads within the allowable stresses, even though the section is cracked (Region 3).

Following convergence, a check is made of the initial assumption that tension in steel controls. If not valid, then the compression controls and a new set of equations that do not require iteration are used.

E-3 The following is a detailed step-by-step procedure of the analysis methodology described above:

a) Step 1:

Determine the wall condition:

1) Calculate α as the distance from the axial load to the centroid of the tension steel divided by *d*.
2) Calculate the quantity M/*Pd*.
3) Use Table 38 to determine the region for analysis. (*see* Fig. 78)

Table 34 Flexure and Axial loading - Wall Analysis
(Clause E-3)

Sl No. (1)	Region (2)	Condition of Wall (3)	Test (4)
i)	1	Wall is in compression and not cracked	$\frac{M}{Pd} \leq \left[1 - \frac{l_w}{3d} - \alpha\right]$
ii)	2	Wall is cracked but steel is in compression	$\frac{M}{Pd} \leq \left[\frac{2}{3} - \alpha\right]$
iii)	3	Wall is cracked and steel is in tension	$\frac{M}{Pd} > \left[\frac{2}{3} - \alpha\right]$

b) Step 2:

Calculate the allowable moment.

1) Region 1:

The moment is limited by flexural compression in the masonry.

$$M_m = \frac{b l_w^2}{6} F_b - P \frac{l_w}{6}$$

If M_m is greater than M applied, the section is satisfactory.

2) Region 2:

The moment is also limited by flexural compression in the masonry.

$$M_m = P(1-\alpha)d - \frac{2}{3}\left[\frac{P^2}{F_b b}\right]$$

If M_m is greater than M applied, the section is satisfactory.

3) Region 3:

The moment may be limited by either the compression in the masonry or tension in the steel. An iterative approach as below shall be adopted.

i) Assume a compression centroid location, *a*.
ii) Perform the following iteration that assumes the tension in the steel controls (remember that A_s may be negative):

$$M_p = P\left(\frac{l_w}{2} - \alpha\right)$$

$$A_s = \frac{M - M_p}{F_s(d-a)}$$

$$\zeta = \frac{\left(P + A_s F_s)n\right)}{F_s b}$$

$$a_2 = \frac{\sqrt{\zeta^2 + 2\zeta d} - \zeta}{3}$$

where

a =estimate of internal compression load centroid distance from the extreme compression fibre.
P = applied axial load at the center of the wall.
M = applied moment.
A_s = area of trim or edge steel.
M_p = moment of applied axial load with respect to the centroid of internal compression force.
l_w = width of the wall for in-plane bending (thickness of wall for out of plane bending).
d = distance from the extreme compression fiber to the steel centroid.
F_s = allowable steel tension stress.
b = width of the wall for out-of-plane bending (thickness of wall for in-plane bending).

Use a_2 for a, and repeat until the value of a converges.

c) Step 3:

If the iteration converges and the resulting *a* is less than the following value, the wall is limited by the tension reinforcement and the analysis is complete. Otherwise continue to step 4.

$$a = \frac{d}{3\left(1 + \frac{F_s/F_b}{n}\right)}$$

d) Step 4:

If the value of *a* is larger than the above value, determine the required steel area using the following:

$$a = \frac{d}{2} - \sqrt{\left(\frac{d^2}{4} - \frac{2(P\alpha d + M)}{3F_b b}\right)}$$

If $\sqrt{\left(\frac{d^2}{4} - \frac{2(P\alpha d + M)}{3F_b b}\right)}$ is negative, there is inadequate compression capacity.

Increase *b* or F_b or both. The steel area is:

$$A_s = \frac{\left(\frac{3F_b ab}{2} - P\right)}{\left[nF_b\left(\frac{d}{3a} - 1\right)\right]}$$

In all cases the wall trim steel should not be less than minimum values.

E-4 ITERATIVE PROCEDURE

The following is another procedure to determine the state of stress in wall elements subjected to combined action of bending and axial compression. Free body diagram of a wall with axial load, bending moment, and tensile and compressive forces is given in Fig. 80. The location of *C* is based on the neutral axis location *kd*. The location of *T* is at the centroid of the reinforcement.

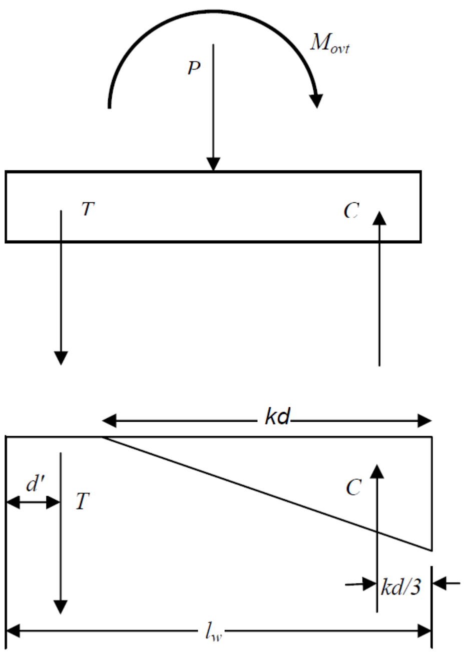


FIG. 80 FLEXURE BODY DIAGRAM OF A WALL

The problem can be solved by taking moments about the centerline of the wall, the tensile reinforcement (assuming that masonry compressive stresses control), or the compressive force (assuming that the stresses in the reinforcement control). By inserting the allowable value of stresses, which is assumed to control, a quadratic equation in terms of *kd* may be developed and solved.

For example if *P* is at the centerline of the wall, taking moments about the centroid of the tensile reinforcement results in the equation:

$$C\left[\left(l_w - d'\right) - \left(\frac{kd}{3}\right)\right] = P\left(\frac{l_w}{2} - d'\right) + M$$

$$\text{but } C = \tfrac{1}{2} F_m bkd \text{ and } l_w - d' = d. \text{ Then:}$$

$$\tfrac{1}{2} F_m bkd\left(d - \frac{kd}{3}\right) = P\left(\frac{l_w}{2} - d'\right) + M$$

Dividing both sides by $\tfrac{1}{2} F_m b$ results in

$$kd^2 - \frac{(kd)^2}{3} = \frac{P\left(\frac{l_w}{2} - d'\right) + M}{\tfrac{1}{2} F_m b}$$

Multiplying by -3 and rearranging terms:

$$(kd)^2 - 3d(kd) + \frac{3\left[P\left(\frac{l_w}{2} - d'\right) + M\right]}{\tfrac{1}{2} F_m b} = 0$$

$$kd = \frac{3d - \sqrt{(-3d)^2 - 4\left(\dfrac{3\left[P\left(\frac{l_w}{2} - d'\right) + M\right]}{\tfrac{1}{2} F_m b}\right)}}{2}$$

Inserting the values for *P, M,* l_w*, b, d* and F_m yields *kd*. Note that this method is iterative since a location of *d′* shall be assumed.

