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IITK-GSDMA Project on Building Codes

Structural Use of Unreinforced Masonry

by

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- The views and opinions expressed are those of the authors and not necessarily of the GSDMA, the World Bank, IIT Kanpur, or the Bureau of Indian Standards.
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PART 1: CODE AND COMMENTARY

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0. – Foreword

0.1 –

~~This Indian Standard (Third Revision) was adopted by the Bureau of Indian Standards on 30 August 1987, after the draft finalized by the Structural Safety Sectional Committee had been approved by the Civil Engineering Division Council.~~
This draft revision of IS: 1905 is prepared as a project entitled "Review of building codes and Handbook" awarded to IIT Kanpur by GSDMA, Gandhinagar through World Bank Finances.

0.2 –

Structural adequacy of masonry walls depends upon a number of factors, among which mention may be made of quality and strength of masonry units and mortars, workmanship, methods of bonding, unsupported height of walls, eccentricity in the loading, position and size of openings in walls: location of cross walls and the combination of various external loads to which walls are subjected.

0.3 –

This code was first published in 1961. In its revision in 1969, basic compressive stresses and stress factors for slenderness were modified resulting in increased permissible stresses in load bearing brick and block walls. Subsequently two more revisions were published in 1980 & 1987. The following major changes were made in its second revision:

a) Use of stones (in regular sized units), concrete blocks, lime based blocks and hollow blocks were included as masonry units;
b) Mix proportions and compressive strengths of mortars used in masonry were revised;
c) Optimum mortar mixes for maximum strength of masonry for units of various strengths were indicated;
d) Provisions for lateral supports to walls had been amplified so as to include stability requirements;
e) Conditions of support for calculation of effective height of masonry walls and columns, and effective length of masonry walls were spelt out more clearly;
f) Maximum allowable slenderness ratio for load bearing walls was increased;
g) In case of free-standing walls, height to

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thickness ratios were indicated for different wind pressures, based upon requirements for stability;

h) Basic compressive stresses for masonry members were modified so that strength of masonry units correspond to revised values of brick crushing strength specified in IS:1077-1986*;

i) Formula for calculating area reduction factor was modified;

j) Angle of dispersion of concentrated loads, from the direction of such loads was changed from 45° to 30°;

k) Provisions relating to shape modification factors for masonry units other than common bricks were amplified;

l) Values of permissible shear stress was related to the actual compressive stresses in masonry due to dead loads;

m) Provisions on 'corbelling' were amplified.

*Specification for common burnt clay building bricks (Fourth Revision).

0.4 –

The present revision is intended to further modify certain provisions as a result of experience gained with the use of the second revision of the standard. The following major changes have been made in this revision.

(i) The requirements of a masonry element for stability have been modified.

(ii) In the design of a free standing wall, provision has been made for taking advantage of the tensile resistance in masonry under certain conditions.

(iii) Provision regarding effective height of a masonry wall between openings has been modified.

(iv) Method of working out effective height of a wall with a membrane type DPC has been modified,

(v) Criteria for working out effective length of wall having openings have been modified.

(vi) Some general guidelines have been given for dealing with concentrated loads for design of walls.

(vii) Provisions regarding cutting and chases in walls have been amplified.

(viii) The title has been changed for the sake of greater clarity.

0.5 –

The following major changes have been introduced in the present fourth revision :

(a) Permissible stresses in masonry whenever applicable have been

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C0.5 -

Unlike previous versions of this code, this new version addresses both unreinforced and reinforced masonry.

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expressed in terms of compressive strength of masonry.

(b) Permissible strength in shear has been modified to include shear strength corresponding to all likely failure modes.

(c) Some new definitions have been added and 'pier' and 'pillaster' have been re-defined.

(d) Some general guidelines for the proper selection of the mortar have been given.

0.6 – In the present revision the prevailing practices in the country were taken into consideration and assistance has been derived from the following publications:

a) ACI 530-02/ASCE 5-02/TMS 402-02 Building code requirements for Masonry structures.

b) International Building code 2000, International Code Council.

c) Eurocode 6, Design of Masonry Structures – Part 1-1: General rules for buildings – Rules for Reinforced and Unreinforced Masonry, European Committee for Standardization.

d) NZS 4230 Part 1 & 2: 1990, Code of Practice for the Design of Concrete Masonry Structures and Commentary, Standards Association of New Zealand.

e) AIJ Standards for Structural Design of Masonry Structures, 1989 edition.

f) Bangladesh National Building Code, 1993: Final Draft December 1993.

~~a)~~g) AS 1640-1974 - SAA Brickwork Code. Standards Association of Australia.

~~b)~~h) National Building Code of Canada, 1977. National Research Council of Canada.

~~c)~~i) DIN 1053/I Code on brick calculation and performance. Deutsches Institut für Normung.

~~d)~~j) CPIII: Part2: 1970 Structural recommendations for load bearing walls with amendments up to 1976. British Standards Institution.

~~e)~~k) BS 5628: Part 1: ~~1978~~ 1992 & BS 5628: Part 2: 2000Code of practice for structural use of masonry, Part 1 Unreinforced masonry, Part 2 Reinforced masonry. British Standards Institution.

~~f)~~l) CP 12 1: Part 1: 1973 Code of practice for walling, Part 1 Brick and block masonry. British Standards Institution.

~~g)~~m) Recommended practice for engineered brick masonry. Brick Institute of

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America, 1969.

n) Masonry Designer's Guide (Third Edition), The Masonry Society.

0.7 –

It is assumed in this code that design of masonry work is done by qualified engineer and that execution is carried out (according to the recommendations of this code read with other relevant codes) under the directions of an experienced supervisor.

0.8 –

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

*Rules for rounding off numerical values (Revised).

PROVISIONS

1. – Scope

1.1 –

This code gives recommendations for structural design aspect of un-reinforced and reinforced load bearing and non-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 in regard to the materials to be used, maximum permissible stresses and methods of design.

1.2 –

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

COMMENTARY

C1. – Scope

C1.1 –

BIS did not formulate any Code of practice for design and construction of reinforced masonry in the past since it considered the quality of bricks generally available in the country were not suitable for use in reinforced masonry. Despite this reinforcement has been widely used in masonry construction and strongly encouraged for the earthquake resistance by certain BIS Codes of practices. Presently available masonry materials certainly can be used for reinforced masonry, if proper care is exercised about the quality of construction and use of non-corroding reinforcement.

C1.2 –

Mud mortar for masonry as bonding material is normally not used in the present day construction because of its poor bonding quality. Mud mortar does attain some strength on drying, but it readily absorbs moisture on coming in contact with moisture or rain and loses its strength when wet. For temporary and low cost single storeyed houses, however, it is sometimes used particularly in rural areas, when economy in cost is the main consideration.

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2. – Terminology

For the purpose of this code, the definitions given in IS: 2212-1962* and the following shall apply.
*Code of practice for brickwork.

2.1 – Bed Block

A block bedded on a wall, column or pilaster 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, Pilasters and Buttress

2.3.1 – Column

An isolated vertical load bearing member, width of which does not exceed four times the thickness.

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C2. – Terminology

For the consistent use of this code, various terms are assumed to have certain meaning in this code. Many terms as defined in this commentary not always correspond to their meaning in ordinary usage.

Some of the terms defined in this clause are illustrated further to clarify their meaning. Some of the terms defined in this clause are illustrated in Fig. C-l to C-8.

C2.1 – Bed Block

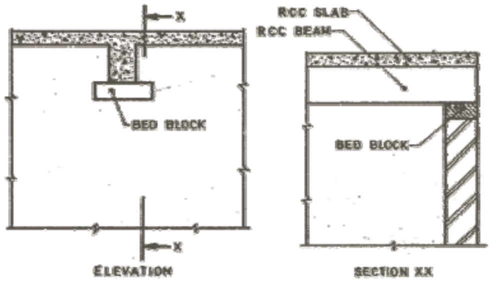

Figure C1: Bed Block

C2.2 – Bond

Typically running bond is preferred when the units of each course overlap the units in the preceding course by between 24% and 75% of the length of the units.

Stack bond is required when the units of each course do not overlap the units of the preceding course by the amount specified for running or stretcher bond.

C2.3 – Column, Pier, Pilaster and Buttress

C2.3.1 – Column

Need to distinguish column from wall arises because a column can take lesser unit load than a wall. This behavior of masonry is based on experimental research where it has been shown that the characteristic failure of wall under compressive loading is due to development of horizontal tensile stresses in the brick, causing vertical splitting of wall masonry in line with the vertical mortar joints. The cracks can develop at such intervals as to result progressively slender columns side by side. The lower elasticity of mortar causes vertical compressive load to impart lateral strain movements to the mortar, which produces tensile

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stresses in the brick by interface bond whilst maintaining the bed-joint mortar in compression. Greater the height to length ratio of the wall, higher the value of horizontal tensile stresses at the vertical joints and, therefore, weaker the wall against vertical splitting under load.

Since a column has greater height to length ratio in comparison to a wall, it has a lower permissible stress under a vertical load.

A masonry column has been defined as a vertical member the width of which does not exceed 4 times the thickness. However, a limiting value of 3 times the thickness for width of the column has also been used by some codes.

2.3.2 – Pier

It is 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.

C2.3.2 – Pier

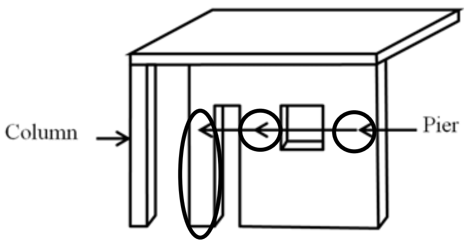

Figure C2-A: Piers and columns

2.3.3 – Pilaster

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~~ pilaster 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).

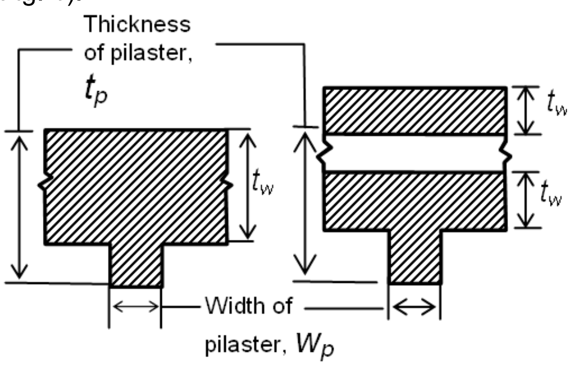

Figure 1: Definition of Pilaster

C2.3.3 – Pilaster

In earlier versions of the codes, the pilaster has been referred as pier. In this version pier, as defined above, has been used to represent the vertical portion of the masonry in a wall which is created by the openings on either side of it.

Pilasters are usually visible from one or both sides of the wall but can be hidden. They help in improving lateral load resistance of the wall and may carry vertical load.

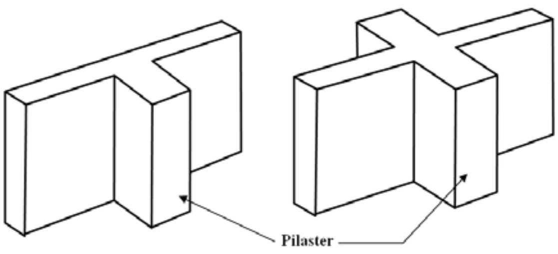

Figure C2-B: Pilasters as parts of a wall.

2.3.4 – Buttress

A ~~pier~~ pilaster 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.

C2.3.4 – Buttress

Like pilaster, buttresses are used to provide lateral support to the masonry wall in the horizontal direction.

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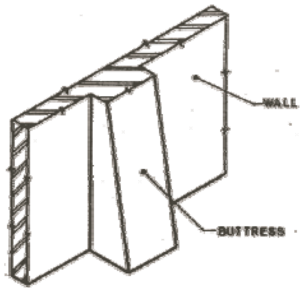


Figure C2-C: Buttress

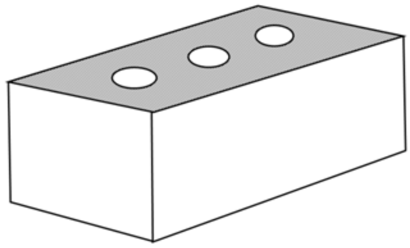
Figure C2: Column, Pier , Pilaster and Buttress

2.4 – Cross-Sectional Area of Masonry Unit

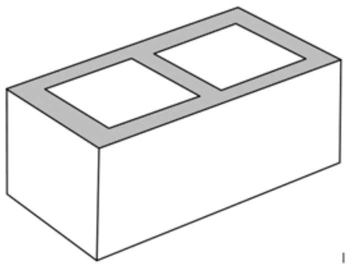
Net cross-sectional area of a masonry unit shall be taken as the gross cross-sectional area minus the area of cellular space. Gross cross-sectional area of cored units shall be determined to the outside of the coring but cross-sectional area of grooves shall not be deducted from the gross cross-sectional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

C2.4 – Cross-Sectional Area of Masonry Unit

Net section area is difficult to ascertain especially in hollow masonry units. In case of full mortar bedding as shown in Figure C3 it is the gross sectional area based on the out-to-out dimension minus hollow spaces. Often alignment of cross webs is not possible while laying hollow units and the load transfer takes place through mortars on the face shells only. In such cases, it is conservative to base net cross-sectional area on the minimum face shell thickness.



(a) Brick more than 75% solid. Net area equals gross area



(b) Hollow Unit: Full Mortar Bedding

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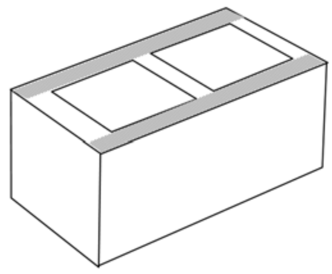

(Requires alignment of cross webs)

(c) Hollow unit: Face Shell bedding

Figure C3: Cross sectional area of masonry unit

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 Figure 2).

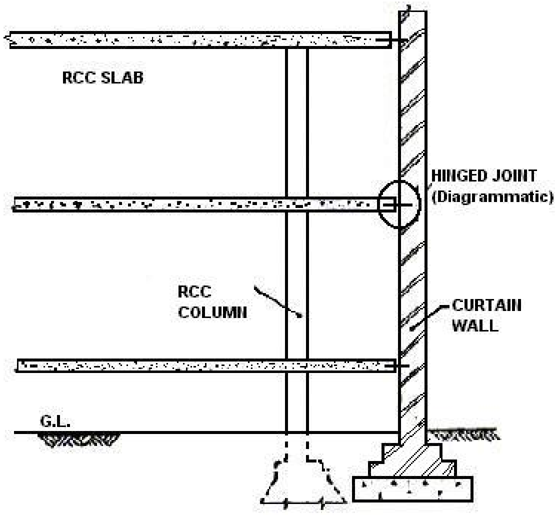

Figure 2:Masonry Curtain Wall

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

C2.9 – Hollow Unit

It has been observed that in perforated bricks, type and distribution of voids influence the strength of

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surface is less than 75 percent of its gross cross-sectional area measured in the same plane (see **2.4** and **2.18**).

2.10 – Grout

A mixture of cement, sand and water of pourable consistency for filling small voids.

2.11 – Grouted Masonry

2.11.1 – Grouted Hollow-Unit Masonry

That form of grouted masonry construction in which certain designated cells of hollow units are continuously filled with grout.

2.11.2 – Grouted Multi-Wythe Masonry

That form of grouted masonry construction in which the space between the wythes is solidly or periodically filled with grout.

2.12 – Jamb

Side of an opening in wall.

2.13 – Joint

2.13.1 – Bed Joint

A horizontal mortar joint upon which masonry units are laid.

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bricks but for perforation areas up to 35 percent of the cross-section, the bricks have been found to behave as if solid.

C2.12 - Jamb

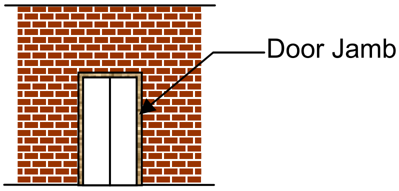

Figure C4: Door Jamb

C2.13 – Joint

Three most common joints in masonry work are bed, head and collar joints as shown in Figure C5.

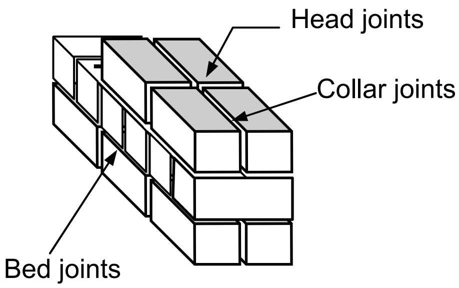

Figure C5: Joints in masonry

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2.13.2 – Cross (Head) Joint

A vertical joint, normal to the face of the wall.

2.13.3 – Wall (Collar) Joint

A vertical joint parallel to the face of the wall.

2.14 – Leaf

Inner or outer section of a cavity wall.

2.15 – 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.

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C2.13.3 – Wall (Collar) Joint

It is the vertical longitudinal joint between wythes of masonry or between masonry wythe and back-up construction which is permitted to be filled with mortar or grout.

C2.15 – Lateral Support

Lateral support is a primary requirement in structural design of masonry. A lateral support may be provided along either a horizontal or a vertical line, depending on whether the slenderness ratio is based on a vertical or horizontal dimension. Horizontal or vertical lateral supports should be capable of transmitting design lateral forces to the elements of construction that provide lateral stability to the structure as a whole.

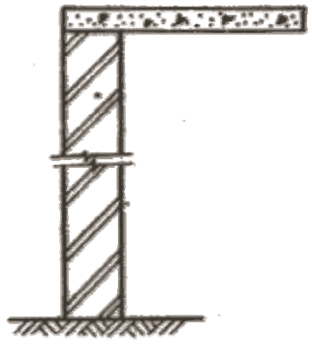

Figure C6-A RCC slab giving lateral support to a wall at top

Figure C6-B Cross walls giving lateral support to a wall

Figure C6-C Pilasters giving lateral support to a wall

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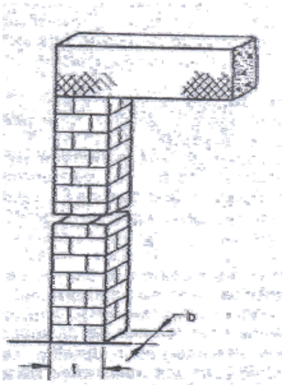
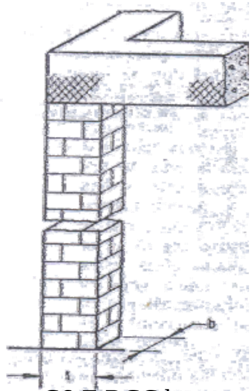


FigureC6-D RCC Beam giving lateral support to a column in the direction of its thickness '*t*'.



FigureC6-E RCC beams giving lateral support to a column in the direction of thickness '*t*' as well as width '*b*'

Figure C6: Lateral supports

Preferably, columns should be provided with lateral support in both horizontal directions.

2.16 – Load Bearing Wall

A wall designed to carry an imposed vertical load in addition to its own weight, together with any lateral load

2.17 – Masonry

An assemblage of masonry units properly bonded together with mortar.

2.18 – Masonry Unit

Individual units which are bonded together with the help of mortar to form a masonry element, such as wall, column, ~~pier~~ pilaster and buttress.

2.19 – Non-Load Bearing Wall

A wall that is not resisting or supporting any loads such that it can be removed with the approval of a structural engineer without jeopardizing integrity of the remaining structure.

2.20 – Partition Wall

An interior non-load bearing wall, one storey or part storey in height.

2.21 – Panel Wall

An exterior non-load bearing wall in framed construction wholly supported at each storey

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but subjected to lateral loads in out plane direction such as wind loads.

2.22 – Prism

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

2.23 – Shear Wall

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

2.24 – Slenderness Ratio

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

2.25 – Specified Compressive Strength of Masonry

Minimum Compressive strength, expressed as force per unit of net cross- section area, required of the masonry used in construction by the contract document, and upon the project design is based. Whenever the quantity f_m is under the radical sign, the square root of numerical value only is intended and the result has units of MPa.

2.26 – Solid Unit

A masonry unit whose net cross-sectional area in any plane parallel to the bearing surface is 75 percent or more of the gross cross-sectional area in the same plane.

2.27 – Types of Walls

2.27.1 – Cavity Wall

A wall comprising two leaves, each leaf being built of masonry units and separated by a

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C2.23 – Shear Wall

Horizontal (lateral) force acting on the wall A is resisted by cross walls B which act as shear wall.

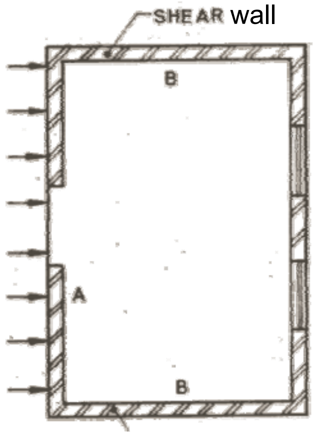

Figure C7: Shear wall

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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 waterproofing material.

2.27.2 – 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 and Fig. 4).

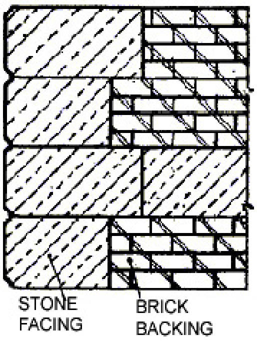

Figure 3:Typical faced wall

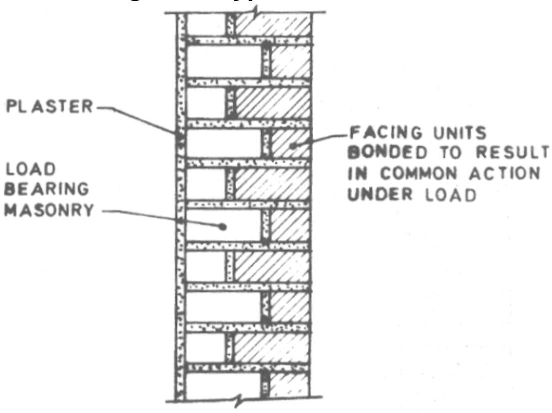

Figure 4: Faced Wall

C2.27.2 – Faced Wall

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

2.27.3 – 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.

C2.27.3 – Veneered Wall

Veneer walls have no structural effect, due to it’s own weight.

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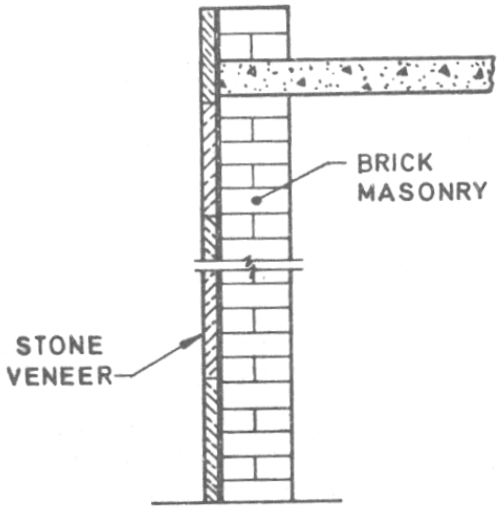

Figure C8: Veneered wall

2.28 – Wall Tie

A metal fastener which connects wythes of masonry to each other or to other materials.

2.29 – Wythe

A continuous vertical tie of masonry one unit in thickness. Plinth Band, Lintel band

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3. – Materials

3.1 – Masonry Units

Masonry units used in construction shall comply with the following standards:

Burnt Clay Building bricks	IS : 1077-1986* or IS : 2180-1985 or IS : 2222-1979
Stones (in regular sized units)	IS : 3316-1874§ or IS : 3620-1979**
Sand lime bricks	IS : 4139-1976∂
Concrete blocks (Solid & hollow)	IS:2185(Part 1)-1979¥ or IS:2185(Part 2)-1983€
Lime based blocks	IS : 31151978;±
Burnt clay hollow blocks	IS : 3952-19785‖
Gypsum partition blocks	IS : 2849-1983╫
Autoclaved cellular concrete blocks	IS:2185 (Part 3)-1984╤

NOTE 1 -Gypsum partition blocks are used only for construction of non-load bearing partition walls.
NOTE 2 - Use of other masonry units, such as precast stone blocks, not covered by the above specifications, can also be permitted based on test results.

* Specification for common burnt clay building bricks *(Fourth revision)*
 Specification for heavy-duty burnt clay building bricks*(second revision)*
 Specification for burnt clay perforated building bricks*(second revision)*
§ Specification for structural granite *(First revision).*
** Specification for late rite stone block for masonry (*First revision*).
∂ Specification for sand lime bricks *(First revision).*
¥ Specification for concrete masonry units: Part 1
Hollow and solid concrete blocks *(second revision).*
€ Specification for concrete masonry units : Part 2
Hollow and solid lightweight concrete blocks *(First revision)***.**
± Specification for lime based blocks (F*irst revision*).
‖ Specification for burnt clay hollow blocks for

COMMENTARY

C3. – Materials

C3.1 – Masonry Units

Choice of masonry units is generally made from the consideration of: (a) local availability, (b) compressive strength, (c) durability, (d) cost, and (e) ease of construction. Brick has the advantage over stone that it lends itself to easy construction and requires less labour for laying. Stone masonry, because of practical limitations of dressing to shape and size, usually has to be thicker and results in unnecessary extra cost. Thus, the first choice for a building at any place would be brick, if it is available at reasonable cost with requisite strength and good quality. In hills as well as in certain plains where soil suitable for making bricks is not available or cost of fuel for burning bricks is very high and stone is locally available, the choice would be stone. If type and quality of stone available is such that it cannot be easily dressed to shape and size, or if the cost of dressing is too high, use of concrete blocks may prove to be more economical, particularly when construction is to be more than two storeys, since thickness of walls can be kept within economical limits by using concrete blocks. In areas where bricks and stone of suitable quality are not available and concrete blocks cannot be manufactured at reasonable cost, and lime and sand of good quality are available, masonry units could be of sand-lime bricks. However, for manufacture of sand-lime bricks, special equipment is required, and thus use of sand-lime bricks is not common in India as yet.

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walls and partitions *(First revision).*

ǂ Specification for non-load bearing gypsum partition blocks (solid and hollow types) *(First revision)*

╤ Specification for concrete masonry units : Part 3 Autoclaved cellular (aerated) concrete blocks *(First revision).*

3.1.1 –

Masonry units that have been previously used shall not be reused in brickwork or block work construction, unless they have been thoroughly cleaned and conform to the code for similar new masonry units.

3.1.2 –

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.

3.2 – Mortar

Mortar for masonry shall comply with the requirements of IS: 2250-1981**.

** Code of practice for preparation and use of Masonry mortars *(first revision).*

COMMENTARY

C3.1.1 –

Bond between mortar and masonry units is largely influenced by suction rate (initial rate of water absorption) of masonry units. Masonry units, which have been previously used in masonry would not possess adequate suction rate and as a result may not develop normal bond and compressive strengths when reused. It is therefore not advisable to reuse such units in locations where requirement of masonry strength is critical.

C3.1.2 –

As a general rule, apart from strength of masonry units and grade of mortar, strength of masonry depends on surface characteristics and uniformity of size and shape of units as well as certain properties of mortar. Units which are true in shape and size, can be laid with comparatively thinner joints, thereby resulting in higher strength. For this reason, use of A grade bricks gives masonry of higher strength as compared to that with B grade bricks, even though crushing strength of bricks of the two grades may be same. For similar reasons ashlar stone masonry which uses accurately dressed and shaped stones is much stronger than ordinary coursed stone masonry.

C3.2 – Mortar

Particulars of mortars for masonry are contained in IS: 2250 - 1981.

It has been observed from experimental results that lime-based mortars give higher ratio of strength of brickwork to mortar as compared to non-lime mortars. This can be explained as follows: Normally brickwork fails under a compressive load on account of vertical tensile splitting, for which bond strength of mortar is more important than its compressive strength. Since lime-based mortars have much higher bond strength, as compared to cement the former produce brickwork of higher strength. Table C-1 giving test results abstracted from SIBMAC proceedings illustrates this point very clearly.

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Table C1: Effect of Mortar Mix on Strength of Brickwork [using clay brick of strength 32.7 MPa]			
Mortar mix (Cement: Lime: Sand)	Mortar Compressive Strength (28 days) X	Brickwork compressive strength (28 days) Y	Ratio Y/X
(1)	(2)	(3)	(4)
	MPa	MPa	
1:¼:3	17.8	8.9	0.5
1:½:4½	10.8	9.3	0.86
1:1:6	4.7	8.5	1.82
1:2:9	1.7	4.6	2.69

NOTE: Lime used was in the form of well matured putty

3.2.1 –

Mix proportions and compressive strengths of some of the commonly used mortars are given in Table 1**.**

Table 1: Mix Proportion and Strength of Mortars for Masonry *(Clause 3.2.1)*							
S L N o	Grade of Mortar	Mix Proportions (By Loose Volume)					Minimum Compressive Strength at 28 Days In MPa
		Cem-ent	Lime	Lime Pozzo-lana Mixture	Pozzo-lana	Sand	
1	H1	1	¼ C or B	0	0	3	10
2(a)	H2	1	¼ C or B	0	0	4	7.5
2(b)	H2	1	½ C or B	0	0	4½	6.0
3(a)	M1	1		0	0	5	5.0
3(b)	M1	1	1 C or B	0	0	6	3.0
3(c)	M1	0	0	1(LP-40)	0	1½	3.0
4(a)	M2	1	0	0	0	6	3.0
4(b)	M2	1	2B	0	0	9	2.0
4(c)	M2	0	1A	0	0	2	2.0
4(d)	M2	0	1B	0	1	1	2.0
4(e)	M2	0	1 C or B	0	2	0	2.0
4(f)	M2	0	0	1(LP-40)	0	1¾	2.0
5(a)	M3	1	0	0	0	7	1.5
5(b)	M3	1	3B	0	0	12	1.5
5(c)	M3	0	1A	0	0	3	1.5
5(d)	M3	0	1B	0	2	1	1.5
5(e)	M3	0	1 C or B	0	3	0	1.5
5(f)	M3	0	0	1(LP-40)	0	2	1.5
6(a)	L1	1	0	0	0	8	0.7
6(b)	L1	0	1B	0	1	2	0.7
6(c)	L1	0	1 C or B	0	2	1	0.7
6(d)	L1	0	0	1(LP-40)	0	1½	0.7
6(e)	L1	0	0	1(LP-20)	0	2½	0.7
7(a)	L2	0	1B	0	0	3	0.5
7(b)	L2	0	1C or B	0	1	2	0.5
7(c)	L2	0	0	1(LP-7)	0	1½	0.5

C3.2.1 –

Mortars are intimate mixtures of some cementing materials, such as cement, lime and fine aggregate (such as sand, burnt clay/surkhi, cinder, etc). When only fat lime is used, which sets very slowly through the process of carbonation, it becomes necessary, for the sake of better strength, to use some pozzolanic material, such as burnt clay/surkhi or cinder. Plasticizers are used in plain cement-sand mortars to improve workability. Mortars could be broadly classified as cement mortars, lime mortars and cement-lime mortars. Main characteristics and properties of these three categories of mortars are as under:

a) *Cement mortars:* These consist of cement and sand, varying in proportion from 1:8 to 1:3, strength and workability improving with the increase in the proportion of cement. Mortars richer than 1:3 are not used in masonry because these cause high shrinkage and do not increase in strength of masonry. Mortars leaner than 1:5 tend to become harsh and unworkable and are prone to segregation. Cement mortars set early and gain strength quickly. Setting action of mortar is on account of chemical changes in cement in combination with water, and thus these mortars can set and harden in wet locations. In case of lean mortars, voids in sand are not fully filled, and therefore, these are not impervious. Rich mortars though having good strength have high shrinkage and are thus more liable to cracking.

b) *Lime mortars:* These consist of intimate mixtures of lime as binder and sand, burnt clay/surkhi, cinder as fine aggregate in the proportion 1:2 to 1:3. As a general rule, lime mortars gain strength slowly and have low ultimate strength. Mortars using hydraulic lime attain somewhat better strength than those using fat lime. In fact, lime mortars using fat lime do not harden at

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all in wet locations. Properties of mortar using semi-hydraulic lime are intermediate between those of hydraulic and fat lime mortars. When using fat lime, it is necessary to use some pozzolanic material such as burnt clay/surkhi or cinder to improve strength of the mortar. The main advantage of lime mortar lies in its good workability, good water retentivity and low shrinkage. Masonry in lime mortar has, thus, better resistance against rain penetration and is less liable to cracking, though strength is much less than that of masonry in cement mortar .

c) *Cement/lime mortars:* These mortars have the good qualities of cement as well as lime mortars, that is, medium strength along with good workability, good water retentivity, freedom from cracks and good resistance against rain penetration. Commonly adopted proportions of the mortar (cement: lime: sand) are 1:1:6, 1:2:9 and 1:3:12. When mix proportion of binder (cement and lime) to sand is kept as 1:3, it gives a very dense mortar since voids of sand are fully filled.

NOTE 1 - Sand for making mortar should be well graded. In case sand is not well graded, its proportion shall be reduced in order to achieve the minimum specified strength.
NOTE 2 - For mixes in Sl No. 1 and 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 it improves workability.
NOTE 3- For mixes in Sl No. 3(a), 4(a), 5(a) and 6(a), either lime C or B to the extent of l/4 part of cement (by volume) or some plasticizer should be added for improving workability.
NOTE 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.
NOTE 5 - It is essential that mixes in Sl No. 4(c), 4(d), 4(e), 5(d), 5(e), 6(b), 6(c), 7(a) and 7(b) are prepared by grinding in a mortar mill.
NOTE 6 - Mix in Sl No. 2(b) has been classified to be of same grade as that of Sl No. 2(a), mixes in Sl No. 3(b) and 3(c) same as that in Sl No. 3(a) and mixes in Sl No. 4(b) to 4(f) same as that in Sl No. 4(a), even though their compressive strength is less. This is from consideration of strength of masonry using different mix proportions.
NOTE 7 - A, B and C denote eminently hydraulic lime, semi-hydraulic lime and fat lime respectively as specified in relevant Indian Standards.

3.2.2 – Selection of Mortar

C3.2.2 – Selection of Mortar

Mortar for masonry should be selected with care keeping the following in view. It should be noted

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that cement-lime mortars are much better than cement mortars for masonry work in most of the structures.

3.2.2.1 –

Requirements of a good masonry for masonry structures are workability, flow, water retentivity in the plastic state and bond, extensibility, compressive strength, and durability in the hardened state. Compressive strength of mortar, in general, should not be greater than masonry unit.

C3.2.2.1 –

Requirements of a good mortar for masonry are strength, workability, water retentivity and low drying shrinkage. A strong mortar will have adequate crushing strength as well as adequate tensile and shear strength. It is necessary that mortar should attain initial set early enough to enable work to proceed at. a reasonable pace. At the same time it should gain strength within reasonable period so that masonry is in a position to take load early. A workable mortar will hang from the trowel and will spread easily. A mortar with good water retentivity will not readily lose water and stiffen on coming in contact with masonry units, and will remain plastic long enough to be easily adjusted in line and level. This property of good water retentivity will enable the mortar to develop good bond with masonry units and fill the voids, so that masonry has adequate resistance against rain-penetration.

3.2.2.2 –

For commonly-used mortars conforming to Table 1, the optimum mortar mixes from the unit strength consideration only are given in Table 2:

Table 2: Unit Strength of Mortar
(Clause 3.2.3.2)

Mortar type	Masonry unit strength (MPa)
M2	Below 5
M1	5-14.9
H2	15-24.9
H1	>25

C3.2.2.2 –

Optimum mortar mixes from consideration of maximum strength of brickwork for various brick strengths based on Madras Detailed Standard Specification – 1956 (Reprint 1964), (Second Series).

3.2.2.3 –

Compressive strength shall not be sole-criterion for the selection of mortar. Bond strength, in general, is more important, as is good workability and water retentivity, which are required for maximum bond. Lime-based mortars of Table 1 should be preferred for it is desirable to sacrifice some compressive strength of the mortar in favour of improved bond. A set of preferred mortar mixes are given in Table 3.

C3.2.2.3 –

An unnecessarily strong mortar concentrates the effect of any differential movement of masonry in fewer and wider cracks while a weak mortar (mortar having more of lime and less of cement) will accommodate movements, and cracking will be distributed as thin hair cracks which are less noticeable. Also stresses due to expansion of masonry units are reduced, if a weak mortar is used. Lean mortars of cement alone are harsh, pervious and less workable. Thus when strong mortars are not required from considerations of strength or for working under frosty conditions or for work in wet locations, it is preferable to use composite mortars of cement, lime and sand, in appropriate proportions. Figure C9 based on

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Table 3: Mortar Mix Composition (Clause 3.2.3.3)			
Mortar type	Preferred mix		
	Cement	Lime	Sand
H1	1	¼	3
H2	1	½	4½
M1	1	1	6
M2	1	2	9

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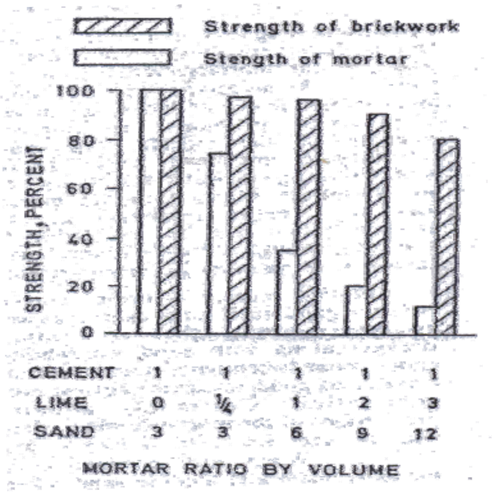
Madras Detailed Standard Specification – 1956 (Reprint 1964) illustrates the relation between strength of mortar and brickwork for a number of mortar mixes when bricks of medium strength (20 to 35 MPa according to British Standards) are used. As the proportion of lime in mortar is increased, though mortar loses strength, reduction in strength of brickwork is not much.



Effects of mortar mix proportions on the crushing strengths of mortar and brickwork built with medium strength bricks
Strengths are shown relative to the strength of a 1:3 cement-sand mortar and the brickwork built with it

Figure C9: Relation between strength of brickwork and strength of mortar

3.3 – Material Properties

3.3.1 – General

Unless otherwise determined by test, the following modulus shall be used in determining the effects of elasticity.

3.3.2 – Elastic modulus

For steel reinforcement,

$E_s = 200$ GPa $= 2.0 X 10^5$ MPa

For clay masonry and concrete masonry,

$E_m = 550 f_m$

or the chord modulus of elasticity taken between 0.05 and 0.33 of the maximum compressive strength of each prism determined by test in accordance with Appendix B.

C3.3 – Material Properties

C3.3.1 – General

Material properties can be determined by appropriate tests of the materials to be used.

C3.3.2 - Elastic modulus

Traditionally large elastic modulus has been prescribed by many masonry codes, however, research indicates that lower values are more typical. Further, large variation has been reported in the relationship between elastic modulus and compressive strength of masonry, f_m. A limited tests conducted at IIT Kanpur recently further confirm this observation and a lower value (about $550 f_m$) for elastic modulus agrees with data reasonably well. Other codes prescribe a higher value because the actual compressive strength is usually higher than the f_m especially for clay brick masonry. ACI 530 specifies that for working stress design procedure, the elastic modulus as the slope of stress strain curve below allowable flexural compressive stress ($0.33 f_m$) is most appropriate.

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3.3.3 – Shear modulus

For clay and concrete masonry, the shear modulus is 0.4 times the elastic modulus.

COMMENTARY

Data at very low stress (below *0.05* f_m) usually include the deformations of seating if measurements are made on the testing machine loading platens. As shown in Figure C10 the elastic modulus of the masonry is taken as chord modulus of stress-strain curve obtained during a prism test between stress levels of 0.05 and 0.33 times f_m.

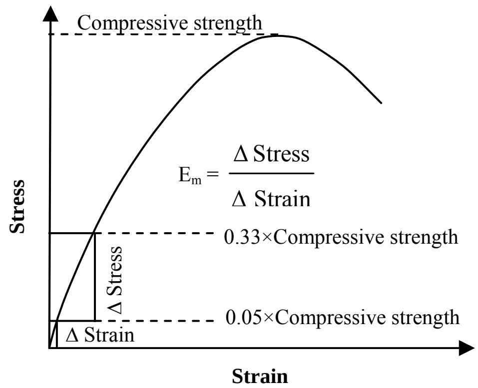

Figure C10: Chord modulus of elasticity

C3.3.3 – Shear modulus

The relationship between the modulus of rigidity and the modulus of elasticity has been given as $0.4E_m$ without any experimental evidence to support it.

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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 to:

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

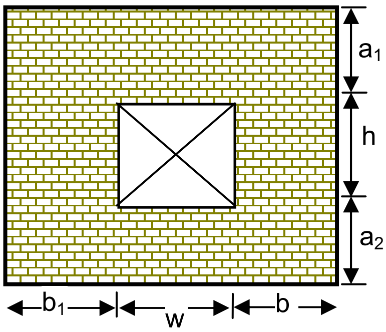
COMMENTARY

C4. – DESIGN CONSIDERATIONS

C4.1 – General

In order to ensure uniformity of loading, openings in walls should not be too large. and these should be of 'hole in wall' type as far as possible; Bearings for lintels and bed blocks under beams should be liberal in sizes; heavy concentration of loads should be avoided by judicious planning and sections of load bearing members should be varied where feasible with the loadings so as to obtain more or less uniform stress in adjoining parts of members. One of the commonly occurring causes of cracks in masonry is wide variation in stress in masonry in adjoining parts.

NOTE- A 'hole in wall' type opening is defined as an opening where total width or height of solid masonry around the opening is equal to or greater than the corresponding window dimension.



$b_1 + b_2 \geq w$

$a_1 + a_2 \geq h$

Figure C11: Hole in wall

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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 9 cm;

   2) RCC floor/roof slab not bearing on the supported wall or cross wall is anchored to it with non-corrodible metal ties of 60 cm length and of section not less than 6 x 30 mm, and at intervals not exceeding 2 m as shown in Fig. 5; and

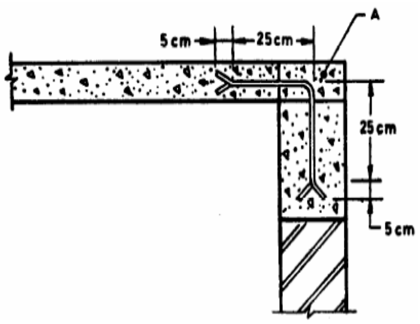

Figure 5: Anchoring of RCC slab with Masonry wall (When slab does not bear on wall)

   3) Timber floor/roof anchored by non-corrodible metal ties of length 60 cm and of minimum section 6 x 30 mm, securely fastened to joists and built into walls as shown in Figure ~~5~~6 and ~~6~~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;

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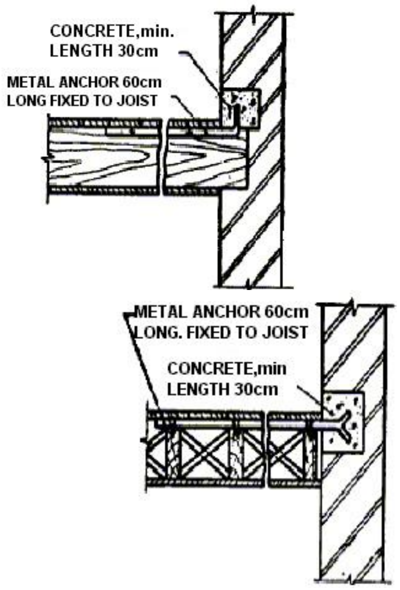


Figure 6: Typical Details for Anchorage of Solid Walls

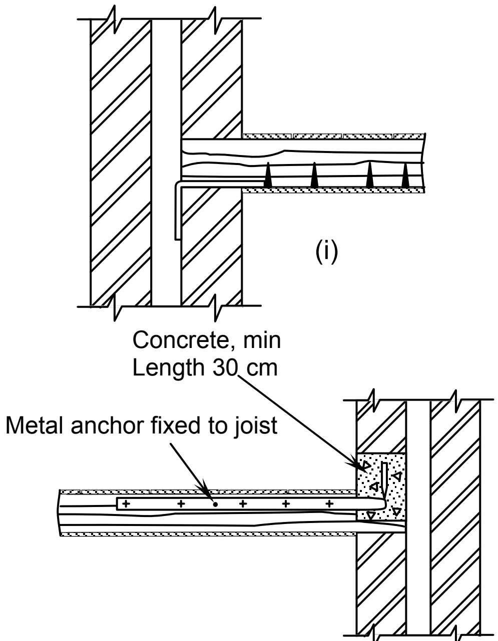
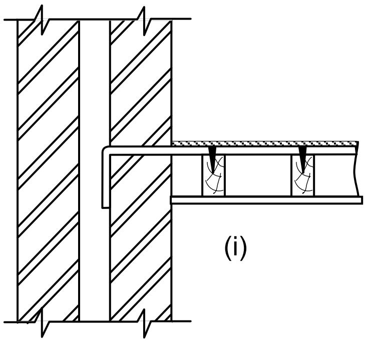


Figure 7A: Timber joist right angle to wall



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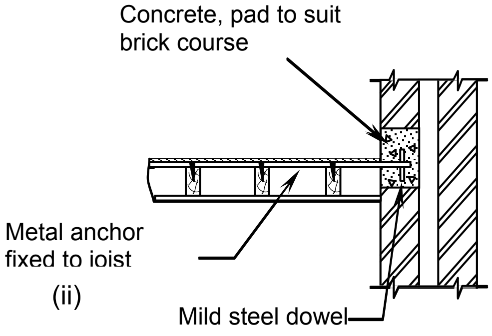

Figure 7B: Timber joist parallel to wall

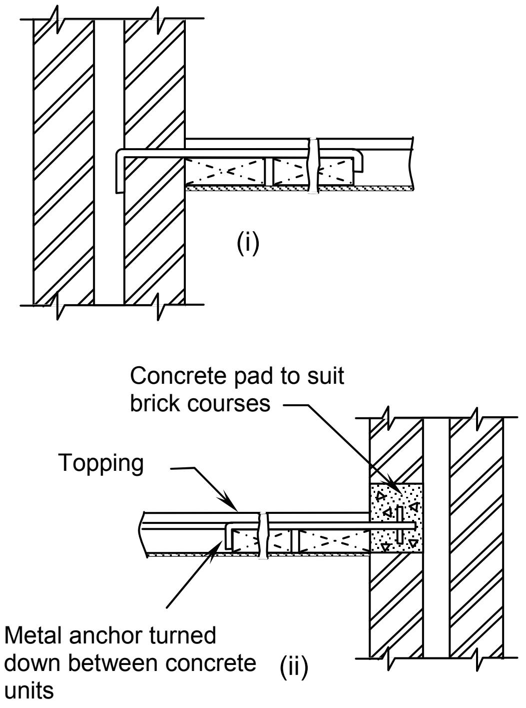

Figure 7C: Precast concrete floor units parallel to wall

Figure 7: Typical details for anchorage of cavity walls

NOTE 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.

NOTE 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.

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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 Figure ~~7~~8) and bonded to it according to provision of 4.2.2.2 (d);

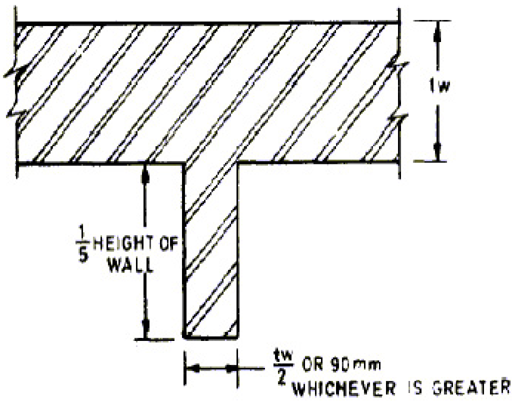

Figure 8: Minimum dimensions for masonry wall or buttress providing effective lateral support

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

c) 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.