Having solved for *kd* one can calculate *C.*

By $\Sigma F_v = 0$, $T = C - P$

From strain compatibility,

$f_s = \left(\frac{1-k}{k}\right) nF_m$, Thus f_s can be calculated.

If $f_s < F_s$, the masonry compressive stress controls and the solution is finished by calculating $A_s = T/f_s$. If however, $f_s > F_s$ the assumption that masonry stresses control is incorrect, and the neutral axis location is wrong.

A good starting assumption to calculate f_m is to take T assuming that masonry compressive stresses control, and to divide by F_s to get a trial A_s. This allows the designer to choose a bar layout and verify the initial assumption of d', the distance to the centroid of reinforcement.

$$A_s = T/F_s$$

Since $C = T + P$ the effects of the axial load can be included by using an effective amount of reinforcement.

$$(A_s)_{eff} = A_s + \frac{P}{F_s} \ (or \ \frac{T+P}{F_s})$$

$$\rho_{eff} = \frac{(A_s)_{eff}}{bd}$$

$$k = \left[(n\rho)^2 + 2n\rho\right]^{\frac{1}{2}} - n\rho$$

$$j = 1 - \frac{k}{3}$$

The steel stress may be checked using

$$f_s = \frac{M'}{(A_s)_{eff}\ jd}$$

where,

$$M' = P\left(\frac{l_w}{2} - d'\right) + M$$

$$T_{eff} = (A_s)_{eff} \times f_s$$

$$T = T_{eff} - P$$

The masonry stress may be checked by using:

$$f_m = \frac{2M'}{bjkd^2}$$

If $f_s > F_s$, a second iteration may be required may be required with a larger bar size or an extra bar. In a shear wall if an extra bar is added, the value of d', and the moments calculated using,

$$P\left(\frac{l_w}{2} - d'\right) + M$$

will need to be re-evaluated. Then one can reiterate from,

$$(A_s)_{eff} = A_s + \frac{P}{F_s}$$

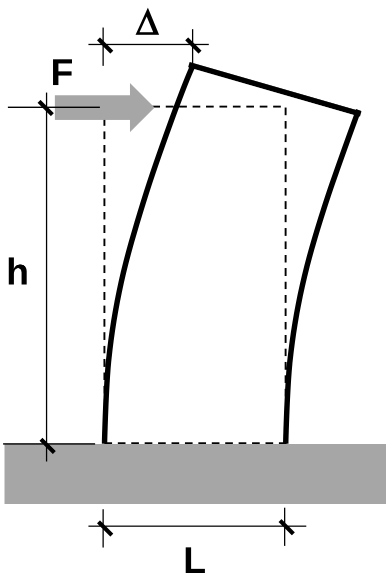
to reduce f_s so that it is less than F_s.

ANNEX F

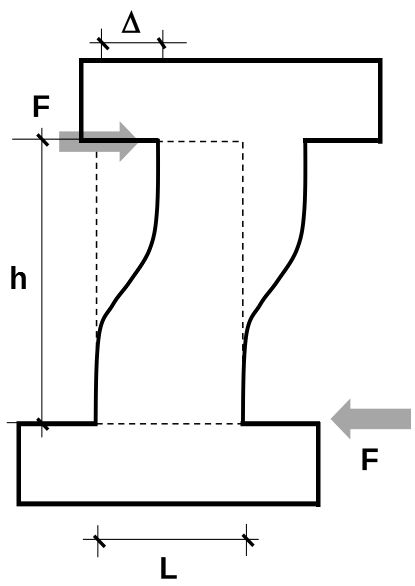
(*Clause* 10.7.1)

DISTRIBUTION OF LATERAL FORCES IN THE PIERS OF A MASONRY WALL

F-1 Masonry walls are often seen to be perforated to make arrangement for windows and doors. The distribution of lateral force in a masonry wall is dependent on the position of the openings and the relative rigidity of the masonry piers created due to the presence of the openings in the masonry wall. The relative rigidity is dependent on the height by length ratio (*h*/*L* Ratio) of the piers and the end conditions of those masonry piers as the deflection of the masonry piers due to horizontal loading changes due to the end condition of the piers (*see* Fig. 81). Here is a simple process is described which can be used to distribute the lateral force in a wall which can be considered to be consist of some piers with some specific arrangements.



81A CANTILEVERED WALL OR PIER (TOP FREE TO ROTATE)



81B PIER BETWEEN OPENINGS (TOP RESTRAINED FROM ROTATION)

FIG. 81 DEFLECTION DUE TO END CONDITIONS OF PIERS

F-2 In any kind of placing of opening, the wall can be represented as a horizontal and vertical combination of piers with their respective end condition which will be used to find out their rigidities. Where large openings occur, it is difficult to obtain effective coupling of the wall segments or piers. If the wall is analysed as a horizontal combination of piers as shown in the Fig. 82 (a) and the combined rigidity

$$R = R_{c1} + R_{c2} + R_{c3}$$

where R_{c1} , R_{c2} , R_{c3} are the rigidities of the piers 1, 2, 3, respectively.

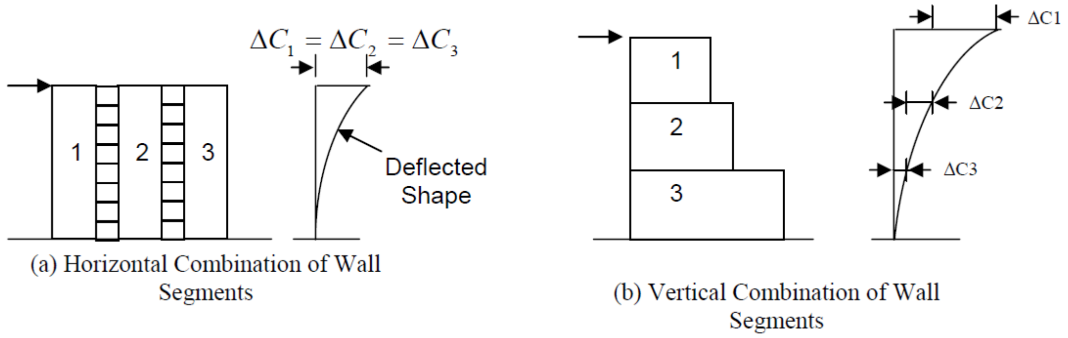


FIG. 82 WALL COMBINBATIONS FOR CALCULATING RIGIDITIES OF WALLS WITH OPENINGS

If the segments are combined vertically, as shown in Fig. 82 (b), the combined rigidity can be calculated as:

$$R = \frac{1}{\Delta C_1 + \Delta C_2 + \Delta C_3} = \frac{1}{\frac{1}{R_{c1}} + \frac{1}{R_{c2}} + \frac{1}{R_{c3}}}$$

Where R_{c1} , R_{c2} , R_{c3} are the rigidities of the piers 1, 2, 3 respectively. Combination of these two types can be used to find the effective relative stiffness of a masonry wall.