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 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.

COMMENTARY

C4.2.2 – Stability

In a masonry structure, there are out of balance vertical forces due to imperfection in workmanship and vertical alignment of walls which tend to make the structure unstable. Thus for stability calculations of a lateral support, horizontal force equal to 2.5 percent of all vertical loads acting above that lateral support is assumed for checking the adequacy of that support. This horizontal force is in addition to any other lateral force, namely wind or seismic that the structure may be subjected to.

It should be noted that assumed horizontal force of 2.5 percent is the total out of balance force due to vertical loads at the particular support and it does not include out of balance forces acting at other

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supports. Further it should be kept in view that horizontal force of 2.5 percent of vertical loads need not be considered for elements of construction that provide lateral stability to the structure as a whole.

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 unreinforced 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 spacing as given in Table ~~2~~ 4 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 Figure 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;

d) And 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 x 35 mm and length 60 cm with ends bent up at least 5 cm; maximum vertical spacing of ties being 1.2 m (see Figure ~~9~~10).

C4.2.2.2 –

Provision in sub-clause (a) of height to width ratio of building for stability has been a traditional requirement.

A cross wall acting as a stiffening wall provides stability to the wall at its junction with the cross wall thereby resisting movement of wall at horizontal intervals and sharing a part of the lateral load. Further in conjunction with the diaphragm supported on the wall, it resists horizontal movement of the top of the wall. For the first mode of stiffening, it is necessary that cross wall is built jointly with the load bearing wall or is adequately anchored to it and there should be opening in the cross wall close to its junction with the main wall (refer clause 4.2.2.2(b) of the code); for the second mode, the diaphragm should be capable of acting as a horizontal girder and also the diaphragm should be so connected to the cross walls that lateral forces are transmitted to function the cross walls through shear resistance between diaphragm and cross walls.

When bricks of old size that is, 23 X 11.5 X 7.7 cm are used, Table C-3 may be used in place of Table 4 of the Code for buildings up to 3 storeys.

Table C3: Thickness and Spacing of stiffening walls (Brick Size 23 X 11.5 X 7.7 cm)

Sl. no.	Thickness of load bearing wall to be stiffened (cm)	Height of storey (m)	Stiffening wall	
			Minimum thickness (cm)	Maximum spacing (cm)
(1)	(2)	(3)	(4)	(5)
1	11.5	3.25	11.5	4.50
2	23	3.25	11.5	6.00
3	34.5 and above	5.00	11.5	8.00

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Table 4: Thickness and spacing of stiffening walls [Clause 4.2.2.2 (b)]

No.	Thickness of load bearing wall to be stiffened	Height* of storey not to exceed	Stiffening Wall*		
			Thickness not less than		Maximum spacing
			1 to 3 storey	4 storey	
(1)	(2)	(3)	(4)	(5)	(6)
	cm	m	cm	cm	cm
1	10	3.2	10	-	4.5
2	20	3.2	10	20	6.0
3	30	3.4	10	20	8.0
4	Above 30	5.0	10	20	8.0

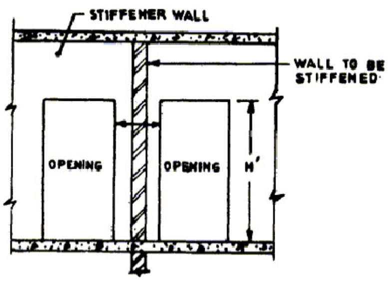

Figure 9: Opening in stiffening walls

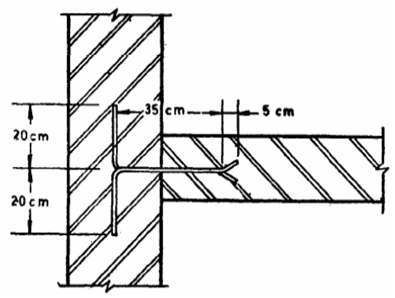

Figure 10: Anchoring of stiffening wall with support wall

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.

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C4.2.2.3 –

Cross walls in conjunction with floors and roof diaphragms in a building provide stability to the structure against the effect of lateral loads. In case of large rooms, halls, etc, we have only end walls and there are no intermediate cross walls. If hall is longer than 8.0 m, the end walls may not be able to provide adequate stability (depending upon the extent of lateral loads) and therefore, it is necessary to check stability and stresses by structural analysis.

Rigid diaphragms function as a horizontal girder for transmitting the lateral loads to the end walls (shear walls). The long walls (in out-of-plane direction) will therefore function as propped cantilevers, and should be designed accordingly

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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 and shear wall to take horizontal loads and consequent bending moments, increases when parts of bearing walls act as flanges to the cross wall. Maximum overhanging length of bearing wall

COMMENTARY

and if found necessary, stiffeners to be provided. Also end walls will be subjected to shear and bending and should be designed for permissible shear and no-tension in case of unreinforced masonry. It is necessary that diaphragms must bear on the end walls so that lateral load is transmitted to these walls through shear resistance.

C4.2.2.4 –

When a hall or a large industrial building is provided with trussed roofing the longitudinal walls cannot be deemed to be laterally supported at the top unless trusses are braced at the tie beam level as shown in Figure C12. With braced trusses as lateral supports, longitudinal walls will act as propped cantilevers and should be designed accordingly.

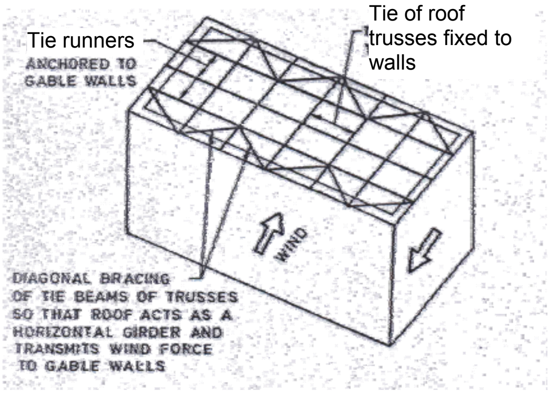

Figure C12: Diagonal bracing at bottom tie of trusses

When bricks of size 23 X 11.5 X 7.7 cm are used Table C-4 may be used in place of Table 5 of the Code.

Table C4: Minimum thickness of basement walls
(Brick Size 23 X 11.5 X 7.7 cm)

Sl. no.	Minimum thickness of basement wall (cm)	Height of the ground (m) above basement floor with wall loading (permanent load) of	
		More than 50 kN/m	Less than 50 kN/m
(1)	(2)	(3)	(4)
1	34.5	2.50	2.00
2	23	1.35	1.00

NOTE: Permanent load means only dead or fixed load and it does not include live load.

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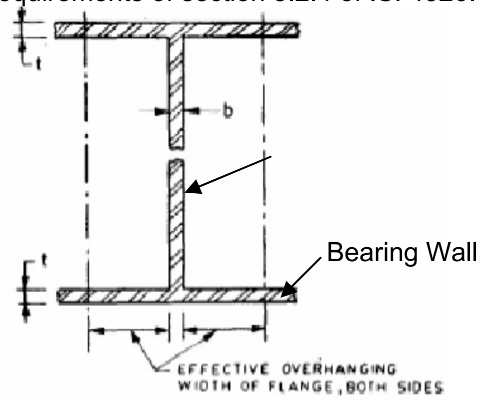
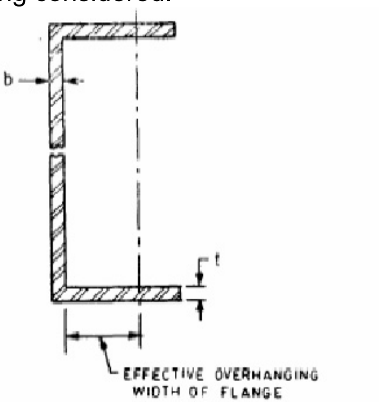
which could effectively function as a flange should be taken as *12 t* or *H/6*, whichever is less, in case of *T or I* shaped walls and *6 t* or *H/6*, whichever is less, in case of *L or U* shaped walls, where *t* is the thickness of bearing wall and *H* is the total height of wall above the level being considered as shown in Fig. 11.

The connection of intersecting walls shall conform to one of the following requirements:
a) Providing proper masonry bonds such that 50% of masonry units at the interface shall interlock.
b) Connector or reinforcement extending in each of the intersecting wall shall have strength equal to that of the bonded wall
c) Requirements of section 8.2.4 of IS: 4326.



Effective overhanging width of flange = 12 *t* or *H*/6 whichever is less, *H* being the total height of wall above the level being considered.



Effective overhanging width of flange = 6 *t* or *H*/6 whichever is less, *H* being the total height of wall above the level being considered.

Figure 11: Effective overhang width of flange

4.2.2.6 –

In case of external walls of basement and plinth stability requirements of 4.2.2 may be

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deemed to have been met with if:

a) bricks used in basement and plinth have a minimum crushing strength of 5 MPa and mortar used in masonry is of Grade Ml 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 5.

Table 5: Minimum thickness of basement walls

SL No.	Minimum thickness of basement walls	Height of the ground above basement floor level with wall loading (permanent load)	
		More than 50 kN/m	Less than 50 kN/m
(1)	(2)	(3)	(4)
	cm	m	m
1	40	2.50	2.00
2	30	1.75	1.40

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

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 overturning 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 5.10.2.1. If mortar used for masonry can not be relied upon for taking flexural tension (see 5.7.1), stability of free-standing wall shall be ensured such that stabilizing moment of wall due to self weight equals or exceeds 1.5 times the overturning moment.

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C4.2.2.7 – Walls Mainly Subjected to Lateral Loads

A free standing wall has no cross walls to give it stability against overturning due to lateral loads that is, wind or seismic loads. It thus acts like a cantilever fixed at the base and free at the top.

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b) **Retaining wall -** Stability for retaining walls shall normally be achieved through gravity action to ensure that the entire cross-section is in compression but flexural moment of resistance could also be taken advantage of under special circumstances at the discretion of the designer (see 5.8)

If a wall is intended to retain some dry material and there is no likelihood of any hydrostatic pressure, the design of wall could be based on permissible tension in masonry. A retaining wall intended to support earth should be designed as a gravity structure, placing no reliance on flexural moment of resistance, since water can get access to the back of the wall and impose pressure through tensile cracks if any and endanger the structure.

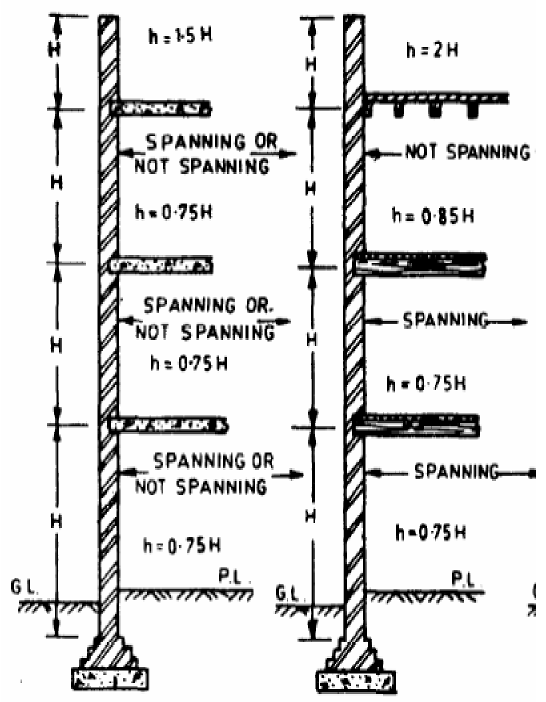
4.2.3 Structural Continuity

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.

4.3 – Effective Height

4.3.1 – Wall

Effective height of a wall shall be taken as shown in Table 6 (see Fig. 12).



C4.3 – Effective Height

C4.3.1 –Wall

Wall (Table 6-Note 1)
Referring to Note 1 of Table 4, strictly speaking actual height of a wall for the purpose of working out its effective height should be taken to be the clear distance between the supports. However, in the Code it has been given as the height between centres of supports, which is in accordance with the provisions of British Standard CP III: Part 2: 1970 as well as Australian Standard 1640-1974. Since thickness of floors is generally very small as compared to height of floors, this method of reckoning actual height will not make any appreciable difference in the end results. One could, therefore, take actual height as given in the Code or clear distance between supports as may be found convenient to use in calculations.

Wall (Table 6-Note 5)
Implication of this note is that when wall thickness is not less than 2/3 of the thickness of the pilaster, a concentrated load on the pilaster, will be borne by the pilaster as well as the wall. In this case we may design the element just as a wall supporting a concentrated load, taking advantage of the increase in the supporting area due to the pilaster projection. In case thickness of wall is less than 2/3 of the thickness of pilaster, we have to design the pilaster just like a column, for which permissible stress is less because of greater effective height and further supporting area will be only that of the pilaster that is, without getting any benefit in design of the adjoining walls on either side. However in case, the

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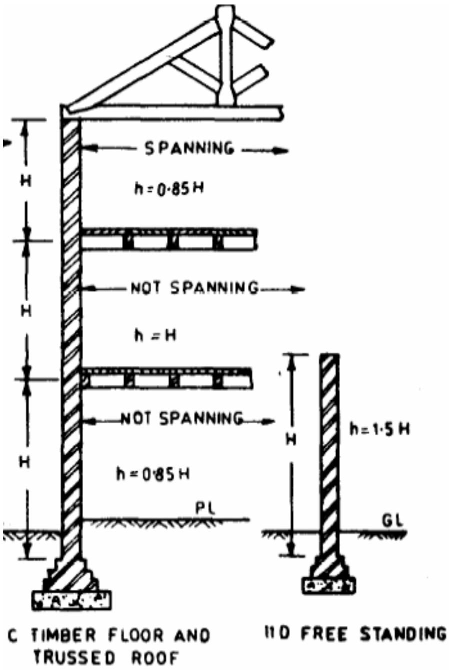

Figure 12: Effective height of the wall

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.

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 Figure 13).

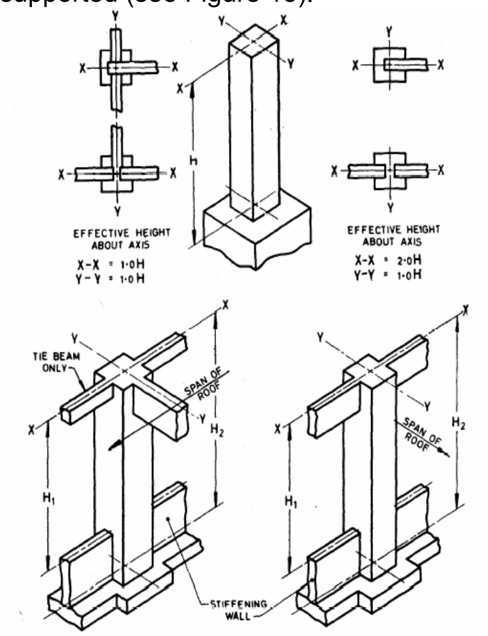

Figure13: Examples of Effective Height of the Columns

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wall and pilasters are supporting a distributed load, we would get the advantage of stiffening effect of pilasters as in 4.5.2 of the Code.

C4.3.2 – Column

In case of columns actual height should be taken as the clear height of a column between supports as illustrated in Figure C13.

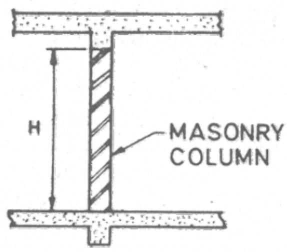

Figure C13: Actual height of a column

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NOTE 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.

NOTE 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

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 the plane of the 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.

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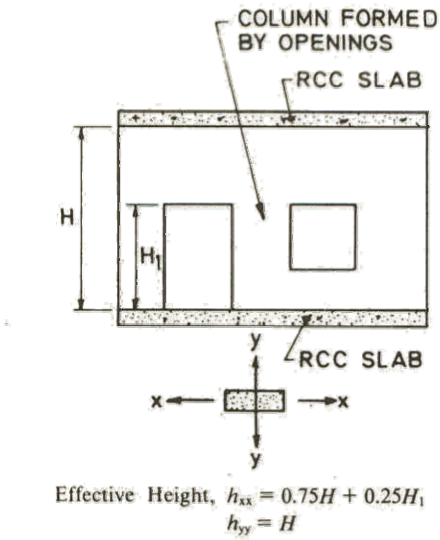
C4.3.3 – Openings in Walls

An RCC slab bearing on a wall is assumed to provide full restraint to the wall while a timber floor comprising timber joints and planking is assumed to provide only partial restraint. The clause makes stipulations for reckoning effective height of columns formed by openings in a wall for the two cases:
a) when wall has full restraint at top and bottom; and
b) when wall has partial restraint at top and bottom.
These two cases are illustrated in Figure C14:



(a) Walls having full restraint

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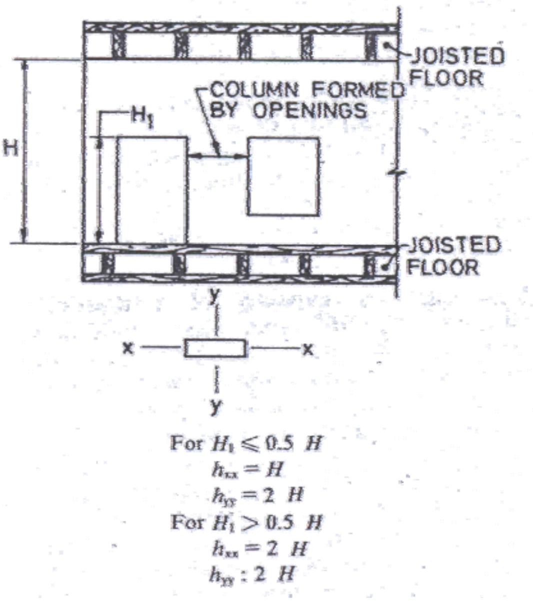

(b) Walls having partial restraint

Figure C14: Effective height of walls with openings

b) When wall has partial restraint at the top:

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

ii) In the case of (b) (*see* Figure C14), if height of neither opening exceeds *0.5H,* wall masonry would provide some support to the column formed by openings in the direction parallel to the wall and for this reason effective height for the axis perpendicular to the wall is taken as *H* and otherwise it is to be taken as *2H.* For the direction perpendicular to the wall, there is a likelihood of a situation when no joist rests on the column formed between the openings and thus effective height is taken as *2H* that is, for a column having no lateral support at the top.

Table 6: Effective Height of Walls
(Clause 4.3.1)

SL No.	Condition Of Support	Effective Height
(1)	(2)	(3)
1	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 9 cm), irrespective of the direction of the span (foundation footings of a wall give lateral as well as	*0.75 H*

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	rotational restraint).	
2	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*
3	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*
4	Lateral restraint as well as rotational restraint -at bottom but have no restraint at the top. For example, parapet walls, on RCC roof with slab having adequate bearing on the lower wall, or a compound wall with proper foundation on the soil.	*1.50 H*

NOTE 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 Figure 12.

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

NOTE 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 *0.75H*
b) Consider *H* from top of DPC and take effective height as *0.85H*.

NOTE 4 - When assessing effective height of

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Strictly speaking actual height of a wall for the purpose of working out its effective height should be taken to be the clear distance between the supports. However, in the Code it has been given as the height between centres of supports, which is in accordance with the provisions of other masonry codes. Since thickness of floors is generally very small as compared to height of floors, this method of reckoning actual height will not make any appreciable difference in the end results. One could, therefore, take actual height as given in the Code or clear distance between supports as may be found convenient to use in calculations.

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walls, floors not adequately anchored to walls shall not be considered as providing lateral support to such walls.

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

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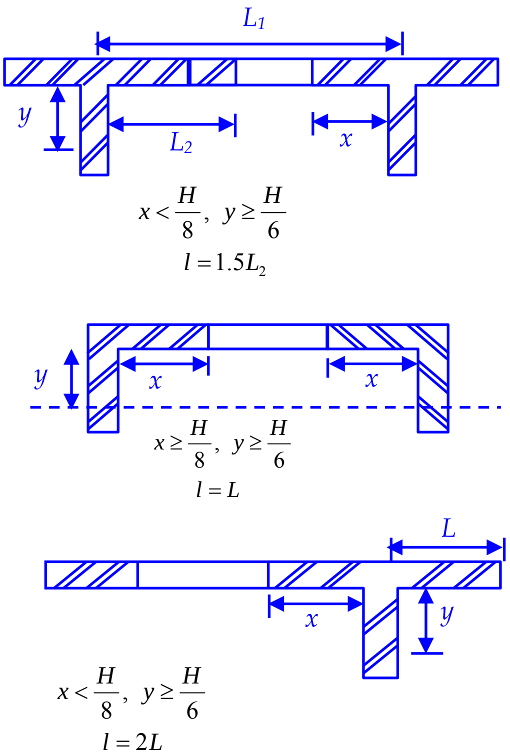
Implication of this note is that when wall thickness is not less than 2/3 of the thickness of the pilaster, a concentrated load on the pilaster, will be borne by the pilaster as well as the wall. In this case we may design the element just as a wall supporting a concentrated load, taking advantage of the increase in the supporting area due to the pilaster projection. In case thickness of wall is less than 2/3 of the thickness of pilaster, we have to design the pilaster just like a column, for which permissible stress is less because of greater effective height and further supporting area will be only that of the pilaster that is, without getting any benefit in design of the adjoining walls on either side. However in case, the wall and pilasters are supporting a distributed load, we would get the advantage of stiffening effect of pilaster as in 4.5.2 of the Code.

4.4 – Effective Length

Effective length of a wall shall be as given in Table 7.

C4.4 – Effective Length

When a wall has more than one opening such that there is no opening within a distance of *H/8* from a cross wall and the wall length between openings are not columns by definition, the design of the wall should be based on the value of slenderness ratio obtained from the consideration of height or length, whichever is less.



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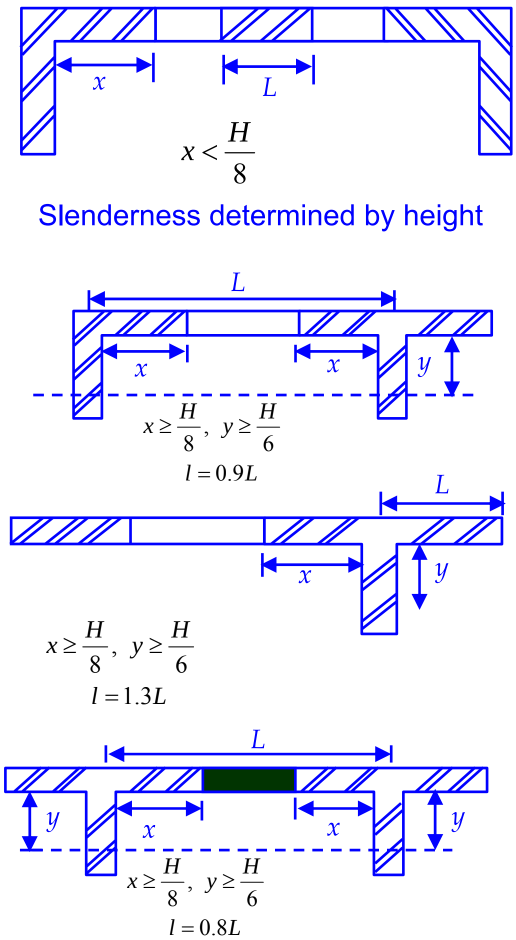

Figure 14: Effective Length of the Wall

Table 7: Effective Length Of Walls
(Clause 4.4)

SL No.	Condition Of Support (see Fig. 13)	Effective Length
(1)	(2)	(3)
1	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 OR Where a wall is continuous and is supported by ~~piers~~ pilaster /buttresses conforming to 4.2.1.2 (b)	*0.8 L*

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2	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~~ pilaster /buttress at one end and continuous with ~~pier~~ pilaster /buttress at other end conforming to 4.2.1.2 (b)	*0.9 L*
3	Where a wall is supported at each end by cross wall OR Where a wall is supported at each end by a ~~pier~~ pilaster /buttress conforming to 4.2.1.2 (b)	*1.0 L*
4	Where a wall is free at one end and is continuous with a cross wall at the other end OR Where a wall is free at one end and continuous with a ~~pier~~ pilaster /buttress at the other end conforming to 4.2.1.2 (b)	*1.5 L*
5	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~~ pilaster / buttress conforming to 4.2.1.2 (b)	*2.0 L*

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

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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 4.5.4.

4.5.1 –

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

C4.5.1 –

In case of masonry using modular bricks, actual thickness of a one-brick wall for design calculation is taken as 190 mm, though nominal thickness is 200 mm. Similarly in case of brick masonry with bricks of old size (FPS System) actual thickness of one-brick wall would be taken as 220 mm though nominal size of brick is 230 mm.

4.5.2 –

For solid walls adequately bonded into ~~piers~~ pilaster /buttresses, effective thickness for determining slenderness ratio based on

C4.5.2 –

When the ratio t_p/t_w is 1.5 or less and the wall is having distributed load, Note 4 of Table 6 would be

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effective height shall be the actual thickness of wall multiplied by stiffening coefficient as given in Table 8. 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 8 on the assumption that walls are equivalent to ~~piers~~ pilaster of width equal to the thickness of the cross wall and of thickness equal to three times the thickness of stiffened wall.

Table 8: Stiffening Coefficient For Walls Stiffened By Pilasters, Buttresses or Cross Walls
(Clauses 4.5.2 And 4.5.3)

SL. No.	S_p/w_p	Stiffening coefficient		
		$t_p/t_w = 1$	$t_p/t_w = 2$	$t_p/t_w = 3$ or more
(1)	(2)	(3)	(4)	(5)
1	6	1.0	1.4	2.0
2	8	1.0	1.3	1.7
3	10	1.0	1.2	1.4
4	15	1.0	1.1	1.2
5	20 or more	1.0	1.0	1.0

where

S_p = centre-to-centre spacing of the ~~piers~~ pilaster or cross wall,

t_p = the thickness of ~~piers~~ pilaster as defined in 2.3.2 (see Fig. 1),

t_w = actual thickness of the wall proper (see Fig. 1),

w_p = width of the ~~piers~~ pilaster 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 should be taken as two-thirds 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

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applicable. It follows from this that interpolation of values in Table 6 are valid only when t_p/t_w exceeds 1.5.

C4.5.4 –

It has been observed from tests that a cavity wall is 30 percent weaker than a solid wall of the same thickness as the combined thickness of two leaves of the cavity wall, because bonding action of ties cannot be as good as that of normal bond in a solid wall. That explains why effective thickness of a cavity wall is taken as two-thirds of the sum of the actual thickness of two leaves.

C4.5.5 –

In this type of wall either one leaf (inner) or both leaves could be load bearing. In the former case, effective thickness will be two-thirds the sum of

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of the cavity wall shall be two-thirds 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 – Effective span

4.6.1 –

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

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

4.6.2 –

Effective span of a cantilever shall be taken as

a) distance between the end of cantilever and the center of it's support
b) distance between the end of cantilever and the face of support plus half it's effective depth whichever is greater.

4.7 – Slenderness Ratio

4.7.1 – 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.

~~Table 7: Maximum slenderness ratio for a load bearing wall~~		
~~No. of storey~~	~~Maximum slenderness ratio~~	
	~~Using Portland Cement or Portland Pozzolana Cement in Mortar~~	~~Using Lime Mortar~~
~~(1)~~	~~(2)~~	~~(3)~~

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the two leaves or the actual thickness of the loaded leaf whichever is more. In the latter case effective thickness will be two-thirds of the sum of thickness of both the leaves, or the actual thickness of the stronger leaf, whichever is more.

C4.6.2 –

In case, it forms the end of a continuous beam, the length to the center of support should be taken.

C4.7 – Slenderness Ratio

The limits on the ratio of wall thickness (or column lateral dimension) to distance between lateral support is specified to exercise a control on the flexural tension stress within the wall (or column) and limits possible buckling under compressive stresses. Masonry wall or column should be laterally supported in horizontal and vertical direction at intervals not exceeding those given in Sec. 4.6.1 and 4.6.2, lateral support should be provided by cross walls, pilasters/buttresses, structural frames when limiting horizontal distances and floor and roof diaphragms, and structural frames should be used when limiting distance is taken vertically.

C4.7.1 – Walls

Under a vertical load a wall would buckle either around a horizontal axis parallel to the length of the wall or around a vertical axis as illustrated in Figure C15. Buckling is resisted by horizontal supports such as floors and roofs, as well as by vertical supports such as cross walls, piers and buttresses. Thus capacity of the walls to take vertical loads depends both on horizontal supports that is, floor or roof as well as on vertical supports that is, cross walls, piers and buttresses. However, for the sake of simplicity and erring on safe side, lesser of the two slenderness ratio namely, one derived from height and the other derived from length is taken into consideration for determining permissible stresses in masonry walls, thus

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~~Not exceeding 2~~	~~27~~	~~20~~
~~Exceeding 2~~	~~27~~	~~13~~

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ignoring strengthening effect of other supports.

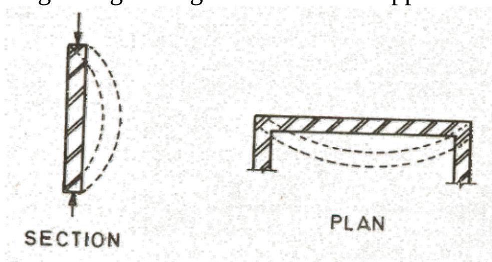

Figure C15: Buckling of walls

4.7.1.1 –Wall

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 27.

C4.7.1.1 –Wall

Load carrying capacity of a masonry member depends upon its slenderness ratio. As this ratio increases, crippling stress of the member gets reduced because of limitations of workmanship and elastic instability. A masonry member may fail, either due to excessive stress or due to buckling (see Figure C15). For materials of normal strength with SR less than 30, the load carrying capacity of a member at ultimate load is limited by stress, while for higher value of SR failure is initiated by buckling. Further, mode of failure of a very short member having *h/t* ratio of less than 4 is predominantly through shear action, while with *h/t* = 4 or more failure is by vertical tensile splitting. From consideration of structural soundness and economy of design, most codes control the maximum slenderness ratio of walls and columns so as to ensure failure by excessive stress rather than buckling.

4.7.2 – 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.

C4.7.2 – Columns

Limiting values of slenderness ratio for column is less than that of walls because column can buckle around either of the two horizontal axes where walls can buckle around horizontal axis only. In case of columns, there will be two values of slenderness ratio as illustrated in Fig 8 of code. For the purpose of a design, higher of the two values is taken into account since column will buckle around that axis with reference to which the value of slenderness ratio is critical ie., greater.

4.8 – Minimum Design Dimensions

4.8.1 - Minimum Thickness of Load Bearing Walls

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

4.8.2 - Parapet Wall

Parapet walls shall be at least 200mm thick

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and height shall not exceed 3 times the thickness. The parapet wall shall not be thinner than the wall below.

4.8.3 –

Minimum dimension shall be 200 mm. Slenderness ratio shall not exceed 20.

4.9 –

4.9.1 – 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. Exact calculations are not 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 Appendix A.

COMMENTARY

C4.8.3 –

Due to the structural importance of columns and their vulnerability as isolated members, many codes specify a 200 mm nominal minimum dimension.

C4.9 –

C4.9.1 – Eccentricity

Eccentricity of vertical loading on a masonry element increases its tendency to buckling and reduces its load carrying capacity; its effect is thus similar to that of slenderness of the member. Thus combined effect of slenderness and eccentricity is taken into consideration in design calculations by the factor known as Stress reduction factor (k_s) as given in Table 11 of the Code.

Eccentricity caused by an eccentric vertical load is maximum at the top of a member, that is, at the point of loading and it is assumed to reduce linearly to zero at the bottom of the member that is, just above the bottom lateral support, while eccentricity on account of slenderness of a member is zero at the two supports and is maximum at the middle. Taking the combined effect of eccentricity of loading and slenderness critical stress in masonry occurs at a section *0.6H* above the bottom support as shown in Figure C16.

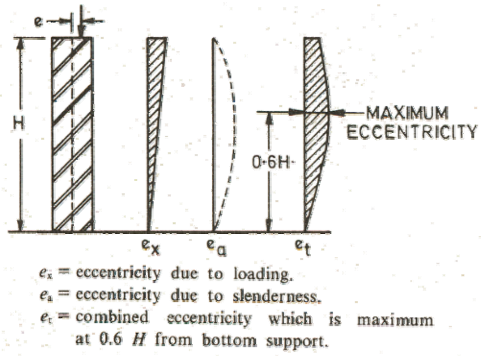


Figure C16: Eccentricity of loading on a wall

For the sake of simplicity, however, in design calculations, it is assumed that critical section in a storey height is at the top of bottom support and masonry is designed accordingly. In other words the design method commonly adopted includes extra self weight of *0.6H* of the member and thus errs on the safe side to some extent. In view of the fact that design calculations for masonry are not very precise, the above approximation is justified.

PROVISIONS

4.9.2 –

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

COMMENTARY

C4.9.2 –

Columns are generally not subjected to perfectly concentric axial loads. Eccentricity due to imperfections, lateral loads, and eccentrically applied axial loads occur almost always and they must be considered in design. Hence many masonry codes require a minimum eccentricity of 10% of side dimension.

PROVISIONS

5. – STRUCTURAL DESIGN

5.1 – General

The building as a whole shall be analyzed 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.

COMMENTARY

C5. – STRUCTURAL DESIGN

C5.1 – General

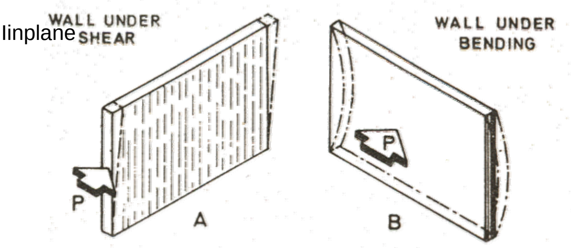
Some general guidance on the design concept of load bearing masonry structures is given in the following paragraphs.

i) A building is basically subjected to two types of loads, namely:

a) vertical loads on account of dead loads of materials used in construction, plus live loads due to occupancy; and

b) lateral loads due to wind and seismic forces. While all walls in general can take vertical loads, ability of a wall to take lateral loads depends on its disposition in relation to the direction of lateral load. This could be best explained with the help of an illustration.

In Figure C17, the wall *A* has good resistance against a lateral load, while wall *B* offers very little resistance to such load. The lateral loads acting on the face of a building are transmitted through floors (which act as horizontal beams) to cross walls which act as shear walls. From cross walls, loads are transmitted to the foundation. This action is illustrated in Fig. C18. Stress pattern in cross walls due to lateral loads is illustrated in Fig. C19.

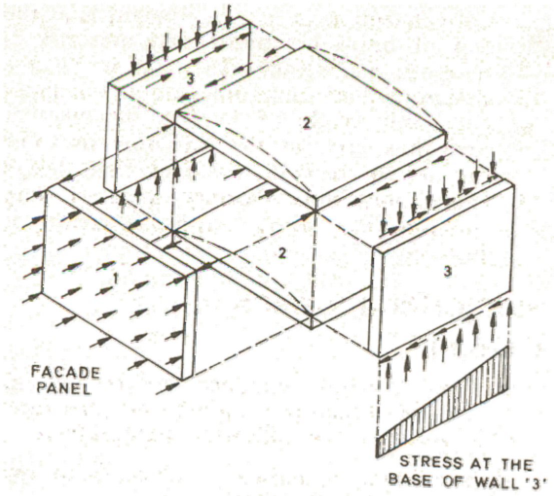


Resistance of brick wall to take lateral loads is greater in case of wall A than that in case of wall B.

Figure C17: In plane and outer plane lateral loads

PROVISIONS

COMMENTARY



Wind load on the facade wall 1 is transferred via floor slabs 2 to the cross walls 3 and thence to the ground.
The strength and stiffness of 2 that is floors as horizontal girders is vital; floors of lightweight construction should be used with care.

Figure C18: Function of lateral support to wall

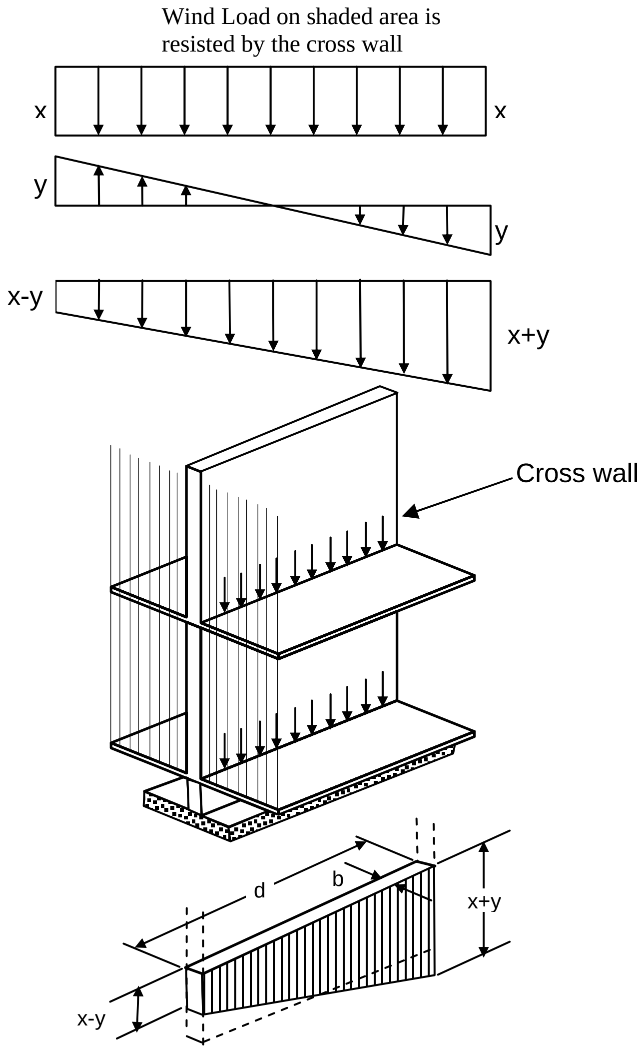


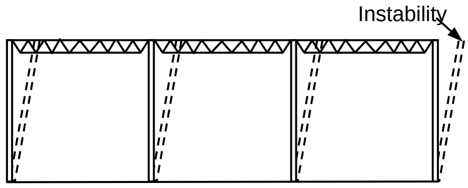
Figure C19: Bending stress pattern in cross walls acting as shear wall

PROVISIONS

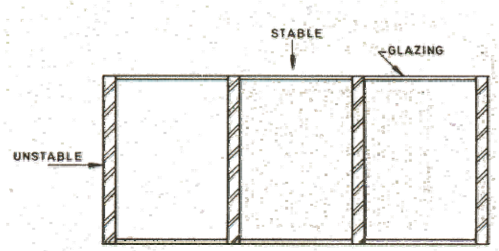
COMMENTARY

ii)As a result of lateral load, in the cross walls there will be an increase of compressive stress on the leeward side, and decrease of compressive stress on the wind-ward side. These walls should be designed for 'no tension' and permissible compressive stress. It will be of interest to note that a wall which is carrying greater vertical loads will be in a better position to resist lateral loads than the one which is lightly loaded in the vertical direction. This point should be kept in view while planning the structure so as to achieve economy in structural design.

iii)A structure should have adequate stability in the direction of both the principal axes. The so called 'cross wall' construction may not have much lateral resistance in the longitudinal direction. In multi-storeyed buildings, it is desirable to adopt 'cellular' or 'box type' construction from consideration of stability and economy as illustrated in Figure C20.

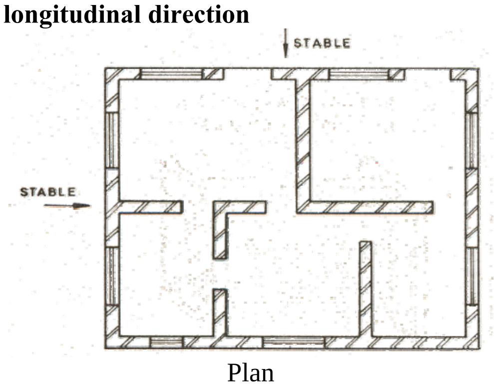


Elevation



Plan

Figure C20-A Cross wall construction-unstable in longitudinal direction



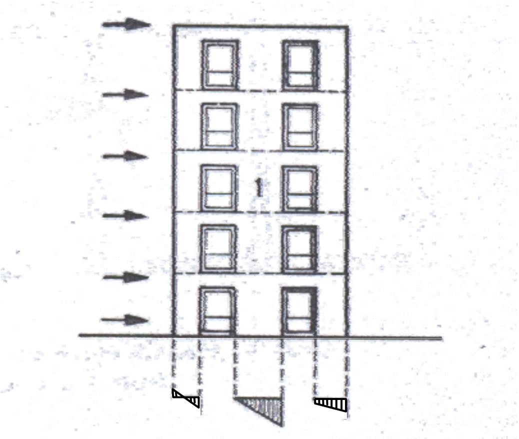
Plan

Figure C20-B Cellular or box type construction stable in both directions

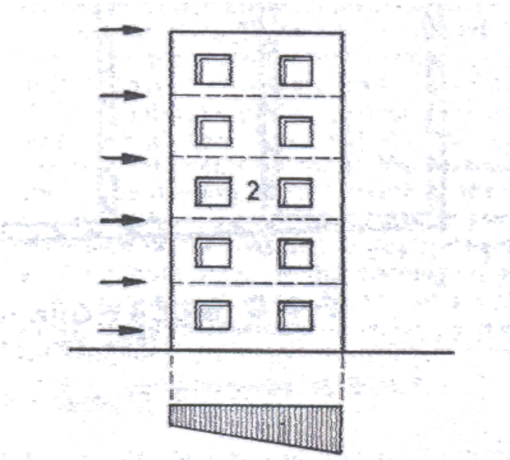
Figure C20: Stability of cross wall and cellular (box type) construction

PROVISIONS | **COMMENTARY**

iv) Size, shape and location of openings in the external walls have considerable influence on stability and magnitude of stresses due to lateral loads. This has been illustrated in Figure C21.



This wall will not resist lateral loading as effectively as wall 2; it tends to act as three separate short lengths rather than one.



This wall will tend to act as one long portion of brickwork and will be more resistant to lateral loading.

Figure C21: Effect of openings on shear strength of walls

v)If openings in longitudinal walls are so located that portions of these walls act as flanges to cross walls, the strength of the cross walls get considerably increased and structure becomes much more stable, as will be seen from Figure C22.

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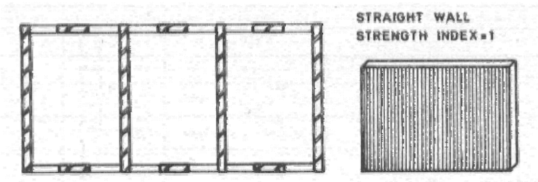

Figure C22-A Brickwork resisting shear (for all four walls)

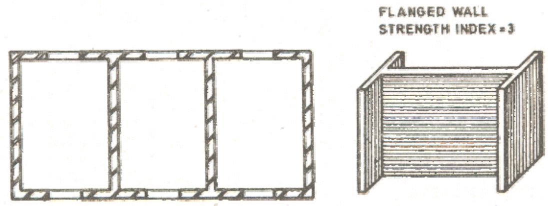

Figure C22-B Brickwork resisting shear (for two central walls)

vi)Ordinarily a load-bearing masonry structure is designed for permissible compressive and shear stresses (with no tension) as a vertical cantilever by accepted principles of engineering mechanics. No moment transfer is allowed for, at floor to wall connections and lateral forces are assumed to be resisted by diaphragm action of floor/roof slabs, which acting as horizontal beams, transmit lateral forces to cross walls in proportion to their relative (moment of inertia). Various modes of failure of masonry are illustrated in Figure C23.

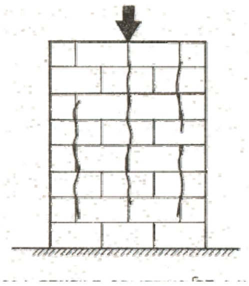

Figure C23-A: Tensile splitting of a wall under vertical compressive load.

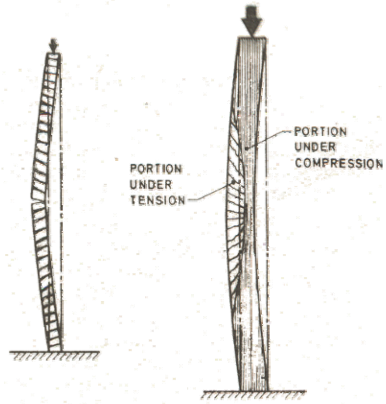

Figure C23-B: Buckling of a wall under vertical compressive load

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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) live loads of floors and roof;
c) wind loads on walls and sloping roo~~t~~f and
d) Seismic forces.

NOTE -When a building is subjected to other loads, such as vibration from railways and machinery, these should be taken into

COMMENTARY

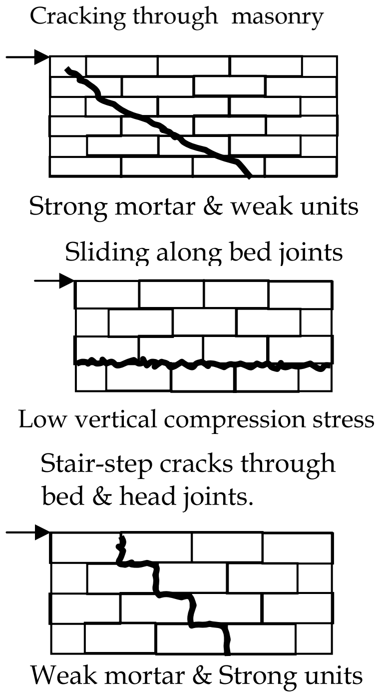

Figure C23-C: Shear failure of a masonry cross wall under lateral loading

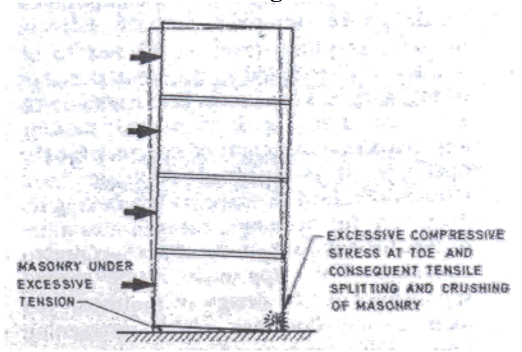

Figure C23-D: Excessive compressive stress in cross walls resulting in crushing of masonry at the toe under lateral loading

Figure C23: Various modes of failure of masonry

PROVISIONS

consideration according to the best judgement of the designer.

5.2.1 – Dead Loads

Dead loads shall be calculated on the basis of unit weights taken in accordance with IS: ~~1911-1967*~~IS:875-I (1987).

*Schedule of unit weights of building materials *(First revision).*

5.2.2 – Live Loads and Wind Loads

Design loads shall be in accordance with the recommendations of IS: 875-~~1964†~~1987 or such other loads and forces as may reasonably be expected to be imposed on the structure either during or after construction.

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.

† Code of practice for structural safety of buildings: Loading standards *(revised).*

5.2.3 – Seismic Loads

~~For buildings to be constructed in seismic zones I and II (see IS: 1893-1984), it is not necessary to consider seismic forces in design calculations. In seismic zones III, IV and V, strengthening measures suggested in IS: 4326-1976§ shall be adopted.~~

~~*‡ Criteria for earthquake resistant design of structures (Fourth revision).*~~

~~*§ Code of practice for earthquake resistant design and construction of buildings (First revision).*~~

Seismic loads shall be determined in accordance with the IS 1893- Part 1:2002.