The expression ignores the rotations that occur at the tops of segment 2 and 3 and therefore overestimates the rigidity of the wall. Along with this, it is valid only for the application of loads at the top level of the building.

As already said the rigidity *R* of the pier is dependent on its dimensions, modulus of elasticity *E,* modulus of rigidity *G*, and the support conditions. For a cantilever pier the displacement due to combined action of bending and shear is

$$\Delta_c = \frac{Fh^3}{3EI} + \frac{1.2Fh}{GA}$$

Where

- *F* = horizontal force applied to the pier,
- *h* = height of the pier,
- *I* = moment of inertia of pier = $tL^3/12$,
- *G* = modulus of Rigidity = $0.4E_m$,
- *A* = area of the pier = *Lt,*
- *L* = length of the pier, and
- *t* = thickness of pier.

So, displacement of the cantilever pier is

$$\Delta C = \frac{1}{Et}\left[4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]$$

The rigidity of a wall is proportional to the inverse of the deflection. For cantilever walls the rigidity will be:

$$R_c = \frac{1}{\Delta C} = \frac{Et}{\left[4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]}$$

$$\Rightarrow \frac{R_c}{Et} = \frac{1}{\left[4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]}$$

For a pier with both ends fixed against rotation the deflection due to combined action of bending and shear will be

$$\Delta f = \frac{Fh^3}{12EI} + \frac{1.2Fh}{GA} = \frac{F}{Et}\left[\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]$$

$$\Rightarrow \frac{R_f}{E_t} = \frac{1}{\left[\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right]}$$

Values of R/E_t for different h/L ratio can be taken from the chart in the Fig. 83.

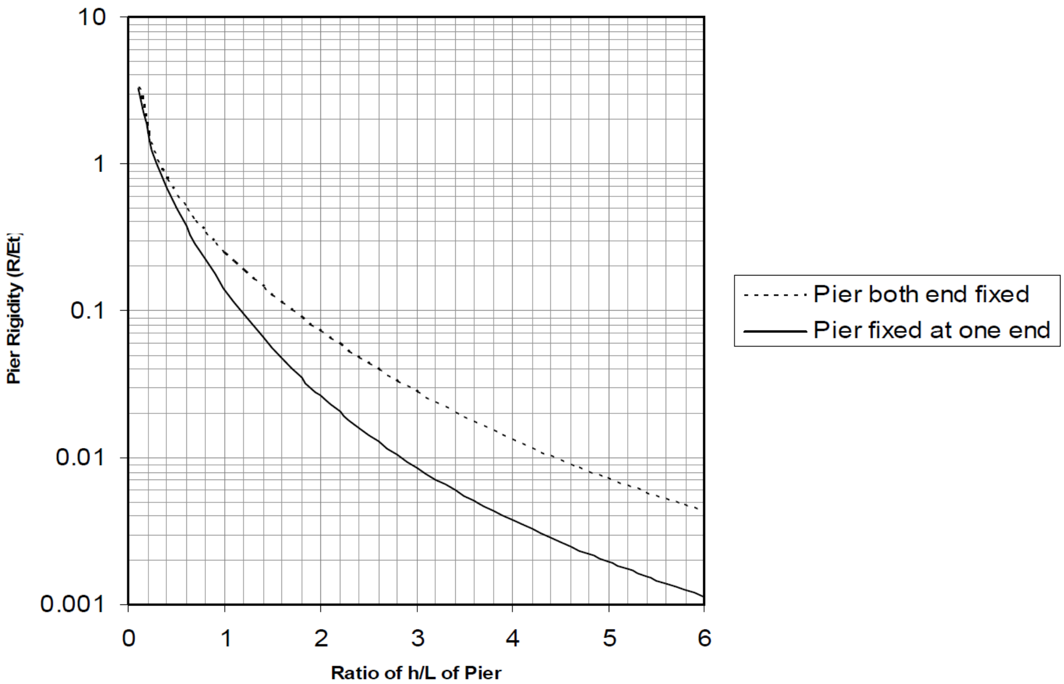


FIG. 83 CHARTS FOR CALCULATING WALL RIGIDITIES

From the relations given in equations above it is found that the relative contributions of the bending and shear deformation depend on the wall aspect ratio (*h/L*) and therefore, the rigidity varies over the height of the building. For high *h/L* ratios, the effect of shear deformation is very small and calculation of pier rigidities based on flexural stiffness is relatively accurate. For very squat walls (with *h/L< 0.25*), rigidities based on shear deformation are reasonably accurate, but for intermediate walls with *h*/*L* from 0.25 to 4, both components of relative rigidity should be considered.

F-3 The method is explained further explained by one illustration. For the wall as shown in Fig. 84,

$$R_{\text{wall}} = \frac{1}{\Delta_{1c} + \Delta_{2,3,4,5,6(\text{f})}} = \frac{1}{\frac{1}{R_{1c}} + \frac{1}{R_{2,3,4,5,6(\text{f})}}} \quad \text{(Vertical combination)}$$

where

$$R_{2,3,4,5,6(f)} = R_{2(f)} + R_{3,4,5,6(f)} \quad \text{(Horizontal combination)}$$

and $R_{3,4,5,6(f)} = \frac{1}{\Delta_{3,4,5(\text{f})} + \Delta_{6(\text{f})}} = \frac{1}{\frac{1}{R_{3,4,5(\text{f})}} + \frac{1}{R_{6(\text{f})}}}$ (Vertical combination)

$$R_{3,4,5(\text{f})} = R_{3(\text{f})} + R_{4(\text{f})} + R_{5(\text{f})} \quad \text{(Horizontal combination)}$$

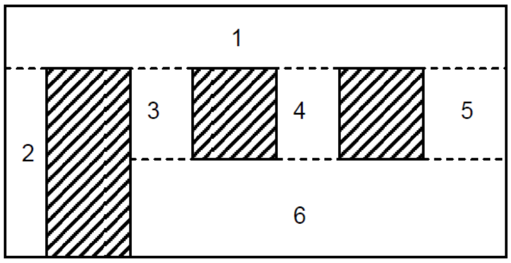


FIG. 84 WALL WITH OPENINGS (CALCULATION OF RIGIDITY AND LOAD DISTRIBUTION)

ANNEX G
(*Clause* 12)

MASONRY WALLS USING RAT-TRAP BOND

G-1 GENERAL

The following covers the construction of masonry walls using rat-trap bond. Only masonry with bricks (burnt clay bricks, fly ash-clay bricks, fly ash-lime bricks and stabilized soil blocks) is dealt herein. Use of other types of bricks/blocks and mud mortar are not specifically covered hereunder.