5.2.4 – Load combinations

In the allowable stress design method followed for the structural design of masonry structures as outlined in this code, 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

5.2.5 – Permissible stresses and loads

Permissible stresses and loads may be increased by one-third for load case b, c, & d of Clause 5.2.4 when wind or earthquake

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C 5.2.4 – Load Combinations

The four load combinations given are consistent with those in other BIS codes. In case of wind and earthquake loads, the reversal of forces needs to be considered. The structure is to be designed for the critical stresses resulting from these load combinations.

C 5.2.5 – Permissible stresses and loads

Traditionally, a 33% increase in permissible stress values has been permitted when considering wind or earthquake forces on a structure. Though the rationale behind this increase has been subject of

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loads are considered along with normal loads.

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some criticism, it is permitted by the code in the absence of more reliable information.

As an alternative of using an increased permissible stress value when checking safety of structural components, one can use a 25% reduced load for load combinations involving wind or earthquake forces and compare with full permissible stress values. Thus, the modified load combinations b, c and d will be:

b) 0.75 [DL + IL + (WL or EL)]
c) 0.75 [DL + WL]
d) 0.75 [0.9DL +EL]

5.3 – Vertical Load Dispersion

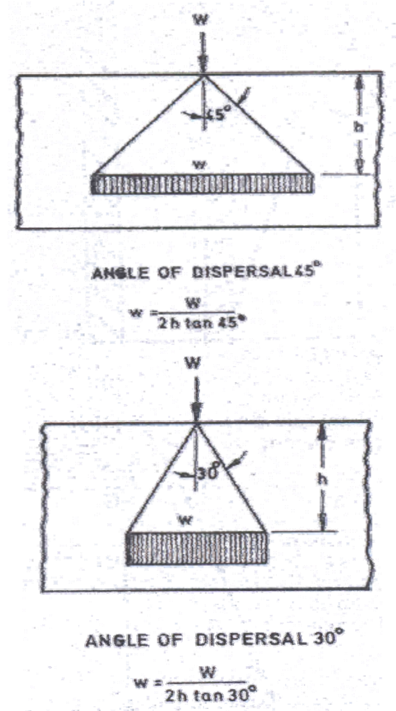
5.3.1 – General

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

C5.3 – Vertical Load Dispersion

C5.3.1 – General

Experiments have shown that dispersion of axial loads does not take place at an angle 45° to vertical as assumed in previous codes. An angle of distribution for axial loads not exceeding 30° is more realistic and is recommended by various other masonry codes. (see Figure C24).



W = Concentrated load
w = Distributed load after dispersal at depth h from plane of application of concentrated load

Figure C24: Dispersal of concentrated load in masonry

5.3.2 – Arching Action

Account may also be taken of the arching action of well-bonded masonry walls

C5.3.2 – Arching Action

i) Arching in masonry is a well known phenomenon by which part of the load over an

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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 section 5.6.

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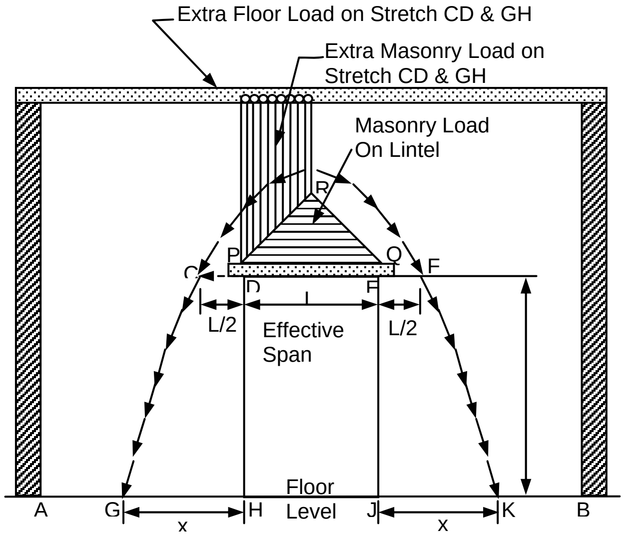
opening in the wall gets transferred to the sides of the opening. For good arching action masonry units should have good shear strength and these should be laid in proper masonry bond using a good quality mortar. Further, portions of the wall on both sides of the opening should be long enough (see C-6.3.3) to serve as effective abutments for the arched masonry above the opening since horizontal thrust for the arch is to be provided by the shear resistance of the masonry at the springing level on both sides of the opening. If an opening is too close to the end of a wall, shear stress in masonry at springing level of imaginary arch may be excessive and thus no advantage can be taken of arching in masonry for design of lintels.

ii)To explain the effect of arching on design of lintels and stress in masonry, let us consider a wall of length *AB* with an opening of effective span *PQ* = *L* as shown in Figure C25 PRQ is an equilateral triangle with PQ as its base.



X= L or (L+H)/2, whichever is less

Figure C25: Arching action in masonry

Because of arching action, loads of floor and masonry above the equilateral triangle get transferred to the sides of the wall. Therefore lintel at PQ is designed for load of masonry contained in the triangle PRQ.

To work out approximate stress in masonry in various stretches, it is assumed that:

a) load from the lintel gets uniformly distributed over the supports,

b) masonry and floor loads above the triangle PRQ get uniformly distributed over the stretches of masonry CD and EF at the soffit level of the lintel, CD and EF being limited in length to *L/2* and over the stretches GH and JK at the floor

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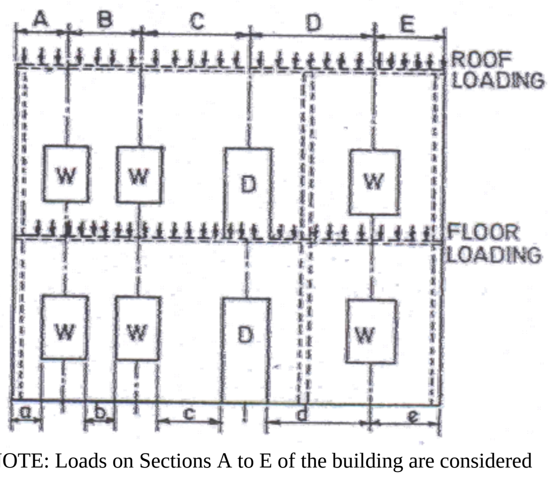
level, limited in length to *L* or *(L-H)/2* whichever is less, *H* being the height of top of the opening from the floor level.
In case some other opening occurs between the lintel and horizontal plane 25 cm above the apex R of the triangle, arching action gets interrupted because of inadequate depth of masonry above the triangle to function as an effective arching ring. Also if there is some other load between the lintel and horizontal plane 25 cm above the apex R of the triangle, loading on the lintel gets affected.

iii) In case of buildings of conventional design with openings of moderate size which are reasonably concentric, some authorities on masonry recommend a simplified approach for design. In simplified approach, stress in masonry at plinth level is assumed to be uniformly distributed in different stretches of masonry, taking loadings in each stretch as indicated in Figure C26 without making any deduction in weight of masonry for the openings. It is assumed that the extra stresses obtained in masonry by making no deduction for openings, compensates more or less for concentrations of stresses due to openings. This approach is of special significance in the design of multi-storeyed load-bearing structure where intervening floor slabs tend to disperse the upper storey loads more or less uniformly on the inter-opening spaces below the slabs and thus at plinth level stress in masonry, as worked out by the above approach is expected to be reasonably accurate.



NOTE: Loads on Sections A to E of the building are considered to be acting on wall lengths *a* to *e* respectively

Figure C26: Stresses in masonry at different floor levels

5.3.3 – Lintels

Lintels, that support masonry construction, shall be designed to carry loads for masonry (allowing for arching and dispersion, where

C5.3.3 – Lintels

i) Lintels over openings are designed taking into consideration arching action in masonry where feasible as explained earlier. It is a common

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applicable) and loads received from any other part of the structure. Length of bearing of lintel at each end shall not be less than 9 cm 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 5.6 (see AppendixC).

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practice to assume that length of walls on both sides of an opening should be at least half the effective span of the opening for transfer of load to sides by arch action. In case it is less, lintel should be designed for full load over the opening regardless of the height of the floor slab as shown in Figure C27-A.

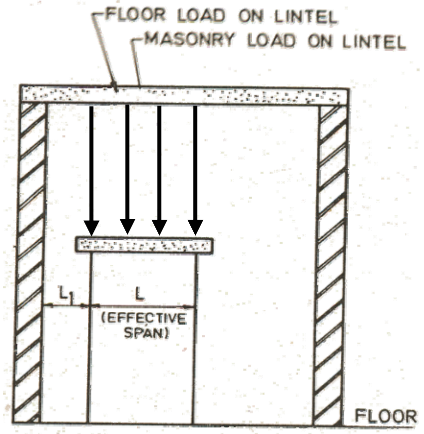

Figure C27-A: Effective load when $L_1 < L/2$

ii) When location and size of opening is such that arching action can take place, lintel is designed for the load of masonry included in the equilateral triangle over the lintel as shown in Figure C27-B. In case floor or roof slab falls within a part of the triangle in question or the triangle is within the influence of a concentrated load or some other opening occurs within a part of the triangle, loading on the lintel will get modified as discussed earlier.

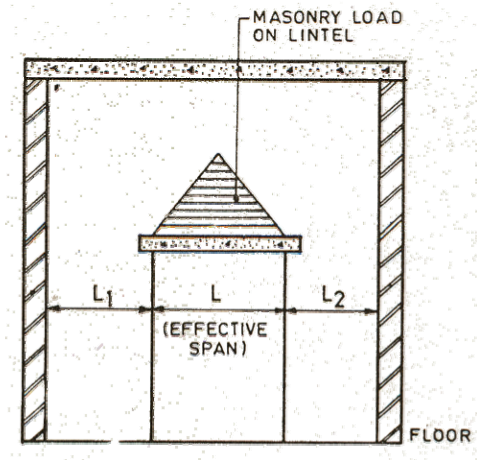

Figure C27-B: Effective load when L_1 and $L_2 \geq L/2$ and floor/roof slab does not intercept the equilateral over the lintel.

iii) When stretches of wall on sides are equal to or greater than *L/2* and equilateral triangle above the lintel is intercepted by the floor / roof slab, the lintel is designed for load of masonry contained in the equilateral triangle plus load from the floor falling within the triangle as shown in Figure C27-C.

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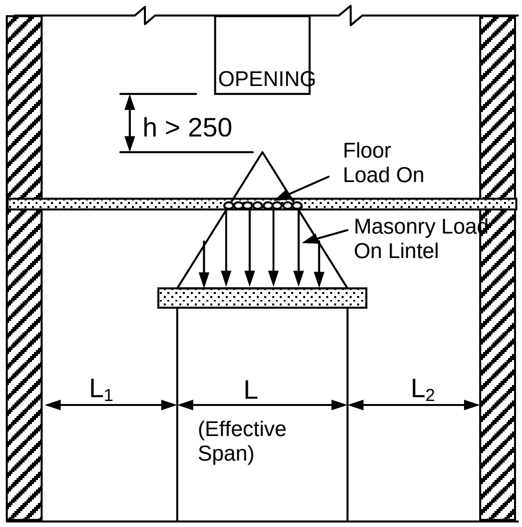

Figure C27-C: Effective load when L_1 and $L_2 \geq L/2$, and equilateral triangle over the lintel is intercepted by floor slab above with no other opening to intercept arch action

iv) When stretches of wall on the sides of the opening are equal to or greater than $L/2$ with the equilateral triangle over the lintel intercepted by floor slab and another opening comes within the horizontal plane 25 cm above the apex of the triangle, lintel is to be designed for loads shown in Fig. C27-D.

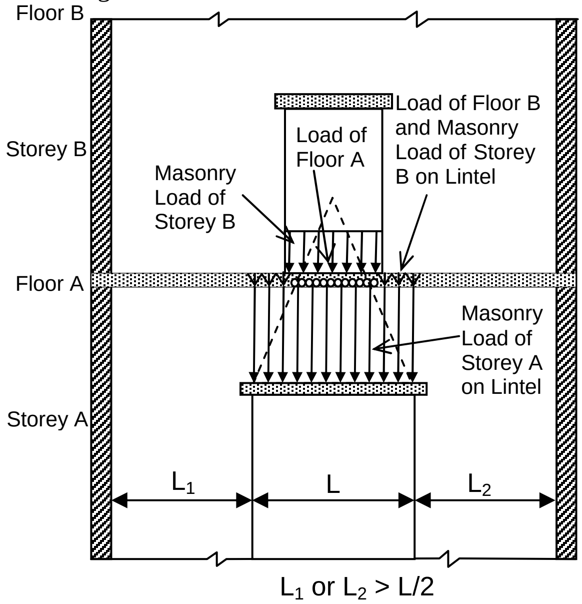

Figure C27-D: Effective load when L_1 and $L_2 \geq L/2$ and equilateral triangle above the lintel is within 25 cm (vertically) of another opening in the upper storey.

v) When any other load is coming between the

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lintel and horizontal plane 25 cm above the apex of the equilateral triangle over the lintel, the latter is designed for the loads as shown in Fig. C-27-E.

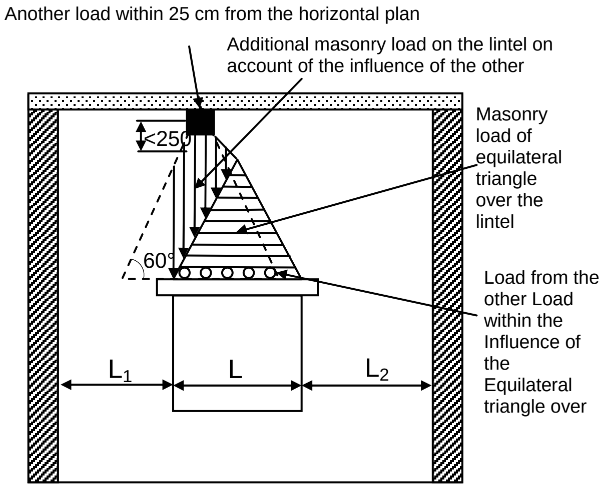

Figure C27-E: Effective load when L_1 and $L_2 \geq L/2$ and equilateral triangle is within the influence of another load
Figure C27: Effective loads on lintels for various situations.

vi) It may be clarified that in fact load coming on a lintel is indeterminate and the above suggestions for the design of lintels are based on empirical rules derived from experience and general principles of engineering.

v) Economy in the design of lintels may be effected by taking advantage, of composite action between lintel and the masonry above it. For this purpose, shuttering of the lintel should not be removed till both masonry (up to 250 mm above the apex of equilateral triangle above the lintel) and RCC of the lintel have gained sufficient strength so as to be able to bear stresses in the composite beam having masonry in compressive zone and RCC lintel in the tensile zone. Behavior of composite beam in this case is analogous to that of grade beam in pile foundation.

5.4 – Lateral Load Distribution

Lateral loads shall be distributed to the structure system in accordance with member stiffness for rigid diaphragms or tributary areas for flexible diaphragms and shall comply with the following requirements

1. Flanges of intersecting walls designed in accordance with section 4.2.2.5 shall be included in stiffness determination.
2. Distribution of load shall include the effect

C6.4 – Lateral Load Distribution

Lateral loads from the wind or earthquakes are generally considered to act in the direction of the principal axes of the building structure. The distribution of lateral loads to various masonry wall elements depends on the rigidities of the horizontal floor or roof diaphragm and of the wall elements.

If a diaphragm does not undergo significant in-plane deformation with respect to the supporting

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of diaphragm rigidity and of horizontal torsion due to eccentricity of wind and seismic loads resulting from non-uniform distribution of mass.

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walls, it can be considered rigid and lateral loads are distributed in various lateral load resisting wall elements in proportion to their relative stiffness. Horizontal torsion developed due to eccentricity of the applied lateral load with the plan centre of the rigidity can cause forces in the wall parallel and perpendicular to load direction. In-plane rigidities are considered in the analysis, which includes both shearing and flexural deformations. Generally rigidities of transverse walls in direction perpendicular to the direction of lateral force, is usually disregarded. However, stiffening effect of certain portion of such walls as permitted by the Code in section 4.2.2.5 can be considered, if the method of connection between the intersecting walls and between walls and diaphragms is adequate for the expected load transfer.

On the other hand, flexible diaphragms change shape when subjected to lateral loads and are incapable of transmitting torsional forces. The distribution of lateral loads to vertical wall elements takes place in proportion to the tributary area associated with each wall element for vertical loads distribution.

5.5 – Basic Compressive Strength of Masonry

The basic compressive strength of masonry f_m shall be determined by the unit strength method or by the prism test method as specified below.

C6.5 – Basic Compressive Strength of Masonry

The code proposes two methods to determine the compressive strength of masonry. The unit strength method eliminates the expense of prism tests but is more conservative than the prism test method

5.5.1 – Unit Strength Method

The basic compressive strength of masonry shall be four times of the basic compressive stress which based on the strength of the units and the type of mortar as given in Table 10.

C6.5.1 – Unit Strength Method

Unit strength method is based on the compressive strength of masonry units and mortar type, and is developed by using prism test data.

5.5.2 – Prism Test Method

Basic compressive strength of masonry shall be determined by prism test as given in Appendix B on masonry made from masonry units and mortar to be actually used in a particular job.

C6.5.2 – Prism Test Method

This is a uniform method of testing masonry to determine its compressive strength and is used as an alternative to the unit strength method.

5.6 – Permissible Stresses

5.6.1 – Basic Compressive Stress

Permissible compressive stress in masonry shall be based on the value of basic compressive stress (f_b) as ~~given in Table 8 and multiplying this value by factor known as stress reduction factor (k,). Area reduction factor (k,) and shape modification factor~~ ~~(k_p) as~~

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~~detailed in 5.4.1.1 to 5.4.1.3.~~given below:

(a) **Prism not tested/Unit Strength Method**:

Values of basic compressive stress given in Table 10 which are based on the crushing strength of masonry unit and grades of mortar, and hold good for values of SR not exceeding 6, zero eccentricity and masonry unit having height to width ratio (as laid) equal to 0.75 or less.

(b) **Prisms tested** :

The basic compressive stress can be obtained by multiplying the specified compressive strength obtained from prism test with 0.25

5.6.2 – Permissible Compressive Stress

Permissible compressive stress in masonry shall be based on the value of basic compressive stress (f_b) as given in Table 10 and multiplying this value by factor known as stress reduction factor (k_s). Area reduction factor (k_a) and shape modification factor (k_p) as detailed in 6.4.1.1 to 6.4.1.3.

Sl. no	Mortar Type	Table 10: Basic compressive strength in MPa corresponding to masonry units of which height to width ratio does not exceed 0.75 and crushing strength in MPa is not less than											
		3.5	**5.0**	**7.5**	**10**	**12.5**	**15**	**17.5**	**20**	**25**	**30**	**35**	**40**
1	**H1**	0.35	0.50	0.75	1.00	1.16	1.31	1.45	1.59	1.91	2.21	2.50	3.05
2	**H2**	0.35	0.50	0.74	0.96	1.09	1.19	1.30	1.41	1.62	1.85	2.10	2.50
3	**M1**	0.35	0.50	0.74	0.96	1.06	1.13	1.20	1.27	1.47	1.69	1.90	2.20
4	**M2**	0.35	0.44	0.59	0.81	0.94	1.03	1.10	1.17	1.34	1.51	1.65	1.90
5	**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
6	**L1**	0.25	0.36	0.53	0.67	0.76	0.83	0.90	0.97	1.11	1.26	1.40	1.06
7	**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

5.6.2.1 –Stress reduction factor

This factor, as given in Table 11, takes into consideration the slenderness ratio of the element and also the eccentricity of loading

C6.6.2.1 – Stress reduction factor

Since slenderness of a masonry element increases its tendency to buckle, permissible compressive stress of an element is related to its slenderness ratio and is determined by applying Stress reduction factor (k_s) as given in Table 11 of the Code. Values of Stress reduction factor have been worked out by taking into consideration eccentricity in loading because of slenderness. Strictly speaking full value of stress reduction factor is applicable only for central one-fifth height of the member. In practice however for the sake of simplicity in design calculations, stress reduction factor is applied to the masonry throughout its storey height (Note 3 under Table 11 of the Code is an exception) and for designing masonry for a

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Table 11: Stress reduction factor for slenderness ratio and eccentricity(Clause *5.6.2.1*)						
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)
6	1.00	1.00	1.00	1.00	1.00	1.00
8	0.95	0.95	0.94	0.93	0.92	0.91
10	0.89	0.88	0.87	0.85	0.83	0.81
12	0.84	0.83	0.81	0.78	0.75	0.72
14	0.78	0.76	0.74	0.70	0.66	0.66
16	0.73	0.71	0.68	0.63	0.58	0.53
18	0.67	0.64	0.61	0.55	0.49	0.43
20	0.62	0.59	0.55	0.48	0.41	0.34
22	0.56	0.52	0.48	0.40	0.32	0.24
24	0.51	0.47	0.42	0.33	0.24	-
26	0.45	0.40	0.35	0.25	-	-
27	0.43	0.38	0.33	0.22	-	-

NOTE 1 - Linear interpolation between values is permitted.

NOTE 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.

NOTE 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.

5.6.2.2 – 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 $k_a = 0.7 + 1.5\,A$, A being the area of section in m^2.

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particular storey height, generally stress is worked out at the section just above the bottom support assuming it to be maximum at that section. Theoretically critical section in a storey occurs at a height *0.6 H* above the bottom support as explained in C-4.8. Thus provisions of the Code and the design procedure in question as commonly followed, is an approximation that errs on the safe side.

Advantage of Note 3 under Table 11 of the Code is taken when considering bearing stress under a concentrated load from a beam. Bearing stress is worked out immediately below the beam and this should not exceed the permissible compressive stress of masonry. Also stress in masonry is worked out at a depth of *H/8* from the bottom of the beam. This should not exceed the permissible compressive stress in masonry. If actual stress exceeds allowable stress in either case, a concrete bed block is provided below the beam.

In accordance with 5.6.2.5 of the Code, some increase in permissible compressive stress is allowed for concentrated loads which are concentric. For checking bearing stress under such a load, however, some authorities on masonry recommend a conservative approach-that is, either to take advantage of Note 3 of Table 11 of the Code or to take advantage of provisions of 5.6.2.5 of the Code but do not apply both the provisions of the code at the same time**.**

C5.6.2.2 – Area Reduction Factor

Area reduction factor due to 'small area' of a member is based on the concept that there is statistically greater probability of failure of a small section due to sub-standard units as compared to a large element. However North American Codes do not include any provision for smallness of area. The reason for this seems to be that factor of safety/load factors inherent in a Code should be enough to cover the contingency mentioned above for this provision. On the other hand, Australian Code (1974) and draft ISO standard (1987) provide this limit for smallness of area as 0.13 and 0.10 m^2, respectively. Strictly speaking necessity for this provision in the Code arises when there is appreciable variation in strength of individual units. In view of the fact that strength of masonry units being manufactured at present in our country can appreciably vary, the necessity for this provision is justified in our code.

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5.6.2.3 – 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 12. This factor is applicable for units of crushing strength up to 15 MPa.

Table 12: Shape Modification Factor For Masonry Units (Clause *5.6.2.3*)

Height to width ratio of units(as laid)	Shape modification factor (k_p) for units having crushing strength in MPa			
	5.0	7.5	10.0	15.0
(1)	(2)	(3)	(4)	(5)
Up to 0.75	1.0	1.0	1.0	1.0
1.0	1.2	1.1	1.1	1.0
1.5	1.5	1.3	1.2	1.1
2.0 to 4.0	1.8	1.5	1.3	1.2

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 be computed for the purpose of design.

5.6.2.4 – Increase in Permissible Compressive Stresses Allowed for Eccentric Vertical Loads and 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 be computed for the purpose of design.

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C5.6.2.3 – Shape Modification Factor

Shape modification factor is based on the general principle that lesser the number of horizontal joints in masonry, greater its strength or load carrying capacity. It has, however, been found from experimental studies that for units stronger than 15 MPa, extent of joints in masonry does not have any significant effect on strength of masonry because of use of the comparatively high strength mortar that normally goes with high-strength units.

C5.6.2.4 – Increase in Permissible Compressive Stresses Allowed for Eccentric Vertical Loads and Lateral Loads under Certain Conditions

i) Eccentric vertical load (vertical load plus lateral load in case of free standing walls) on masonry causes bending stress in addition to axial stress. It has been found that masonry can take 25 percent greater compressive stress, when it is due to bending than when it is due to pure axial load, because maximum stress in case of bending occurs at the extreme fibers and then it gets reduced linearly while in axial compression, stress is more or less uniform throughout the section. For similar reasons permissible compressive stress in concrete for beams also called bending compressive stress, is greater than that in columns subjected to vertical loads. This rule of higher permissible compressive stress when due to bending can also be explained from the consideration that beyond elastic limit redistribution of stresses takes place because of plasticity and thus stress block is in practice more or less rectangular in shape instead of triangular as is normally assumed in accordance with the elastic theory. This enables the member to resist greater load.

ii) When loading on a masonry element has some eccentricity, the Code lays down the design

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approach for various ranges of eccentricity ratios namely (a) eccentricity ratio of 1/24 or less; (b) eccentricity ratio exceeding 1/24 but not exceeding 1/6, and (c) eccentricity ratio exceeding 1/6. Basis of this design approach is explained below.

a) *Eccentricity ratio of 1/24 or less:*
Referring to Fig. C28-B, *W* is total permissible vertical load per unit length of wall with resultant eccentricity *e*, *t* is thickness of wall, f_1 and f_2 are the stresses at the two faces of the wall and $0.25f_m$ is Permissible compressive stress for axial loading.

$$f_1 = \frac{W}{A} + \frac{M}{Z}$$

$$f_2 = \frac{W}{A} - \frac{M}{Z}$$

Substituting values of *A*, *M* and *Z*

$$f_1 = \frac{W}{t} + \frac{We \times 6}{t^2} = \frac{W}{t}\left(1 + \frac{6e}{t}\right)$$

$$f_2 = \frac{W}{t} - \frac{We \times 6}{t^2} = \frac{W}{t}\left(1 - \frac{6e}{t}\right)$$

For eccentricity ratio $e/t = 1/24$, and since W/t is equal to permissible/allowable axial compressive stress F_a

$$f_1 = \frac{W}{t}\left(1 + \frac{1}{4}\right) = 1.25F_a$$

$$f_2 = \frac{W}{t}\left(1 - \frac{1}{4}\right) = 0.75F_a$$

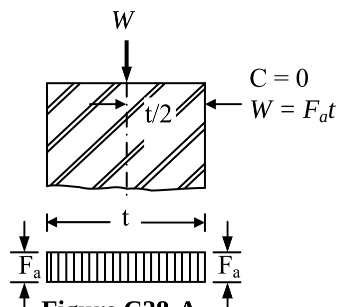

Figure C28-A

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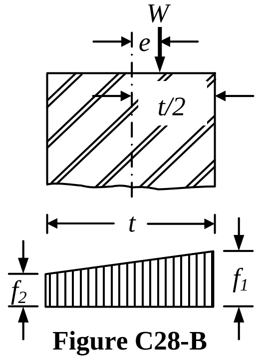

$e = t/24$
$f_1 = 1.25F_a$
$f_2 = 0.75F_a$
$W = F_a t$

Figure C28-B

As we allow 25 percent additional compressive stress in case of eccentric loading, it follows that maximum compressive stress (f_1) for eccentricity ratio up to 1/24 does not exceed axial compressive stress by more than 25 percent which is permitted by the code.

Therefore for eccentricity ratio of 1/24 or less, it is not necessary to compute and add bending stress to the axial stress. The designer is expected to work out only axial compressive stress for the purpose of design and see that it does not exceed Permissible compressive stress for axial load.

$\therefore$ Allowable Design load, $W = F_a t$ per unit length of wall.

b) *Eccentricity ratio exceeding 1/24 but not exceeding 1/6 (see Fig. C28-C and C28-D):*

Bending stress = $\dfrac{We \times 6}{t^2}$

For eccentricity ratios 1/6 (substituting in the above equations),

$$f_1 = \frac{W}{t} + \frac{W}{t} = \frac{2W}{t}$$

$$f_2 = \frac{W}{t} - \frac{W}{t} = 0$$

Thus on one face compressive stresses get doubled and on the other face it is fully nullified by tensile stress and there is no tension in the cross section. For loading with eccentricity ratio between 1/24 and 1/6, we have to limit the maximum stress f_1 to $1.25F_a$.

$$f_1 = \frac{W}{t}\left(1 + \frac{6e}{t}\right) = 1.25F_a$$

Therefore, Total allowable load

$$W = \frac{1.25F_a t}{\left(1 + \frac{6e}{t}\right)}$$

W

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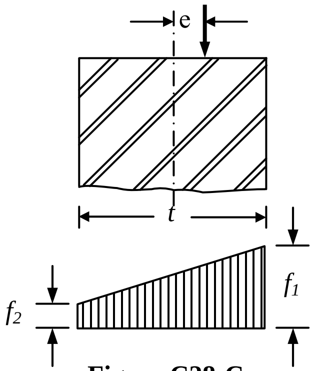


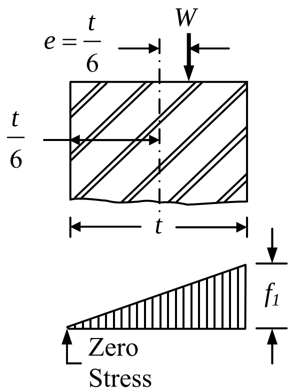
$t/24 < e < t/26$

$f_1 = 1.25F_a$

$= \frac{W}{t}\left(1 + \frac{6e}{t}\right)$

$W = \frac{1.25F_a t}{1 + \frac{6e}{t}}$

Figure C28-C



$e = \frac{t}{6}$

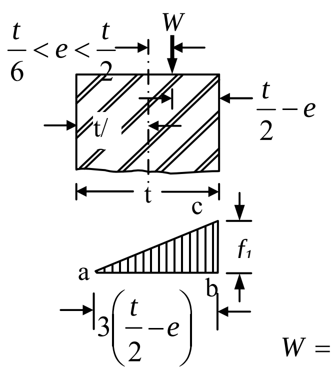
$f_1 = \frac{2W}{t} = 1.25F_a$

$W = \frac{1.25F_a \times t}{2}$

Figure C28-D

c) Eccentricity ratio exceeding 1/6 (see Fig. C28-E):

We had seen from (b) above that when eccentricity ratio reaches the value 1/6, stress is zero on one face; when this ratio exceeds 1/6 there will be tension on one face rendering ineffective a part of the section of the masonry and stress distribution in this case would thus be as shown in Fig. C28-E.



$\frac{t}{6} < e < \frac{t}{2}$

$f_1 = 1.25F_a$

$= \frac{2W}{3\left(\frac{t}{2} - e\right)}$

$W = \frac{1.25F_a \times 3\left(\frac{t}{2} - e\right)}{2}$

Figure C28-

Average compressive stress:

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$$f_{av} = \frac{f_1 + 0}{2} = \frac{f_1}{2}$$

Since f_1 has to be limited to $1.25F_a$

$$f_{av} = \frac{1.25F_a}{2}$$

The Total allowable load *W* in this case will be equal to average compressive stress multiplied by length *ab* of the stress triangle *abc*. Since for equilibrium, the load must pass through the

W = average stress × *ab*

$$= \frac{1.25 \times F_a}{2} \times 3(\frac{t}{2} - e)$$

centroid of the stress triangle *abc* and the load is at a distance of *t/2- e* from the compressive face, we get

$$\frac{ab}{3} = \frac{t}{2} - e \text{ and } ab = 3\left(\frac{t}{2} - e\right)$$

Thus Total allowable load,

W = average stress × *ab*

$$= \frac{1.25 \times F_a}{2} \times 3(\frac{t}{2} - e)$$

From the above equation we can see that theoretically design load *W* is zero when *e= t/2*. However for practical considerations *e* should be limited to *t/3*.

5.6.2.5 – 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 walls 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 load may, therefore, be worked out as per the best judgement of the designer. Some guidelines in this regard are given in Appendix C.

C5.6.2.5 – Increase in Permissible Compressive Stress for Walls Subjected to Concentrated Loads

In Appendix C of the Code, use of concrete bed block has been suggested. It seems necessary to add that in case some tension is likely to develop in masonry because of eccentricity of concentrated loads, the bed blocks should be suitably reinforced and these should be long enough so as to prevent tensile cracks in masonry due to eccentricity of loading.

5.7 – Combined Permissible Axial and Flexural Compressive Stress

5.7.1 –

Members subjected to combined axial compression and flexure shall be designed to satisfy the following:

C5.7 – Combined Permissible Axial and Flexural Compressive Stress

C5.7.1 –

The unity equation assumes a straight line interaction between axial and flexural compressive stresses for unreinforced masonry sections. This is simple portioning of the available allowable

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$$\frac{f_A}{F_a} + \frac{f_B}{F_b} \leq 1$$

Where,
f_a= Calculated compressive stresses due to axial load only
f_b= Calculated Compressive stresses due to flexure only
F_a = Allowable axial compressive stress
F_b = Allowable flexural compressive stress
= 1.25 F_a

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stresses between axial and flexure loads, which can be extended for the biaxial bending, by using the bending stress quotients for both axes. In this interaction formula, the secondary effect of moment magnification for flexure term due to axial loads is not included, which is an error on the unsafe side. However, this error for practical size of walls will be relatively small and large overall safety factor of about 4 is adequate to account for this amplification of flexure term.

The code allows 25% increase in allowable axial compressive stress, if it is due to flexure. The permissible flexural compressive stress can be expressed as a function of masonry prism strength as follows:

$F_b = 1.25\ F_a\ = 1.25 \times 0.25\ f_m\ =\ 0.31\ f_m$

.

5.8 – 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:

Grade M1 or Better mortar

- 0.07 MPa for bending in the vertical direction where tension developed is normal to bed joints.
- 0.14 MPa 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 MPa.

Grade M2 mortar

- 0.05 MPa for bending in the vertical direction where tension developed is normal to bed joints.
- 0.10 MPa 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 MPa.

NOTE 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.

C5.8 – Permissible Tensile Stress

Variables affecting tensile bond strength of brick masonry normal to bed joints include mortar properties, unit initial rate of absorption, surface condition, workmanship and curing condition. Also the aspect ratio of brick unit has a significant effect on the flexural tensile strength. The increase in aspect ratio of the unit results in an increase in strength parallel to bed joints and a decrease in strength normal to bed joints.

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NOTE 2- Allowable tensile stress in bending in the vertical direction may be increased to 0.1 MPa for M1 mortar and 0.07 MPa for M2 mortar in case of boundary walls.

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In accordance with Note 2 of the clause tensile stress up to 0.1 MPa and 0.07 MPa in the masonry of boundary/compound walls is permitted when mortar used in masonry is of M1 and M2 grade respectively or better. This relaxation has been made to effect economy in the design of the boundary/compound walls since there is not much risk to life and property in the event of failure of such walls.

5.9 - Permissible Shear Stress

5.9.1.1–

In-plane permissible shear stress (F_v)shall not exceed any of :

a) 0.5 MPa
b) $0.1+0.2f_d$
c) $0.125\sqrt{f_m}$

Where,

f_d = compressive stress due to dead loads in N/mm^2

C5.9 – Permissible Shear Stress

C5.9.1.1 –

Unreinforced masonry in shear fails in one of the following mode as shown in Fig.C23-C: (a) Diagonal tension cracking of masonry generally observed when masonry is weak and mortar is strong, (b) Sliding of masonry units along horizontal bed joint, especially when masonry is lightly loaded in vertical direction and (c) Stepped cracks running through alternate head and bed joints, usually observed in case of strong units and weak mortars.

Permissible shear stress for unreinforced masonry is based on experimental research for various failure modes. At low pre-compression (<2 MPa), for sliding type of failure mode, a Mohr-Coulomb type failure theory is more appropriate and shear capacity is increased due to increase in the vertical load (Fig.C29). The coefficient of friction of 0.2 has been long used in the masonry codes, however, the recent research indicate that a higher value (about 0.45) is more appropriate. At large pre-compression (> 2 MPa), tensile cracking of masonry is more likely which are expressed in terms of square root of compressive strength of masonry.

Shear stress due to applied loads shall be determined based on the net section properties using the following expression:

$$F_v = \frac{VQ}{Ib}$$ where,

V = design shear force,
Q = first moment about the neutral axis of a section of that portion of the cross section lying between the neutral axis and extreme fiber,
I = moment of inertia of masonry, and
b = width of section.

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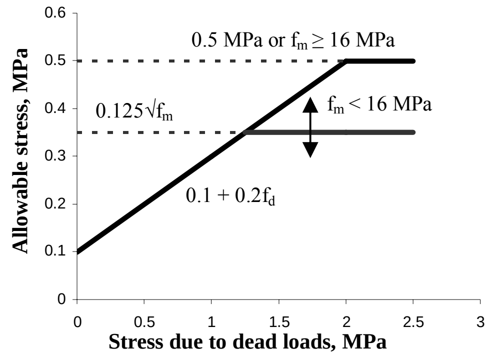

Figure C30: Allowable Shear for Unreinforced Walls

For rectangular section this amounts to parabolic stress distribution and the maximum value will be 1.5 times the average shear stress.

5.10 – Design Criteria: Wall Thickness/Cross-Section and Dimensions

5.10.1 – Walls and Columns Subjected to Vertical Loads

Walls and columns bearing vertical loads shall be designed on the basis of permissible compressive stress. Design involves in determining thickness in case of walls and the 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, further to provisions of 5.9.1.1 to 5.9.1.4

5.10.1.1 – 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.

C5.10.1.1 – Solid Walls

Brick work is generally finished by either pointing or plastering and with that in view, it is necessary to rake the joints while the mortar is green, in case of plaster work raking is intended to provide key for bonding the plaster with the background. Strictly speaking, thickness of masonry for purposes of design in these cases is the actual thickness less depth of raking. However in case of design of masonry based on permissible tensile stress (as for example, design of a free standing wall), if walls are plastered over (plaster of normal thickness i.e. 12 to 15 mm) with mortar of same grade as used in the masonry or M2 grade-whichever is stronger or if walls are flush pointed

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with mortar of M1 grade or stronger, raking thickness can be ignored.

5.10.1.2 – 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 two leaves. The outer leaf shall, therefore, be supported at least at every third storey or at every 10m 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 5.8.1.1).

C5.10.1.2 – Cavity Walls

The structural design concept for cavity walls is that both wythes contribute in resisting lateral wind or seismic loads and that one or both wythes can carry superimposed vertical loads. If only one wythe supports the superimposed axial load, then that wythe should be designed to independently resist the entire compression force.

5.10.1.3 – Faced Wall

The permissible load per length of wall shall be taken as the product of the total thickness

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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.

5.10.1.4 – 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.

5.10.2 – Walls and Columns Mainly Subjected to Lateral Loads

5.10.2.1 – 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 zone II, freestanding walls may be apportioned without making any design calculations with the help of Table 14, provided the mortar used is of grade not leaner than M1.

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C5.10.2 – Walls and Columns Mainly Subjected to Lateral Loads

C5.10.2.1 – Free-Standing Walls

1980 version of the Code provided for design of a free-standing wall as gravity structure that is, without placing reliance on the flexural moment of resistance of the wall due to tensile strength of masonry. It was seen that this approach to design resulted in fairly thick walls and maximum height of an unplastered 230 mm thick wall (one-brick thick of conventional size) could be only about 0.86 m while it has been a common practice since long to build such walls to heights much greater than 0.86 m. From a study of practices being followed in some other countries in this regard, it is evident that, for design of free-standing walls, it is appropriate to take into consideration flexural moment of resistance of masonry according to the grade of mortar used for the masonry.

The self-weight of a free standing wall reduces tensile stress in masonry caused by lateral load that is, wind pressure. Thus heavier the masonry units, lesser is the design thickness of wall for a particular height. It is, therefore, advantageous to build compound walls in stone masonry in place of brick masonry when stone is readily available and thickness has to be greater than one brick. Also it should be kept in view that use of light-weight units such as hollow bricks/ blocks in free-standing walls has obvious structural disadvantage.

As a general rule, a straight compound wall of uniform thickness is not economical except for low heights or in areas of low wind pressure. Therefore, when either height is appreciable or wind pressure is high, economy in the cost of the wall could be achieved by staggering, zigzagging or by providing diaphragm walls. It can be shown that for wind pressure of 750 N/m^2, maximum height of a 230 mm thick brick wall using grade

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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 14 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 14 reckoning the height from the lower level at which the wall is restrained laterally. 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.

Table 14: Height to thickness ratio of free-standing walls related to wind speed (Clause 6.8.2.1)

Design Wind Pressure (N/m^2)	Height To Thickness Ratio
(1)	(2)
Up to 285	10
575	7
860	5
1150	4

Note 1: For intermediate values, linear interpolation is permissible,
Note 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.
Note 3: The thickness should be measured including the thickness of the plaster.

5.10.2.2 – 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 stresses at the discretion of the designers.

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M1 mortar can be 1.5 m for a straight wall, 3.2 m for a staggered wall and 4.0 m for a diaphragm wall.

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5.10.3 – Walls and Columns Subjected to Vertical as Well as Lateral Loads

For walls and columns, stresses worked out separately for vertical loads as in 5.8.1 and lateral loads as in 5.8.2, shall be combined and elements designed on the basis of permissible stresses.

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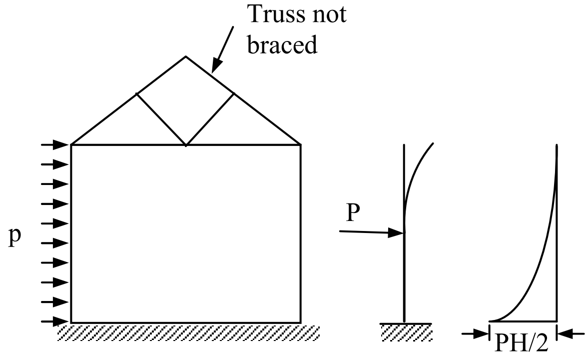
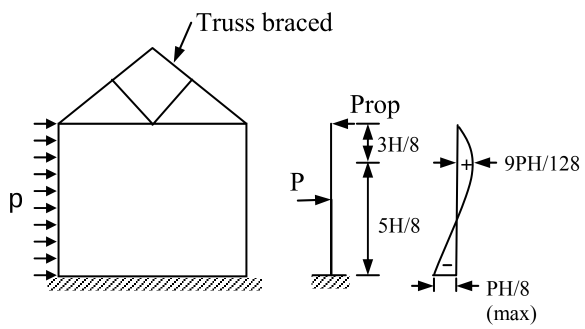
C5.10.3 – Walls and Columns Subjected to Vertical as Well as Lateral Loads

Longitudinal walls of tall single storey wide span buildings with trussed roofs such as industrial buildings, godowns, sports halls, gymnasia, etc, which do not have any intermediate cross walls other than gable walls, tend to be very thick and uneconomical if designed as solid walls, since vertical load is not much and the lateral load due to wind/earthquake predominates. This would be particularly so when the trusses are not adequately braced at the tie beam level so as to be able to act as horizontal girders for transmitting the lateral loads to the gable walls. In this case, the walls act as simple cantilevers and flexural stress at the base will be quite high. When, however, trusses are adequately braced to provide girder action and are suitably anchored to the gable walls, longitudinal walls would function as propped cantilevers, thus resulting in considerable reduction in bending moments on the long walls as shown in Figure C32.



(a) Trusses not braced



(b) Trusses braced

Figure C32: Effect of bracing of trussed roofs on buildings

Masonry diaphragm walls can be adopted in wide-span tall, single storey buildings and have been proved very economical and successful. Principle of a diaphragm wall is similar to that of a rolled

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steel I-joist that is, placing more material at places where stresses are more. As a result section modulus to area ratio of a diaphragm wall is much higher than that of a solid wall, thereby resulting in economy.

A typical arrangement for laying bricks in a diaphragm wall is shown in Figure C33. By varying the depth and spacing of ribs in terms of brick units, designer can obtain an arrangement that meets the requirement in any particular case. Placing of ribs is decided on the consideration that projecting flange length on either side of rib does not exceed 6 times the thickness of the flange. Thus rib-spacing is limited to $12\ t_f + t_r$ where t_f and t_r stand for flange and rib thickness respectively. Brick layout in diaphragm wall is planned such that proper masonry bond is obtained with the least number of cut bricks.

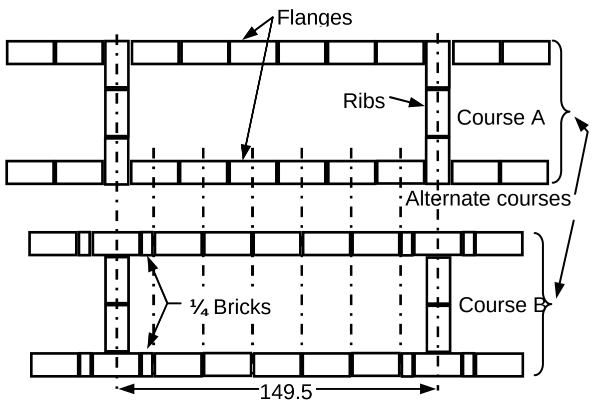

Figure C33: Typical brick laying arrangement for diaphragm walls

5.10.4 – Walls Subjected to In-Plane Bending and Vertical Loads (Shear Walls)

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

C5.10.4 – Walls Subjected to In-Plane Bending and Vertical Loads (Shear Walls)

A cross wall which functions as a stiffening wall to an external load-bearing wall is subjected to in-plane bending. If it is also supporting a floor/roof load, it is subjected to vertical load in addition to in-plane bending. It should be kept in view that such a wall when subjected to vertical load gets strengthened since vertical load reduces or nullifies tension due to bending and also increases the value of permissible shear stress (see also comments on 5.7.4).

5.10.5 – 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

C5.10.5 – Non Load Bearing Walls

Non-load bearing panel and curtain walls if not designed on the basis of guidelines given in Appendix D of the Code may be apportioned with

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present state of the 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 Appendix D.

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the help of Table C-5 which is extracted from Recommended Practices for Engineered Brick Masonry. The table is based on the assumption that wall is simply supported only in one direction either vertically or horizontally without any opening or other interruptions. Where the wall is supported in both directions, the allowable distance between lateral supports may be increased such that the sum of the horizontal and vertical spans between supports does not exceed three times the permissible distance permitted for supporting in the vertical direction.

Guidelines given in Appendix D of the Code are based on some research in which mainly rectangular panels without openings were tested. If openings are small that is hole-in-wall type (see C-4.1 Note), there would be no appreciable effect on strength of panels, since timber or metal frames that are built into the openings compensate to a great extent for the loss of strength of the panel due to the openings. However, when the openings are large or when the openings cannot be categorized as of 'hole-in-wall' type, it may often be possible to design the panel by dividing it into sub-panels as shown in Figure C34.
In situations where design by forming sub-panels is not feasible, panel may be analyzed using theory of flat plates (for example, yield line theory or finite element method) taking into consideration end conditions as appropriate.

Table C5: Span to Thickness Ratio of non-load bearing Panel / Curtain walls

Design wind pressure N/m^2	Vertical span		Horizontal span	
	Cement-lime mortar 1:1:6	Cement-lime mortar 1:½:4½	Cement-lime mortar 1:1:6	Cement-lime mortar 1:½:4½
(1)	(2)	(3)	(4)	(5)
2.5	38	43	54	61
5.0	27	30	38	43
7.5	22	25	31	35
10.0	19	21	27	30
12.5	17	19	24	27
15.0	15	17	22	25

NOTE: Partition walls which are not subjected to any wind pressure that is, internal partition walls may be apportioned with the help of the above Table by assuming a minimum design wind pressure of 250 N/m^2.