G-2 NECESSARY INFORMATION

G-2.1 For efficient planning, design and execution of the work, detailed information with regard to the following shall be furnished to those responsible for the work:

a) Layout plans showing the orientation of the structure;
b) Dimensioned details of the structure with details of sections (to a suitably large scale that is 1:20), levels of foundation, finished ground level, clear floor to floor heights of rooms, sizes of openings, etc.
c) Type and class of brickwork, types of bond and final finish for the brickwork; the mixes of mortar to be used, etc; full size details of architectural features, mouldings and other special work. Also course plans at different levels of sill and lintel to be made available for proper understanding and execution, if required.
d) Location and other details of openings, chases, embedment of service lines, such as for water supply, drainage, electrical installations, etc; and location and details of hearths, flues and chimneys in the brickwork.

G-2.2 All information as in **G-2.1** shall be made available to those who are responsible for the masonry work. Necessary drawings and instructions for planning the work shall be furnished. Arrangements shall also be made for the proper exchange of information between those engaged in masonry work and all those whose work will affect or will be affected.

NOTE — If necessary a brief training of the masons may be carried out orienting them with the technology know-how.

G-3 MATERIALS

G-3.1 Bricks

Burnt-clay bricks, heavy duty bricks and bricks made out of other suitable materials like fly ash, stabilized soil block, etc, conforming to accepted standards [6-4(21)] with compressive strength greater than 6.0 MPa shall be used.

G-3.2 Mortars

Shall conform to grade H2 mortar for good practice [6-4(3)].

G-3.3 Storage of Materials

Storage of materials shall be in accordance with good practice [6-4(1)].

G-3.4 Materials for Damp-Proof Courses

Materials for damp proof courses shall be as specified in [6-4(5)].

G-3.5 Materials for Flashing and Weathering

Lead flashing with sheet thickness between 1.6 and 2 mm or bitumen felts shall be in accordance with the accepted standards [6-4(22)].

G-3.6 Metal Reinforcement

Metal reinforcement for use in brick masonry shall be as per accepted standards [6-4(23)].

G-4 DESIGN CONSIDERATIONS

G-4.1 Selection of Bricks

The brick shall be selected in accordance with Table 1 of good practice [6-4(4)].

When the requirements for strength of masonry predominate in the particular situation of use, the bricks shall be of such class to give the required strength for masonry and shall be selected in accordance with **4**.

G-4.2 Rat-Trap Bond Details

G-4.2.1 The rat-trap bond is laid by placing the bricks on edge forming a cavity of 80 mm (for a traditional brick size) with alternate course of stretchers and headers (Fig. 85). The headers and stretchers are staggered in subsequent layers to impart strength and stability to the walls.

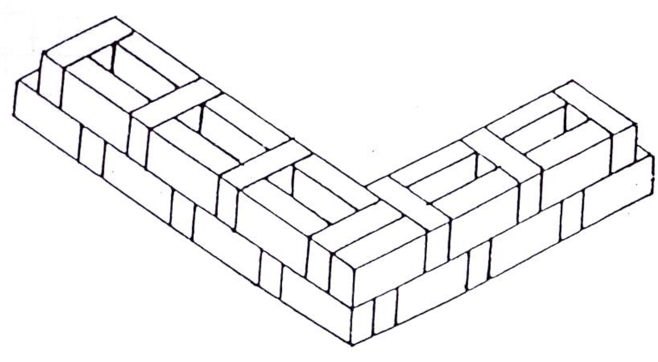
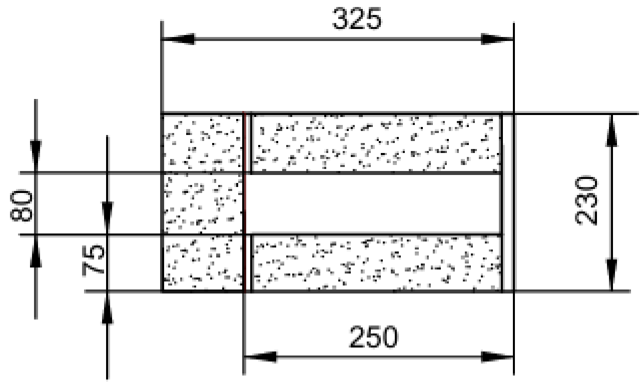
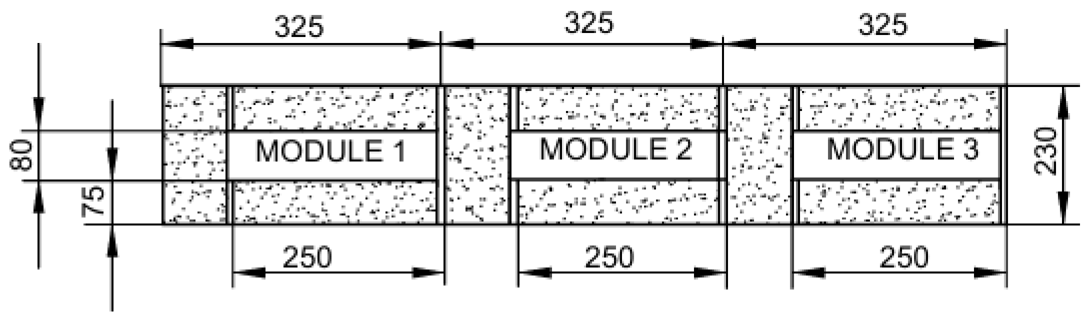


FIG. 85 ARRANGEMENT IN RAT-TRAP BOND

The primary object of bond is to give strength to masonry, but when exposed, it also creates aesthetically pleasing view. This masonry technology is modular in nature, so the dimensions of the wall to be based on the module dimensions in multiples.