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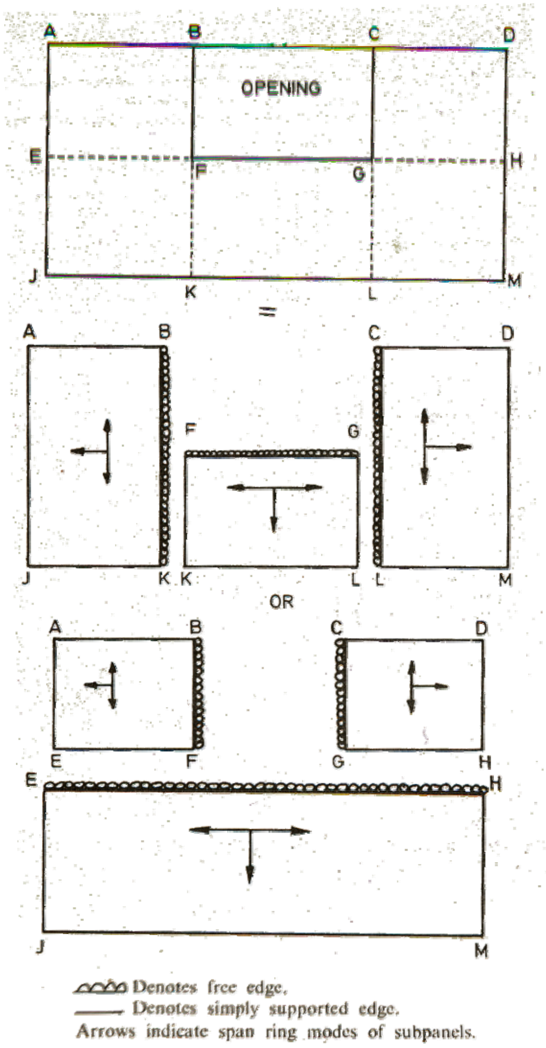

Figure C34: Design of panel having large opening

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6. – GENERAL REQUIREMENTS

6.1 – Methods of Construction

6.1.1 – General

Brickwork	IS : 2212-1962*
Stone masonry	IS : 1597 (Part 1)-1967 IS : 1597 (Part 2)-1967
Hollow concrete block masonry	IS : 2572-1963
Autoclaved cellular concrete block masonry	IS : 6041-1985
Lightweight concrete block masonry	IS : 6042-19697
Gypsum partition blocks	IS : 2849-1983**

The methods adopted in the construction of load bearing and non-load bearing shall comply with the following standards:
* Code of practice for brickwork.
T Code of practice for construction of stone masonry : Part 1 Rubble stone masonry.
$ Code of practice for construction of stone masonry : Part 2 Ashlar masonry.
§ Code of practice for construction of hollow concrete block masonry.
L Code of practice for construction of autoclaved cellular concrete block masonry (first revision).
B Code of practice for construction of lightweight concrete block masonry.
** Specification for non-load bearing gypsum partition blocks (solid and hollow types) (first revision).

6.1.2 – Construction of Buildings in Seismic Zones

~~No special provisions on construction are necessary for buildings constructed in zones I and II. Special features of construction for earthquake resistant masonry buildings in zones III, IV and V shall be applicable as given in IS: 4326-1993tt.~~
~~tt Code of practice for earthquake resistant construction of buildings (first *revision*).~~
Unreinforced masonry buildings shall be designed in accordance with IS 1893 (Part 1) - 2002 and relevant~~-~~provisions of this code. Alternatively, certain residential buildings upto three storey and other buildings not housing essential services may be designed as per the requirements of IS: 4326-1993.

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C6. – GENERAL REQUIREMENTS

C6~~7~~.1.2 – Construction of Buildings in Seismic Zones

Unreinforced shear walls with no reinforcement is used anywhere will in general have poor post-elastic response and therefore can be only used for resisting small earthquake forces only. It should be used only in low seismic regions and for buildings of minor importance and consequence.

Seismic design provisions contained in IS: 4326 are empirical in nature which are based on successful applications in the past and do not require a rational analysis. These can be used for buildings of small scale nature and are based on the premise that they have enough shear walls in the two directions and are properly connected at corners and integrated with roof and floor

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diaphragms to create a box-like system for lateral loads.
Preferably buildings located in high seismic regions IV and V shall be designed for forces in IS: 1893 and provisions of reinforced masonry as per IITK-GSDMA Guidelines (available at *www.nicee.org*)

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 Indian Standards, and thickness suitably increased, where found necessary

C6.2 – Minimum Thickness of Walls from Consideration other than Structural

i) Requirements for thickness of walls from considerations other than strength and stability have been discussed below with regard to fire resistance, thermal insulation, sound insulation and resistance to rain penetration.

ii) *Resistance to Fire*- The subject of fire resistance of buildings has been dealt with comprehensively in appropriate Indian Standards and also in Part IV of the National Building Code of India 1983 which may be referred to in this regard.

iii)*Thermal Insulation* -Thickness of walls in case of non-industrial buildings from consideration of thermal insulation should be worked out for the climatic conditions of the place where a building is to be constructed on the basis of IS 3792: 1978. Even though no Indian Standard has yet been published on the subject for industrial buildings, data and information given in the above Indian Standard would be of some assistance in deciding the thickness of walls from consideration of thermal insulation.

iv) *Sound Insulation of Value of Wall*

a) Indian Standard IS 1950: 1962 lays down sound insulation standards of walls for non-industrial buildings such as dwellings, schools, hospitals and office buildings. Salient features of that standard are summarised below for ready information.

b) While deciding thickness/specifications of walls, it is necessary to consider, firstly the level of ambient noise in the locality where building is to be constructed depending upon intensity of traffic and type of occupancy of the building. Noise level of traffic varies from 70 decibels (abbreviated as dB) for light traffic to 90 dB for heavy traffic. Requirements of sound insulation for different buildings from consideration of ambient noise level and occupancy are given in Table C-6. These values are applicable to

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external walls for reducing outdoor air-borne noise.

Table C-6 Requirements of sound insulation values (dB) of external walls of buildings against air-borne noise

Clause C7.2 (iv)(b)

Sl No.	Type of building	For noisy locations (90 dB Level)	For quiet locations (70 dB Level)
1	Dwellings	45	25
2	Schools	45	25
3	Hospitals	50	30
4	Offices	40	20

c) Sound insulation values of partition and internal walls are decided on considerations of levels of indoor noise emanating from adjacent buildings or adjacent rooms and these should be as given in Table C-7.

Table C-7 Sound Insulation Values for party and internal walls

Sl No.	Situation	Sound insulation values
1	Between living /bed room in one house or flat and living /bed rooms in another	50
2	Elsewhere between houses or flats	25
3	Between one room and another in the same house or flat	30
4	Between teaching room in a school	20
5	Between one room and another in office	30
6	Between one ward and another in a hospital:	
	Normal	40
	Extra quiet	45

d) Sound insulation values of non-porous homogeneous rigid constructions, such as a well plastered brick/ stone masonry or concrete wall, vary as the logarithm of weight per unit area and thus increase with the thickness of wall. These values are given in Table C-8.

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Table C-8 Sound Insulation Values of Solid constructions

Weight per m^2 of wall area (kg)	Sound insulation value (dB)
5	22.8
25	33.2
50	37.6
100	42.0
150	44.7
200	46.4
250	47.9
300	49.1
350	50.0
400	50.9
450	51.6
500	52.3
600	53.6

e) Based on the data given in Table C-8, insulation values of brick walls plastered on both sides work out as in Table C-9.

Table C-9 Sound Insulation Values of Masonry walls plastered on both sides

Thickness of wall (cm)	dB
7.7	45.7
10	47.3
11.5	48.0
20	51.3
23	52.2

f) As a general guide, it may be taken that for noise insulation a one-brick wall (20 or 23 cm thick/plastered on both sides as external wall and a ½ brick wall (10 or 11.5 cm thick) plastered on both sides as internal walls are adequate.

v) *Resistance to Rain Penetration* - Recommendations for thickness of walls of different types of masonry from consideration of resistance to rain penetration based generally on IS 2212: 1962 are given in Table C-10.

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Table C-10 Suitability of Walls for Different Exposures (R-Recommended and NR- Not Recommended)

Sl No.	Particulars of wall	Type of Exposure		
		Sheltered	Moderate	Severe
1	Brick mas onry-burnt clay or sand-Iime			
	a)1 brick wall – not plastered	R	NR	NR
	b)1 brick wall – plastered both sides	R	R	NR
	c)1½ brick wall – not plastered	R	R	NR
	d) 1½ brick wall – plastered both sides	R	R	R
2	Stone masonry			
	a) Minimum thickness 35cm – not plastered	R	R	NR
	b)Minimum thickness 35cm – plastered both sides	R	R	R
3	Concrete block masonry 20cm minimum thickness			
	a)Not plastered	R	NR	NR
	b)Plastered on both sides	R	R	NR
4	Stone blocks – 20 cm minimum thickness			
	a)Not plastered	R	NR	NR
	b)Plastered both sides	R	R	NR
5	Cavity wall of 25 cm minimum thickness	R	R	R

NOTES:
1 Use of cement-lime or lime mortar in place of cement mortar appreciably improves the resistance of a wall to rain. It is also important that joints in masonry are fully filled with mortar.
2 Sheltered conditions' are those where wall is protected by overhangs or adjoining buildings or rainfall is low (less than 750 mm per year and is generally not accompanied by strong winds. 'Severe conditions' occur when wall is subjected to strong winds and persistent rain and there is no sheltering action of overhangs or adjoining buildings, or rain fall is heavy (exceeding 1000 mm). 'Moderate condition' obtains when exposure conditions are between 'Sheltered' and 'Severe' conditions.

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 10 would hold good for commercially obtainable standards of

C6.3 – Workmanship

C6.3.1 – General

Workmanship has significant effect on strength and development of bond. Common defects of workmanship in masonry are:

a) Improper mixing of mortar;
b) Minimum time lapse between spreading of mortar and placing of masonry unit;
c) Incorrect adjustment of suction rate of bricks;

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workmanship with reasonable degree of supervision. If the work is inadequately supervised, strength should be reduced to three-fourths

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d) Unduly thick bed joints;
e) Uneven or deeply furrowed bed joints;
f) Voids in perpend (head) joints; and
g) Disturbance of bricks after laying.

The time lapse between spreading of mortar and placing of unit should not normally exceed one minute and preferably kept lower in hot, dry and windy conditions. Otherwise, mortars ability to flow gets diminished through suction on the unit it is placed on thus resulting in poor bond characteristics.

Retempering of mortars to restore water lost by evaporation is allowed and even encouraged to maintain its original consistency, as long as it is done within 2.5 hours after the original mixing.

Suction rate of bricks has a very pronounced effect on the strength of brick-work and especially on the bond therefore it should be controlled carefully. Water absorbed from mortar by bricks leaves cavities in the mortar, which get filled with air and thereby reduce the strength of mortar. Brick work built with saturated bricks develop poor adherence between brick and mortar. Thus flexural strength as well as shear strength of such brickwork would be low. At the same time such brickwork will be prone to excessive cracking due to high shrinkage and thus rain-resisting qualities of the brickwork will be poor. British Ceramic Association have suggested a suction rate of 2 kg/min/m^2, while in accordance with Canadian Code and American Practice adjustment in suction rate is required, if initial absorption rate exceeds 1.5 kg/min/m^2.
The Commentary on Australian Code specifies that suction of bricks should be between 1.0 to 3.0 kg/min/m^2. Moderate initial rates of absorption of 0.25 to 1.5 kg/min/m^2 at the time of laying generally produces good bond. Optimum suction rate depends on atmospheric conditions, namely, temperature and humidity as well as certain properties of mortar used in masonry.

Strength of masonry gets reduced as the thickness of bed joints increases. Taking normal thickness of bed joints as 10 mm, an increase of 3 mm in thickness of bed joints may reduce the strength of brick masonry by 15 percent and vice versa.

Experiments conducted in other countries indicate that uneven or deeply furrowed joints can reduce strength of brickwork up to about 33 percent. Thus, this is rather a serious defect in masonry construction. Some masons have the habit of making a furrow in the mortar of the bed joint in the middle parallel to the face before laying a course of bricks, so as to lessen squeezing out of

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mortar from the bed joints on pressing into position. This practice should be avoided.

Inadequately filled vertical joints substantially lower the rain resisting property of walls. Disturbance of bricks after laying affect the bond strength as well as shear strength of brickwork and therefore should be avoided. If adjustment in position of bricks after laying becomes necessary bricks as well as mortar should be completely removed and brickwork redone with fresh mortar.

6.3.2 – Bedding of Masonry Units

Masonry units shall be laid on a full bed or 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 relaid 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 conforming to the requirements of IS : 2212-1962*.

*Code of practice for brickwork.

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 offset between load bearing walls in adjacent storey 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

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perpends shall not exceed one-fifth of the specified thickness.

NOTE - These tolerances have been specified from point of view of their effect on the strength of masonry. The permissible stresses recommended in section 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 of IS: 3414- 1968*.

*Code of practice for design and installation of joints in buildings.

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 are not shown in drawings, these may be provided subject to the constraints and precautions specified in 6.5.3 to 6.5.13.

6.5.4 –

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

6.5.5 –

For load bearing walls, depth of vertical and horizontal chases shall not exceed one-third

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and one-sixth of the wall thickness respectively.

6.5.6 –

Vertical chases shall not be closer than 2 m in any stretch of wall and shall not be located within 34.5 cm of an opening or within 23 cm 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.

6.5.7 –

When unavoidable horizontal chases of width not exceeding 6 cm 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 60 cm from a lateral support. No horizontal chase shall exceed one meter in length and there shall not be more than 2 chases in any one wall. Horizontal chases shall have minimum mutual separation distance of 50 cm. Sum of lengths of all chases and recesses in any horizontal plane shall not exceed one-fourth the length of the wall.

6.5.8 –

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.

6.5.9 –

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.

6.5.10 –

Masonry directly above a recess or a hole, if wider than 30 cm, shall be supported on a lintel. No lintel, however, is necessary in case of a circular recess or a hole exceeding 30 cm in diameter provided upper half of the recess or hole is built as a semi-circular arch of adequate thickness and there is adequate length of masonry on the sides of openings to resist the horizontal thrust.

6.5.11 –

As far as possible, chases, recesses and holes in masonry should be left (inserting

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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.

6.5.12 –

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

6.5.13 –

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.

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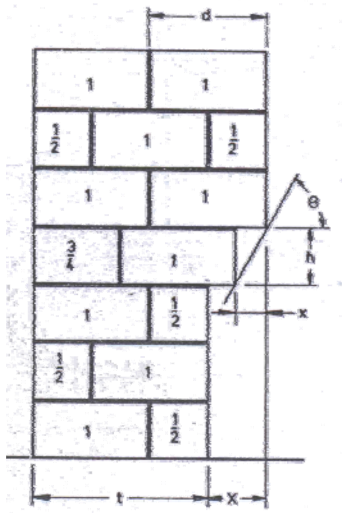
C6.6 – Corbelling

C6.6.1 –

Limitations of a corbel have been illustrated in Fig. C-36. With these limitations, minimum slope of corbelling (angle measured from the horizontal to the face of the corbelled surface) would work out to 63°, when using modular bricks with header courses in the corbelled portion.



$$x \le \frac{h}{2}$$

$$x \le \frac{t}{3}$$

$$x \le \frac{d}{3}$$

$$\theta = \tan^{-1} h / x$$

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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 6.6.

6.6.3 –

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

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x = allowable projection of one unit.
X = total allowable horizontal projection of corbel.
t = nominal wall thickness (actual-plus thickness thickness of one joint)
h = nominal unit height (actual height plus thickness of one joint).
d = nominal bed depth of unit (actual bed depth of unit plus thickness of joint).
θ = slope of corbel (angle measured from the horizontal to the face of the corbelled surface).

Figure C-36 Limitations of a corbel in masonry

Load on a corbel has very high eccentricity. It is, therefore, necessary to exercise great caution in the use of corbelling in buildings since eccentricity in loads appreciably reduces the permissible compressive stress in masonry. As it is not feasible to make precise calculations of actual stress in the corbelled portion of masonry, the Code provides for some empirical rules to limit the stress to within safe limits.

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7. – NOTATIONS AND SYMBOLS

7.1 –

The various notations and letter symbols used in the text of the standard shall have the meaning as given in Appendix E.

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C7. – NOTATIONS AND SYMBOLS

Appendix A

(*Clause 4.8)*

SOME GUIDELINES FOR ASSESSMENT OF ECCENTRICITY OF LOADING ON WALLS

A1. –

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, in extent of one-sixth the bearing width.

A2. –

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.

A3. –

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.

A4. –

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 there from.

A5. –

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.

A6. –

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.

A7. –

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.

Appendix B

(Clause 6.6.1)

CALCULATION OF BASIC COMPRESSIVE STRESS OF MASONRY BY PRISM TEST

B1. – Determination Of Compressive Strength of Masonry By Prism Test

B1.1 – Testing in advance of construction

A set of five masonry prisms shall be 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 40 cm 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 12.

Table 15: Correction Factors for Different *h/t* Ratios
(Clause B-1.1)

Ratio of height to thickness (*h/t*)	2.0	2.5	3.0	3.5	4.0	5.0
Correction factors for brickworks*	0.73	0.8	0.86	0.91	0.95	1.0
Correction factors for blockworks*	1.0	-	1.20	-	1.30	1.37

*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 to 700 kN/m. The load at failure should be recorded.

B1.2 – Testing during construction

When full allowable stresses are used in design, a set of three prisms shall be built and tested during construction in accordance with section B1.1 for each 500 square meters of wall area, but not less than one set of three masonry prisms for any project. No testing during construction shall be required when three-fourths of the allowable stresses are used in design.

Appendix C

(Clauses 5.3.3 and 5.6.2.5)

GUIDELINES FOR DESIGN OF MASONRY SUBJECTED TO CONCENTRATED LOADS

C1. – Extent of Dispersal of Concentrated Load

C1.1 –

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.

C2. – Increase In Permissible Stress

C2.1 –

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 by 50 percent, provided the area of supporting wall is not less than three times the bearing area.

C2.2 –

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

C2.3 –

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.**

C2.4 –

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.

C3. – Criteria of providing bed block

C3.1 –

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 M15 Grade) capable of taking tension.

C3.2 –

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 M-15 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.

C3.3 –

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 M-15 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.

Appendix D

(Clause 5.8.5)

GUIDELINES FOR APPROXIMATE DESIGN OF NON-LOAD BEARING WALL

D1. – Panel Walls

D1.1 –

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 centers 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 would depend upon height to length ratio of panel and flexural strength of masonry. Approximate values or 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 16.

TABLE 16: Bending moments in laterally loaded panel walls, free at top edge and supported on other three edges

H/L	0.30	0.50	0.75	1.00	1.25	1.50	1.75
Bending moment	PL/25	PL/18	PL/14	PL/12	PL/11	PL/10.5	PL/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 17.

TABLE 17: Bending moments in laterally loaded panel walls supported on all four edges

H/L	0.30	0.50	0.75	1.00	1.25	1.50	1.75
Bending moment	PL/72	PL/36	PL/24	PL/18	PL/15	PL/13	PL/12

D2. – Curtain Walls

D2.1 –

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

D3. – Partition walls

D3.1 –

These are internal walls usually subjected to much smaller lateral forces. Behavior 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 and 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 restraint 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.

D3.2 –

Strength of bricks used in partition walls should not be less than 3.5 MPa or the strength of masonry units used in adjoining masonry, whichever is less. Grade of mortar should not be leaner than M2.

APPENDIX E

(Clause 8.1)

NOTATIONS, SYMBOLS AND ABBREVIATIONS

E-1. The following notations, letter symbols and abbreviations shall have the meaning indicated against each, unless otherwise specified in the text of the standard:

A	Area of a section
A_n	Net area
A_{st}	Area of steel
$A_{v,min}$	Minimum area of shear reinforcement
b	Width of bearing
d	Effective depth
d_b	Nominal diameter of bar (mm)
DPC	Damp proof course
e	Resultant eccentricity
E_m	Elastic modulus of clay and concrete masonry
E_s	Elastic modulus of steel reinforcement
f_A	Calculated axial compressive stress
f_B	Calculated bending stress
f_b	Basic compressive stress
f_d	Compressive stress due to dead loads
f_m	Compressive strength of masonry (in prism test)
F_a	Allowable axial compressive stress
F_b	Allowable bending compressive stress
F_s	Permissible tensile/compressive stress in steel (MPa)
F_v	Permissible shear stress
GL	Ground level
H	Actual height between lateral supports
H'	Height of opening
H1, H2	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_d	Development length
L1, L2	Lower strength mortars
M1, M2	Medium strength mortars
P	Total horizontal load
P_o	Permissible compressive force for Reinforced Masonry
PL	Plinth level
RCC	Reinforced cement concrete
RS	Rolled steel
s	Spacing of shear reinforcement
S_p	Spacing of piers/buttresses/cross walls
SR	Slenderness ratio
t	Actual thickness
t_p	Thickness of pier
t_w	Thickness of wall
V	Total applied shear force
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.

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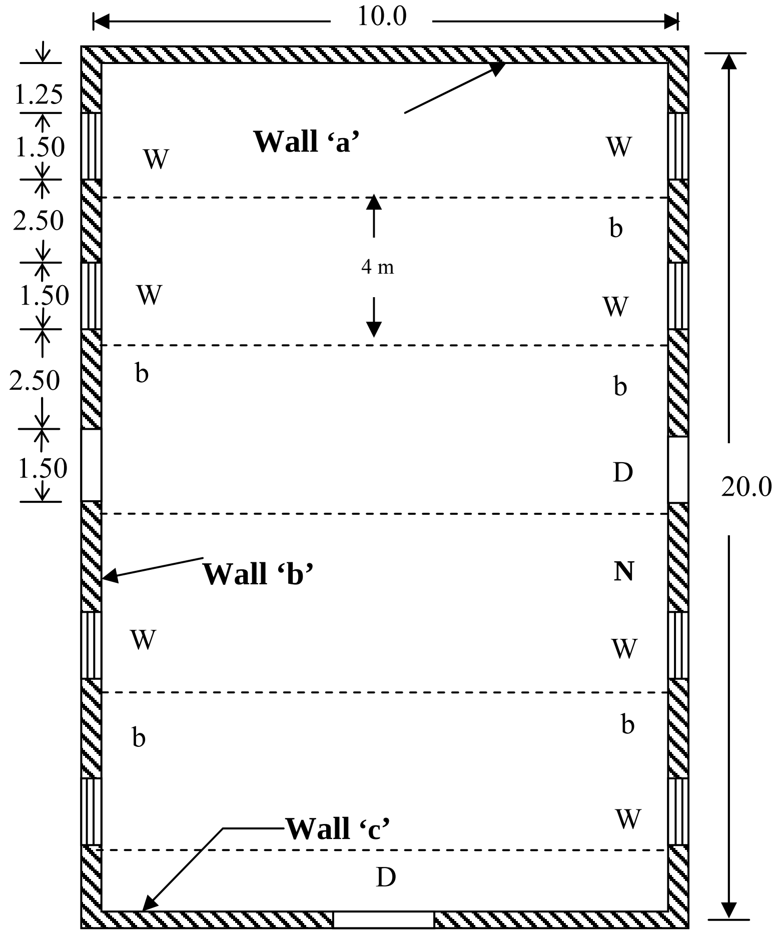
PART 2: EXPLANATORY EXAMPLES

Example 1 — DESIGN OF A HALL SUBJECTED TO WIND LOAD

Problem Statement:

A hall as shown in Figure 1.1 and of inside dimensions 10.0 m × 20.0 m with a clear height of 5.5 m up to the bottom of beam is to be constructed with load bearing masonry walls using modular bricks. Calculate thickness of walls, strength of bricks and grade of mortar for longitudinal and cross walls, assuming a wind pressure of 1200 N/m^2.



(All Dimensions are in meter)

Figure 1.1: Plan of Hall

Solution:

Design Data/Assumptions:

Roof consists of RCC T-beams 400 mm×800 mm with RCC slab 120 mm thick, beams being at 4.0 m centers. Roof covered with lime concrete terrace of 150 mm average thickness.