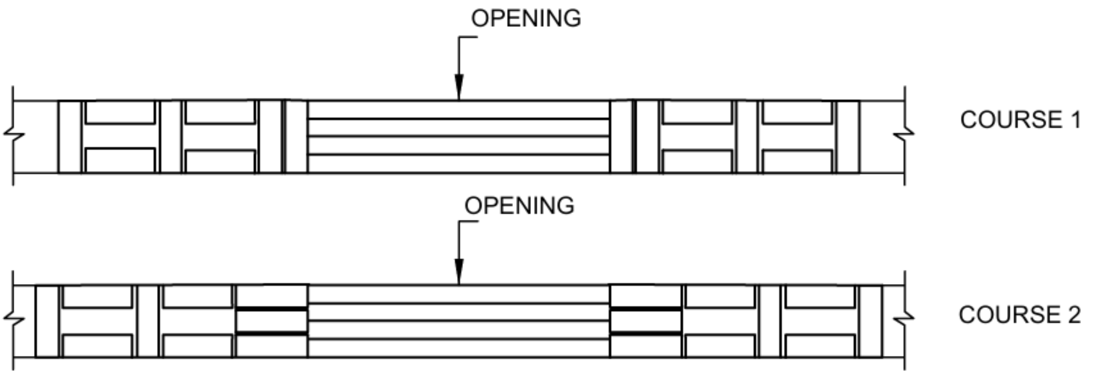
G-4.2.2 A typical module detail is shown in Fig. 86 with conventional bricks of size 230 mm x 110 mm x 75 mm. The brick adjacent to the openings has to be solid as per Fig. 87. So, the opening dimensions also to be in multiple of the rat-trap bond module. This will be important as opening dimensions are predefined depending on the module dimension. In brickwork the cross joints in any course shall not be nearer than a quarter of brick length from those in the course below or above it.





PLAN

FIG. 86 TYPICAL MODULE DETAILS OF RAT-TRAP BOND



PLAN

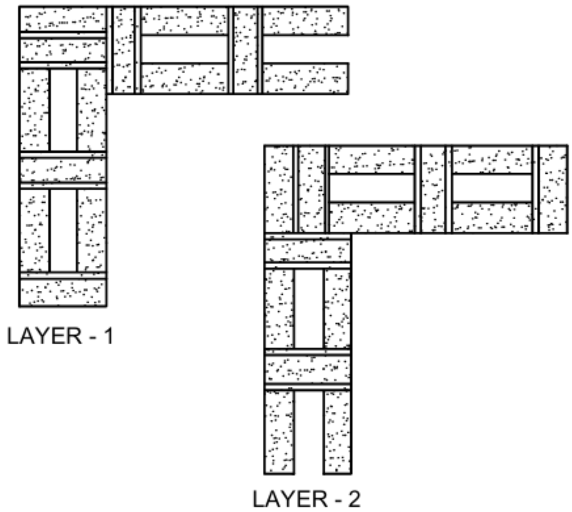
FIG. 87 OPENING IN RAT-TRAP BOND MASONRY

G-4.3 Precautions

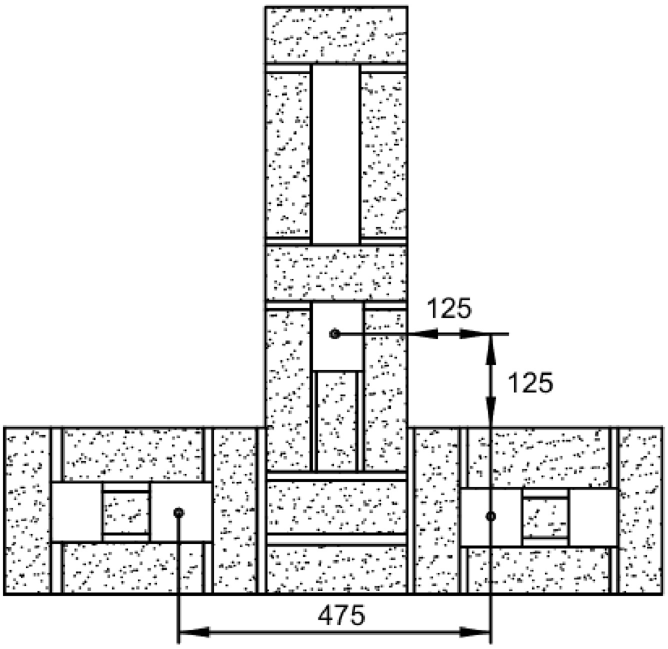
This type of masonry construction shall have to be carried out by trained masons to avoid wastage of mortar falling into the gap. Rat-trap bond shall never be carried out with brick which are below their minimum strength requirements.

G-4.4 Strength of Rat-Trap Bond

The compressive strength of rat-trap bond will be different from that of solid wall of similar thickness. Rat-trap bond wall provides continuous cavity over the entire wall height and hence it is convenient to reinforce if necessary to resist lateral loads. The compressive strength of the unreinforced rat-trap bond wall can be determined either based on the cavity wall design guidelines following **5** or the basic compressive stress can be assessed following the tests on prism specimen given at **G-7**.



L - JUNCTION DETAIL FOR ALTERNATE LAYERS



All dimensions in millimetres.

FIG. 88 TYPICAL LAYING OF RAT-TRAP BOND

G-4.5 Thickness of Joint

No bed joint shall be thicker than 12 mm. Further the thickness of the bed joints shall be such that four courses and three joints taken consecutively shall measure as follows:

a) *In the case of traditional bricks* — Four times the actual thickness of the brick plus 30 mm.
b) *In the case of modular bricks* — 390 mm conforming to the relevant accepted standards. The vertical mortar joints to be of the thickness of 10 mm and be applied directly to the brick before placing it. This requires some special skill, but defines the strength as well as the stability of the Rat-trap bond wall. Besides, applying the mortar joint directly to the brick is a basic skill requirement of a mason and determines the speed of work and finally the cost of the Rat-trap bond wall.
c) The *face* jolts of brickwork may be finished by 'jointing' or by 'pointing'. In case where brick size is less, the elevation of the header bricks to be made even in line with the stretcher brick by cement mortar same grade as bedding mortar. The main reasons for pointing the surface of block or brickwork joints are to increase its weather resistance and to give a neat finish to the work. Pointing can be carried out as construction of the brickwork proceeds, using ordinary mortar in which the bricks are bedded.

G-4.6 Structural and Functional Characteristics of Brickwork

G-4.6.1 *Structural Stability and Strength*

Reference may be made to Part 6 'Structural Design, Section 2 Soils and Foundation' of the Code, and **4** of this Section.

G-4.6.2 *Resistance to Moisture Penetration*

The suitability of various walls under different conditions of exposure is indicated in Table 3 of good practice [6-4(4)].

G-4.6.3 *Thermal Stresses*

The coefficient linear expansions of brick vary from $5 \times 10^{-6}/^{\circ}C$ to $11 \times 10^{-6}/^{\circ}C$. Variations of temperature tend to produce linear changes in walls which, when restrained, may lead to internal stress resulting in cracks especially when the walls exceed 30 m. The effects of these stresses shall be taken into consideration for a proper design.

G-4.7 Control of Shrinkage Cracking

G-4.7.1 To confine cracks to the joints and to dissipate these into a large number of fine joints, it is desirable that the mortar used shall be weaker than the bricks.

G-4.7.2 Cracking due to shrinkage normally will occur at openings or other points where the vertical or horizontal section of a wall changes. A long wall with a few openings will tend to show wider cracks above and below the openings than a similar wall with many openings. Metal reinforcement may be embedded in brickwork at points where cracking is likely to occur.

G-4.8 Thermal Insulation

For requirements of thermal insulation in walls, the provisions of good practice [6-4(24)] shall apply.

G-4.9 Fire Resistance

For requirements regarding resistance to fire, the provisions of good practice [6-4(25)] shall apply.

G-4.10 Sound Insulation

Requirements for insulation against airborne sound are laid down in good practice [6-4(26)] shall apply.

G-5 LAYING OF BRICKWORK

G-5.1 Brick shall be laid on a full bed of mortar. When laying the bricks, it shall be slightly pressed so that the mortar gets into all the pores of the brick surface to ensure proper adhesion. Cross joints and wall joints shall be properly flushed and packed with mortar so that no hollow spaces are left. Properly filled joints ensure maximum strength and resistance to penetration of moisture which takes place mainly through joints.

G-5.2 The following specific guidelines shall be followed:

a) In areas with a very high ground water table, it is advisable to build the first 4 to 5 courses of the wall, with either English or a Flemish bond. This is to avoid ground and spill water related dampness problems.
b) Always clean the surface where the first layer of bricks is to be laid, which is usually above the plinth level.
c) Arrange the joints between the bricks in such a way that only full bricks (shiners) fit. Ensure that the bonding is correct.
d) To properly maintain the brick cross, lay the first two masonry courses by marking mark each of the shiner and the header bricks at the centre.
e) Place the shiner/stretcher and header brick now in such a way that the marks are exactly matching. The remaining courses may then be laid to match with the first two courses.
f) Spread a full bed of mortar, and furrow it with a trowel. This will ensure that plenty of mortar is on the bottom of the bricks for the first course.
g) Lay the corner bricks first. Carefully position the corner bricks first. Be sure to lay all bricks with the frog inside the cavity of the wall in order to create an aesthetically pleasing outer wall surface.
h) The vertical mortar joints shall be applied directly to the brick before placing it. This requires some special skill, but defines the strength as well as the stability of the rat-trap bond wall.
j) The maximum rat-trap bond brick wall height per day should not exceed more than 9 layers.
k) Cavities of rat-trap bond shall be closed for the first course after damp proof course, below sill level, below and above lintel and at the top most course; this can be made by a layer of headers or headers.

G-5.3 Protection Against Damage

Care shall be taken during construction that edges of jambs, sills, heads, etc are not damaged. In inclement weather, newly built work shall be covered with gunny bags or tarpaulin so as to prevent the mortar from being washed away.

G-6 INSPECTION

G-6.1 General

The object of inspection of brick masonry work is to ensure its satisfactory performance and also to ascertain whether all the recommended practice of workmanship is adopted at every stage. As the correct strength of masonry cannot be ascertained without destruction, a close supervision during the course of construction is necessary to ensure satisfactory performance. The strength of brickwork depends on the strength of individual brick, strength of mortar, bond and workmanship.

G-6.2 Inspection of Materials

In case of large works, samples of bricks, sands, *Surkhi*, lime, cement, etc, which goes to form the brickwork, shall be periodically tested in a laboratory to make sure that they conform to the requirements stated in **G-3**. Simple field tests may suffice in the case of small works.

G-6.3 Inspection of Workmanship

A close supervision while the work is in progress will ensure a better quality work with the materials available for use. The following shall be observed at the time of inspection:

a) All loose materials, dirt and set lumps of mortar which may be lying over the surface over which brickwork is to be freshly started, shall be removed with a wire brush.
b) All the bricks shall be thoroughly soaked in potable water before use.
c) The surface over which the brickwork is to be started shall be made moist.
d) Plastic mortar results in thorough bedding of the brick and more complete filling of the joints which ensure greater strength. Care shall be taken to see that the required quantity of water is added to the mortar at the mixing platform itself and not over the courses.
e) All the joints shall, as far as possible, be thin and the specifications mentioned in **G-4.5** in this regard shall be strictly adhered to.
f) Care shall be taken to see that there is no through joints and the lap is not less than half the width of the brick, and that all the vertical joints are properly filled with mortar.
g) The verticality of the walls and horizontality of the courses shall be checked very often with plumb bob and spirit level respectively.
h) No portion of the work shall be left more than 1 m lower than the other. Where the masonry of one part has to be delayed the work shall be 'raked back' suitably at an angle not exceeding 45° according to bond and not toothed.
j) Where plastering is required to be done all the vertical as well as horizontal joints shall be raked to a depth of about 10 mm while the mortar is wet, and this will ensure satisfactory adhesion between the plaster and brickwork, and
k) Care shall be taken to ensure that the brick work is kept wet for first seven days using appropriate curing methods.

G-7 TESTING METHOD FOR RAT-TRAP BOND WALL SAMPLES

G-7.1 A Prism specimen as shown in Fig. 89 shall be used. The prism specimen should have similar mortar and bricks/blocks as that to be used in the rat-trap bond wall of the structure. The prism height (H) to width (T) ratio should be between 2 and 5 and based on H/T ratio the prism strength should be corrected following the correction factors given in Table 35.

The number of prism specimens to be tested and the necessary correction factors to be applied to derive table 35 of the prism shall be ascertained following the procedure given in Annex B. The modified mean prism compressive strength shall be divided by 4 to assess the basic compressive stress for the rat-trap bond wall, which can be used in calculating the compressive strength of rat-rap bond masonry wall. A similar test as above should be done in an English bond per coupon (*see* Fig. 90).

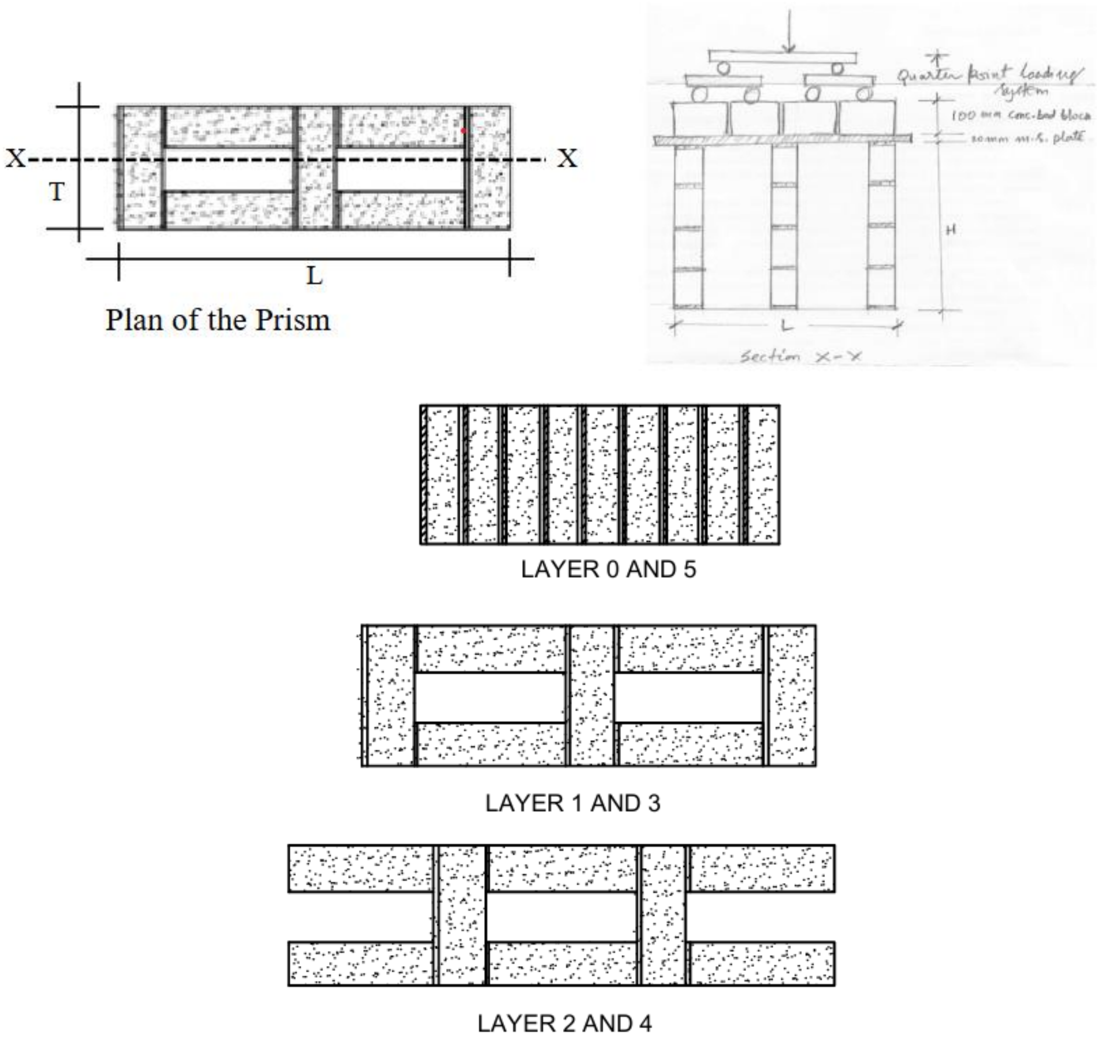
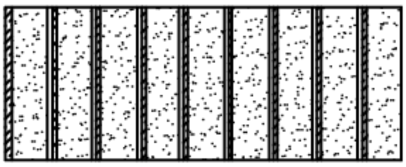
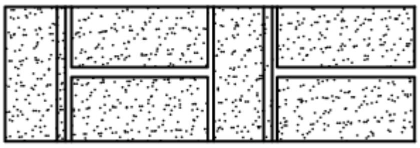


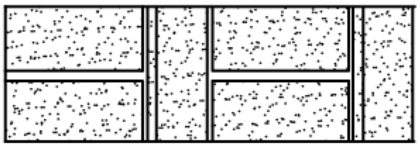
FIG. 89 PRISM SIZE AND LOADING ARRANGEMENT FOR THE RAT-TRAP BOND PRISM SPECIMEN



LAYER 0 AND 7



LAYER 1,3 AND 5



LAYER 2, 4 AND 6

FIG. 90 ENGLISH BOND TEST SPECIMEN

G-7.2 The load on rat-trap bond specimen should be not less than 85 percent of the load capacity of English bond wall compressive strength.

G-7.3 While designing the compressive stress on rat-trap bond wall, it should be ensured that the stress is not more than 0.85 of that of solid wall as per **4** of this Section.

ANNEX H
(*Clause* 14)

NOTATIONS, SYMBOLS AND ABBREVIATIONS

H-1 The following notations, letter symbols and abbreviations shall have the meaning indicated against each, unless otherwise specified in the text of this Section of the Code:

A	=	Area of a section
b	=	Width of bearing
DPC	=	Damp proof course
e	=	Resultant eccentricity
f_b	=	Basic compressive stress
f_c	=	Permissible compressive stress
f_d	=	Compressive stress due to dead loads
f_s	=	Permissible shear stress
f_m	=	Compressive strength of masonry (in prism test)
GL	=	Ground level
H	=	Actual height between lateral supports
H'	=	Height of opening
*H*1, *H*2	=	High strength mortars
h	=	Effective height between lateral supports
k_a	=	Area factor
k_p	=	Shape modification factor
k_s	=	Stress reduction factor
L	=	Actual length of wall
*L*1, *L*2	=	Lower strength mortars
*M*1, *M*2	=	Medium strength mortars
P	=	Total horizontal load
PL	=	Plinth level
RCC	=	Reinforced cement concrete
RS	=	Rolled steel
S_p	=	Spacing of piers/buttresses/cross walls
SR	=	Slenderness ratio
t	=	Actual thickness
t_p	=	Thickness of pier
t_w	=	Thickness of wall
W	=	Resultant load
W_1	=	Axial load
W_2	=	Eccentric load
w_p	=	Width of piers/buttresses/cross walls
μ	=	Ratio of flexural strength of wall in the vertical direction to that in the horizontal direction

LIST OF STANDARDS

The following list records those standards which are acceptable as `good practice' and `accepted standards' in the fulfillment of the requirements of the Code. The latest version of a standard shall be adopted at the time of enforcement of the Code. The standards listed may be used by the Authority as a guide in conformance with the requirements of the referred clauses in the Code.

In the following list, the number appearing in the first column within parentheses indicates the number of the reference in this Part/Section.

	IS No.	*Title*
(1)	1077 : 1992	Specification for common burnt clay building bricks (*fifth revision*)
	1725 : 2023	Specification for stabilized soil blocks used in general ~~building~~ construction (*third revision*)
	2180 : 1988	Specification for heavy duty burnt clay building bricks (*third revision*)
	2185	Concrete masonry units — Specification
	(Part 1) : 2005	Hollow and solid concrete blocks (*third revision*)
	(Part 2) : 1983	Hollow and solid light weight concrete blocks (*first revision*) (superseding IS 3590)
	(Part 3) : 1984	Autoclaved cellular (Aerated) concrete blocks (*first revision*) (superseding IS 5482)
	(Part 4) : 2008	Cellular preformed foam ~~cellular~~ concrete blocks
	2222 : 1991	Specification for burnt clay perforated building bricks (*fourth revision*)
	2849 : 1983	Specification for non-load bearing gypsum partition blocks (solid and hollow types) ~~(*first revision*)~~
	3115 : 1992	Specification for lime based blocks (*second revision*)
	3316 : 1974	Specification for structural granite (*first revision*)
	3620 : 1979	Specification for laterite stone block for masonry (*first revision*)

	3952 : 2013	Burnt clay hollow bricks and blocks for walls and partitions — Specification ~~(*third revision*)~~
	4139 : 1989	Specification for calcium silicate bricks (*second revision*)
	12440 : 1988	Specification for precast concrete stone masonry blocks
	12894 : 2002	Specification for pulverized fuel ash lime bricks (*first revision*)
	13757 : 1993	Specification for burnt clay fly ash building bricks
	16720 : 2018	Pulverized Fuel Ash-Cement Bricks — Specification
(2)	4082 : 1996	Recommendations on stacking and storage of construction materials and components at site (*second revision*)
(3)	2250 : 1981	Code of practice for preparation and use of masonry mortars (*first revision*)
(4)	2212 : 1991	Code of practice for brickwork (*first revision*)
	1597	Code of practice for construction of stone masonry
	(Part 1) : 1992	Part 1 Rubble stone masonry (*first revision*)
	(Part 2) : 1992	Part 2 Ashlar masonry (*first revision*)
	2572 : 2005	Code of practice for construction of hollow and solid concrete block masonry (*first revision*)
	2110 : 1980	Code of practice for in-situ construction of walls in building with soil-cement (*first revision*)
	2849 : 1983	Specification for non-load bearing gypsum partition blocks (solid and hollow types) (*first revision*)
	3630 : 1992	Code of practice for construction of non-load bearing gypsum block partitions (*first revision*)
	6041 : 1985	Code of practice for construction of autoclaved cellular concrete block masonry (*first revision*)

	6042 : 1969	Code of practice for construction of lightweight concrete block masonry
(5)	2212 : 1991	Code of practice for brickwork (*first revision*)
(6)	3414 : 1968	Code of practice for design and installation of joints in buildings
(7)	10440 : 1983	Code of practice for construction of RB and RBC floors and roofs
(8)	1893 : 1984	Criteria for earthquake resistant design of structures (*fourth revision*)
(9)	1893 (Part 1) : 2016	Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*sixth revision*)
(10)	456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
(11)	13920 : 2016	Code of practice for ductile design and detailing of reinforced concrete structures subjected to seismic forces (*first revision*)
(12)	1077 : 1992	Specification for common burnt clay building bricks (*fifth revision*)
(13)	2185 (Part 1) : 2005	Concrete masonry units — Specification: Part 1 Hollow and solid concrete blocks (*third revision*)
(14)	1597 (Part 2) : 1992	Code of practice for construction of stone masonry: Part 2 Ashlar masonry (*first revision*)
(15)	2751 : 1979	Code of practice for welding of mild steel plain and deformed bars for reinforced concrete construction (*first revision*)
(16)	9417 : 2018	Welding of high strength steel bars for reinforced concrete construction — Recommendations (*second revision*)
(17)	883 : 2016	Code of practice for design of structural timber in building (*fifth revision*)

(18)	432 Part 1 : 1982	Mild Steel and Medium Tensile Steel Bars and Hard-Drawn Steel Wire for Concrete Reinforcement: Part 1 Mild steel and Medium Tensile Steel Bars *(third revision)*
(19)	1786 : 2008	High strength deformed steel bars and wires for concrete reinforcement - Specification (fourth revision)
(20)	2502 : 1963	Code of practice for bending and fixing of bars for concrete reinforcement
(21)	1077 : 1992	Specification for common burnt clay building bricks (*fifth revision*)
	2180 : 1988	Specification for heavy duty burnt clay building bricks (*third revision*)
	1725 : 2023	Stabilized soil blocks used in general ~~building~~ construction - Specification (*third revision*)
(22)	405 (Part 2) : 1992	Lead Sheets and Strips- Specification: Part 2 for other than chemical purposes (*Third Revision*)
	1322 : 1993	~~Specification for~~ Bitumen felts for water proofing and damp-proofing ~~(*fourth revision*)~~
(23)	432	Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement:
	(Part 1) : 1982	Part 1 Mild steel and medium tensile steel bars ~~(*third revision*)~~
	(Part 2) : 1982	Part 2 Hard- drawn steel wire ~~(*third revision*)~~
	1566 : 1982	Specification for hard-drawn steel wire fabric for concrete reinforcement (*second revision*)
	412 : 1975	Specification for expanded metal steel sheets for general purposes (*Second Revision*)
	2062 : 2011	Hot rolled medium and high tensile structural steel- Specification (*Seventh Revision*)
(24)	3792 :1978	Guide for heat insulation of non-industrial buildings

(25)	1642 : 2013	Code of practice for fire safety of buildings (general): Details of construction (*second revision*)
(26)	1950 : 1962	Code of practice for sound insulation of non-industrial buildings
(27)	17848 : 2022	Confined masonry for earthquake resistance — Code of Practice