Modular bricks are used of the nominal size of 20 cm × 10 cm × 10 cm

Height of parapet h_p = 200 mm above slab level

Plinth height h_{ptf} = 0.5 m

Height of plinth above foundation footing = 0.7 m

Clear height of hall, h=5.5 m

c/c spacing of beams, s_b=4.0 m

Wind pressure, f_w = 1200 N/m^2

Hall Dimension Length, L = 20 m

Width, B = 10 m

Size of T-beam: Width, b = 400 mm

Depth, d = 800 mm

Unit wt. of concrete, $\gamma_c = 25$ kN/m^3

Unit wt. of masonry, $\gamma_m = 20$ kN/m^3

Minimum Thickness of Wall:

According to Section 4.7.1.1 of Draft Code IS: 1905, maximum slenderness ratio for cement or cement-lime mortar, $SR_{max} = 27$

Height of long wall

$H_{lw} = 0.7+5.5+0.8/2 = 6.6$ m

(from top of foundation to center of T-beam)

Minimum thickness of long wall required,

$=0.75H_{lw}/SR_{max}$

$= 0.75\times6.6/27 = 0.183$ m

Adopt 1.5 brick wall with raked joint up to depth of 10 mm. Nominal thickness of long wall, $t_{lw} = 0.3$ m

Height of cross wall (from top of foundation to center of slab),

$H_{cw} = 0.7+5.5+(0.8-0.06)/2 = 6.94$ m

Minimum thickness of cross wall required,

$= 0.75H_{cw}/SR_{max}$

$= 0.75\times6.94/27 = 0.193$ m

Adopt 1.5 brick wall with raked joint up to depth of 10 mm. Nominal thickness of cross wall, $t_{cw} = 0.3$ m

Calculation of Loads:

Roof Load

RCC slab, $P_{sl}= 0.12\times25000 = 3 \times 10^3$ N/m^2

Terrace, $P_{lt}= 0.15\times20000 = 3 \times 10^3$ N/m^2

Live Load, $P_l = 1500$ N/m^2

Total roof load,

$P_r = 3000+3000+1500 = 7500$ N/m^2

Effective span of beam, $l_{eff} = 10.3$ m

Self-weight of beam,

$P_{bw} = b \times (d-t_s) \times \gamma_c$

$=0.4\times(0.8-0.12)\times25000 = 6800$ N/m

Loads on Long Wall

Load from beam,

$P_l= (7500\times4+680)\times10.3/2 = 190$ kN

Self-load of wall including parapet assuming 30 mm plaster thickness. Since we will be considering combined stresses due to vertical loads and wind load, we will work out all loads at the top of foundation footing.

Self weight of wall,

$P_{sw} = (0.29+0.03)\times(0.7+5.5+0.8+0.2)\times20000$

$= 46.1$ kN/m

Loads on Cross wall

Load from slab, = 7500×4/2= 15000 N/m

Total load on wall at plinth level,

$P_{cw} =15+46.1= 61.1$ kN/m

Calculation of Stress in Wall Due to Vertical Loads:

Long Wall

Length of openings, $l_{op} = 1.5$ m

Stress at footing-top level due to self-weight,

$$f_{lw} = \frac{46.1\times10^3\times4}{2.5\times290\times1000} = 0.25 \text{ MPa}$$

Length of wall supporting concentrated load from beam is given by:

$l_{bc} = b + 4\times t = 0.40 + 4\times0.29$

$= 1.56$ m length of wall

Stress due to concentrated load,

$$f_{lw} = \frac{190000}{1.56\times290\times1000} = 0.42 \text{ MPa}$$

Therefore, total axial stress at plinth level,

$f_{la} = f_{lw} + f_{lc} = 0.25+0.42 = 0.67$ MPa

Cross wall without Opening

Compressive stress at plinth level of wall 'a' is given by

$$f_{caa} = \frac{61.1\times10^3}{290\times1000} = 0.22 \text{ MPa}$$

Cross wall with Opening

Compressive stress at plinth level of wall 'b' is given by

$$f_{cab} = \frac{61.1 \times 10^3}{290 \times 1000}\left[\frac{10}{10-1.5}\right] = 0.25\,\text{MPa}$$

Calculation of Stress in Wall due to Lateral Loads and Combined Stresses:

Long Walls

Since long walls are not adequately stiffened in accordance with the requirements of clause 4.2.2.2(b) of draft code IS:1905, it is necessary to work out bending stresses due to wind load in longitudinal as well as cross wall. Obviously wind load normal to the long walls will be critical and therefore we will work out bending stresses in long as well as cross walls on account of wind load normal to the long walls.

Wind load on long wall per bay is given by

$P_b \;= f_w \times (h + d + h_p) \times s_b$

$\quad = 1.2 \times (5.5+0.8+0.2) \times 4$

$\quad = 31.2$ kN

Note: Wind load on exposed portion of wall below plinth has been ignored

Total wind load on long wall:

$P \;= P_b \times L/s_b = 31.2\times5 = 156$ kN

It can be assumed that the lateral support from RCC beams and slabs will be adequate as a horizontal girder to transmit the wind force to the cross walls. The long wall will thus function as propped cantilever and the maximum bending moment will be at bottom support as shown in Fig. 1.2

Maximum B.M. on long wall per bay,

$$M_{l\max} = \frac{P_b}{8} \times \left(h + \frac{d}{2} + h_{plf}\right)$$

$\quad = 31.2\times(5.5+0.4+0.7)/8$

$\quad = 25.74$ kNm

Section modulus of long wall,

$$Z_l = \frac{4 \times 0.29^2}{6} = 0.0561\,\text{m}^3$$

Bending stress in long wall is given by

$$f_{lb} = \frac{M_{l\max}}{Z_l} = \frac{25.74 \times 10^6}{0.0561 \times 10^9} = 0.46\,\text{MPa}$$

Combined stresses in long wall:

$f_{l1} \;= f_{la} + f_{lb} \;= 0.67+0.46 = 1.13$ MPa

$f_{l2} = f_{la} - f_{lb} = 0.67-0.46 = 0.21$ MPa

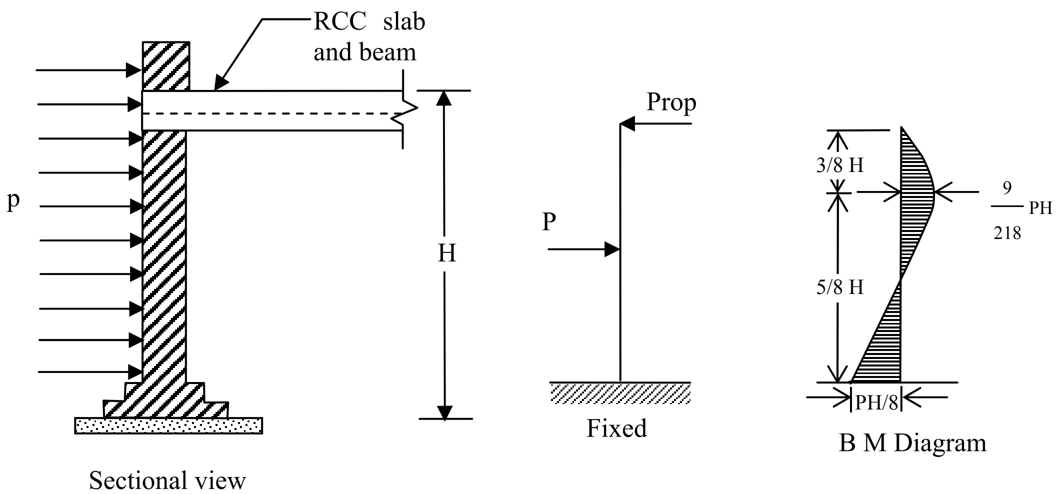


Figure 1.2: Bending moment diagram of wall

Cross walls:

Wind forces are shared by the cross walls in the ratios of their stiffness. In this case, it is assumed that the walls are identical and the loads shared equally by two cross walls. Total wind load on a cross wall may be assumed to be acting at mid-height. Thus total B.M. on one cross wall may be given by,

M_c= 0.25×1560000×(0.7+5.5+0.2+0.8)

= 280.8 kNm

A part of long wall will act as flange with cross wall and the effective overhanging length of flange is the least of the following. (as per section 4.2.2.5, Draft Code IS: 1905)

a) Actual length of wall up to window = 1.25 m

b) 6t : 6 × 0.29 = 1.74 m

c) H/16 : (0.7+5.5+0.8+0.2)/16 = 0.45 m *(controls)*

Moment of inertia of cross wall

Moment of inertia about neutral axis is given by:

$I_c = I_0 + I_1$

where, $I_0 = \dfrac{0.29 \times 10^3}{12} = 24 \text{ m}^4$

$$I_1 = \frac{2 \times (0.45 + 0.29) \times 0.29 \times 5.15^2}{12} = 0.9 \text{ m}^4$$

I_c = 24+0.9 = 24.9 m^4

Check for Combined Stress:

y_{max} = 5+0.29 = 5.29 m

Bending stresses at extreme fibers,

$$f_{cb} = \frac{M_c \times y_{\max}}{I_c} = \frac{280.8 \times 5.29}{24.9} = 0.06 \text{ MPa}$$

Combined stresses in cross walls

= axial stress + bending stress

In case of cross wall 'a' combined stresses are:

$f_{ca1} = f_{caa} + f_{cb}$ = 0.22+0.06 = 0.28 MPa

$f_{ca2} = f_{caa} - f_{cb}$ = 0.22-0.06 = 0.16 MPa

(both compressive)

In case of cross wall 'b' combined stresses are:

$f_{cb1} = f_{cab} + f_{cb}$ = 0.25+0.06 = 0.31 MPa

$f_{cb2} = f_{cab} - f_{cb}$ = 0.25-0.06 = 0.19 MPa

(both compressive)

Check for Shear Stress in Cross wall:

We will consider wall 'b' which will have greater shear stress.

Shear load on the cross wall,

$V = P/2$ = 156/2 = 78 kN

In the view of 33% increase in the allowable stress level due to wind/earthquake load, we will reduce the combined load to 75% and use 100% of the permissible stress value.

Section of wall being rectangular, we will assume parabolic shear distribution and maximum shear stress will be 1.5 times the average shear. Since flanges do not make any contribution for resisting shear load, maximum shear stress on wall

$$f_v = \frac{1.5 \times 0.75 \times 75 \times 10^3}{0.29 \times (10 - 1.5) \times 10^6} = 0.04 \text{ MPa}$$

Compressive stress due to dead loads (i.e. due to self weight and load from slab)

$$f_d = \frac{0.75 \times \left(61.1 \times 10^3 + 6 \times 10^3 \times 2\right)}{0.29 \times 1000} = 0.19 \text{ MPa}$$

Permissible shear stress (F_v) is the least of the following:

i) 0.5 MPa

ii) 0.1 + 0.2f_d = 0.14 MPa

iii) 0.125 $\sqrt{f_m}$= 0.395MPa

(Assuming crushing strength of masonry as 10 MPa)

Hence, the permissible shear stress is 0.14 MPa. Actual stress being only 0.04 MPa, the wall is safe in shear. Thus both cross walls are safe in shear and tension. Use M2 grade mortar.

Masonry for walls:

Long Wall

Masonry of cross wall should be designed for maximum compressive stress i.e., 1.21 MPa

Slenderness ratio is given by:

$$\text{SR} = \frac{0.75 \times (0.7 + 5.5 + 0.4)}{0.26} = 19$$

As per Table 11, Draft code IS: 1905, Stress

Reduction Factor, $k_s = 0.65$

Shape modification factor is taken as unity. Required basic compressive stress is given as:

$f_b = f_{l1} / k_s = 1.13/0.65 = 1.74$ MPa

Referring to Table 10 of draft code IS: 1905, bricks should be of strength 25 MPa and mortar should be of grade H1. If brick of this strength are not available it would be necessary to introduce piers under the beams so as to increase the supporting area thereby reducing stress in masonry.

Cross walls

Masonry of cross wall should be designed for maximum compressive stress that is, 0.34 N/mm^2

Slenderness ratio is given by:

$$= \frac{0.75 \times (0.7 + 5.5 + 0.8 - 0.06)}{0.26} = 20$$

Stress Reduction Factor, $k_s = 0.62$

(Table 11, Draft Code IS: 1905)

Basic compressive stress for unit shape modification factor = 0.31/0.62 = 0.5 MPa

Referring to table 10 and 12 of draft code IS: 1905, the bricks should be of strength 7.5 MPa and with shape modification factor equal to 1.1.

Thus the basic compressive stress required is given by

$f_b = f_{cb1} / k_s = 0.5/1.1 = 0.45$ MPa

Grade of mortar should be of grade M3. However from the consideration of shear stress, M2 mortar should be used.

Example 2 – DESIGN OF A DIAPHRAGM TYPE FREE STANDING WALL

Problem Statement:

A brick masonry wall (see Fig. 2.1) is built in mortar of grade M1. Find the maximum safe height for this wall, when it is subjected to a wind velocity of 47 m/s and is located in a built up urban area. Bricks used are of nominal size 23 × 11.5 × 7.7 cm.(inclusive of 10 mm mortar joint).

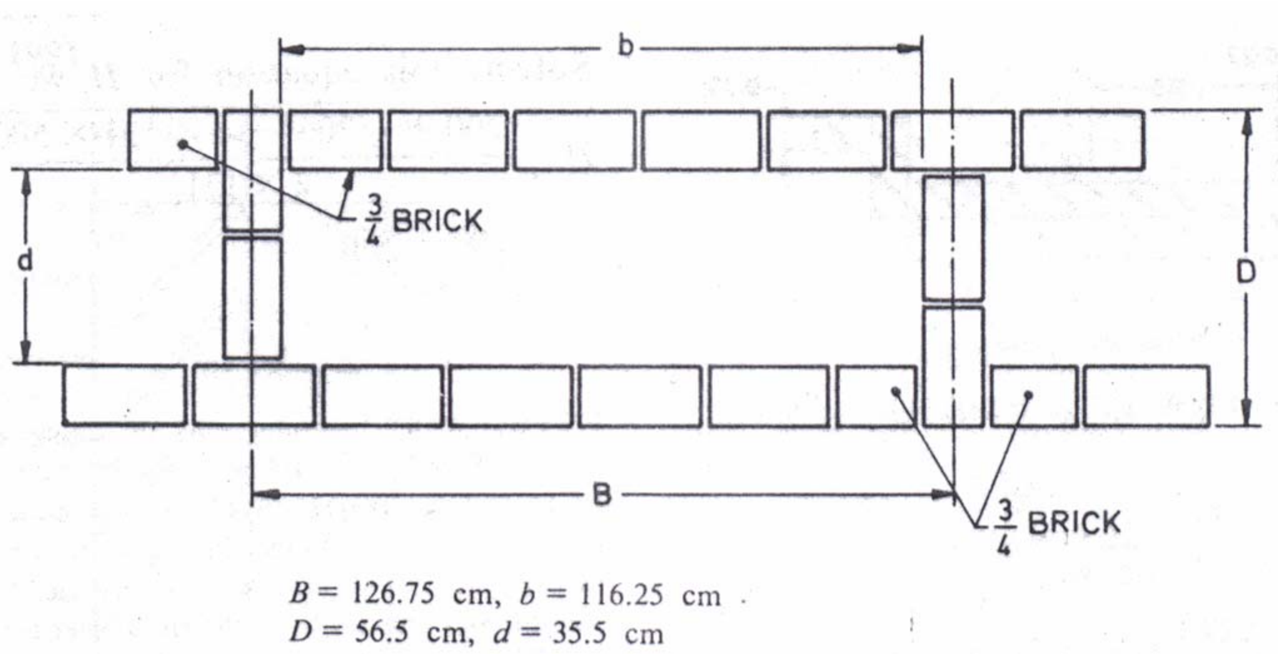


Figure 2.1: Plan View of a diaphragm wall

Solutions:

Design Data/ Assumptions:

Grade of Mortar = M1

Actual Size of Bricks, $l_b = 0.22$ m,

$b_b = 0.105$ m,

$t_b = 0.077$ m

Wind Velocity, $V_b = 47$ m/s

Risk Coefficient factor, $k_1 = 0.73$
(for boundary wall)

Terrain & Height factor, $k_2 = 0.91$

Topography factor, $k_3 = 1.00$

Permissible tension in masonry with M1 mortar,

$f_t = 70000$ N/m^2

Unit weight of masonry, $w = 20000$ N/m^3

Calculation of Wind Pressure:

According to IS: 875 (Part 3): 1987,

Design wind speed, $V_z = k_1 \times k_2 \times V_b$

$= 0.73 \times 0.91 \times 47$

$= 31.2$ m/s

Wind pressure, $p_z = 0.6V_z^2$

$= 0.6(31.2)^2$

$= 584.9$ N/m^2

Calculation of Moment of Inertia:

Consider the diaphragm unit of length B and height H

Length of diaphragm unit,

$$B = 5.25 \times 22 + \frac{10.5}{2} + 6$$

$$B = 1.27 \text{ m}$$

Overall width of unit, $D = 22 \times 2 + 10.5 + 2$

$D = 0.565$ m

Internal length of unit, $b = B - b_b$
$= 1.155$ m

Internal width of unit, $d = D - 2 \times b_b$
$= 0.355$ m

Moment of inertia of diaphragm unit is given by:

$$I = \frac{B \times D^3}{12} - \frac{b \times d^3}{12}$$

$$= \frac{1.27 \times (0.563)^3}{12} - \frac{1.16 \times (0.355)^3}{12}$$

$= 0.015$ m^4 (Refer to Fig 2.1)

$y = D/2 = 0.282$ m

In accordance with the code, permissible tension in masonry with mortar M1 is

0.07 N/mm^2 = 70000 N/m^2

and $w = 20000$ N/m^3

Calculating Height of Wall:

Bending moment,

$$M = \frac{p_z \times B \times H^2}{2}$$

$$= \frac{584.9 \times 1.27 \times H^2}{2} = 371.4 \times H^2$$

Allowable tension in masonry,

$$f_t = \frac{M \times y}{I} - \frac{W}{A} = \frac{M \times y}{I} - w \times H$$

$$70000 = \frac{371.4 \times H^2 \times 0.282}{0.015} - 20000 \times H$$

$$= 6982H^2 - 20000H$$

Transposing and simplifying,

$698.2H^2 - 2000H - 7000 = 0$

$H^2 - 2.86H - 10 = 0$

Solving the quadratic equation we get

$$H = \frac{2.86 + \sqrt{(2.86)^2 + 4 \times 1 \times 10}}{2 \times 1} = 4.9\,\text{m}$$

= 5.0 m (say)

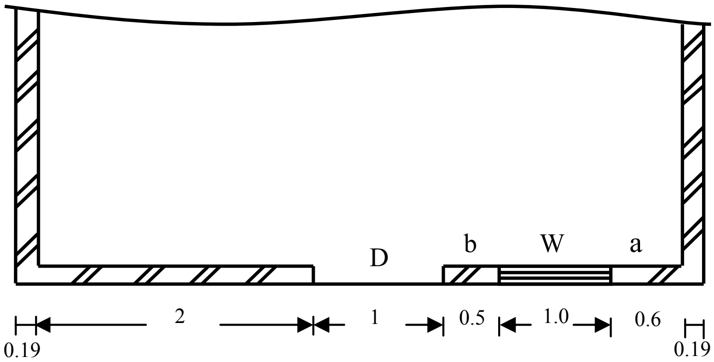
Hence, the wall can be built to a height of 5 m with M1 mortar.

Example 3 – DESIGN A WALL OF A ROOM WITH OPENING

Problem Statement:

Design a wall of a room with openings

as shown in Figure 3.1. The wall is of 1 brick thick. The height of floor to ceiling is 2.8 m and height of plinth is 1.2 m. The RCC roof slab is 100 mm thick with clear span of 3.0 m. Height of 100 mm thick parapet above roof slab is 0.8 m. Height of taller door opening is 2.0 m. Unit weight of masonry and concrete may be taken as 20 kN/m^3 and 25 kN/m^3 respectively. Assume live load as 1.5 kN/m^2.



(All dimensions are in meters)

Figure 3.1: Plan of wall with opening

Solution:

From Figure 3.1 it is observed that portion 'b' of wall will have the maximum stress. We will, however, for the sake of comparison and illustration, work out stress at plinth level in portion 'a' of the wall as well. Since there are no openings below PL, load disperses below plinth and corresponding stresses get reduced notwithstanding the increase in self-load of masonry.

Given Data:

Wall thickness = 0.2 m

Plinth height = 1.2 m

Floor to ceiling height = 2.8 m

Clear span of RCC slab = 3.0 m

Thickness of slab = 0.1 m

Parapet Details:

Height of parapet above roof slab = 0.8 m

Thickness of parapet = 0.1 m

Plaster thickness = 0.03 m

Unit weight of masonry, γ_m = 20 kN/m^3

Unit weight of concrete, γ_c = 25 kN/m^3

Live load, LL= 1.5 kN/m^2

Unit weight of lime terrace, γ_{lt} = 20 kN/m^3

Calculation of Loads:

Parapet load, P_{pt} = (0.19+0.03)×0.8×20×10^3

= 3.5 kN/m

Roof load:

Roof slab, Psl = 0.1×25×10^3 = 2.5 kN/m^2

Lime terrace of 120 mm thick,

P_{lt} = 0.12×20×10^3 = 2.4 kN/m^2

Total roof load, P_t = 1.5+2.5+2.4 = 6.4 kN/m^2

Effective Span of slab,

$l_{eff} = l_s + 0.1$=3.0+0.1 = 3.1 m

Roof load on wall,

P_{rw} = 0.5×6.4×3.1 = 9.92 kN/m

Self weight of wall,

P_{sw} = (0.19+0.03)×2.8×20×10^3 = 12.32 kN/m

Portion "a" of wall:

Length of wall (up to centre of cross wall)

= 0.6+0.5×0.19 = 0.69 m

This portion bears additional load on account of opening on one side, which is 1.0 m in width.

Calculation of compressive stress

Total load on wall,

P = (3.5+9.92+12.32)×(0.69+0.5)

= 30.65 kN

Since the wall is plastered on both sides, it may be assumed to have raked joints on both sides.

Effective thickness = 0.19-0.02 = 0.17 m

Effective area of wall, A = 0.17×0.69 = 0.118 m^2

Compressive stress at plinth level,

$$f_c = \frac{P}{A} = \frac{30.65 \times 10^3}{0.118} = 0.26\,\text{MPa}$$

Calculation of Slenderness Ratio

Total height of a wall, h = 1.2+2.8+0.05 = 4.05 m

Slenderness ratio from consideration of

$$\text{height} = \frac{0.75 \times 4.05}{0.17} = 18$$

Since the wall (a) is supported by cross wall at one end and free at other,

Effective length of wall l_{eff} = 2×0.69 = 1.38 m

Slenderness ratio from consideration of

$$\text{length} = \frac{1.38}{0.17} = 8.0 \quad \text{(Governs)}$$

Stress reduction factor, $k_s = 0.95$

Area reduction factor,

$k_a = 0.7 + 1.5 \times A = 0.7 + 1.5 \times 0.118 = 0.88$

Shape modification factor = 1.0

Basic compressive stress required for masonry,

$$f_b = \frac{f_c}{k_a k_s} = \frac{0.26}{0.93 \times 0.88} = 0.31\,\text{MPa}$$

Portion "b" of wall:

Length of wall, $l_{wb} = 0.5\text{m}$

Length of opening on both sides, $l_{wbo} = 1.0\text{m}$

Calculation of compressive stress

Since the length of wall being less than 4 times thickness of wall, it becomes a column.

Total weight of wall at plinth level, (taking opening at the both side into account)

$$= 25.76 \times \left(\frac{1.0}{2} + 0.5 + \frac{1.0}{2} \right) = 38.64\,\text{kN}$$

Compressive stress in wall at plinth level,

$$= \frac{38.64 \times 10^3}{0.17 \times 0.5} = 0.45\,\text{MPa}$$

Calculation of Slenderness Ratio

Since the wall is supported by RC slabs both at top and bottom, it can be considered as fully restrained along its height.

Total height of wall, H = 4.05 m

Height of taller openings, H_{to}=2.0 m

Effective height of wall "b" is given by,

$h = 0.75 \times 4.05 + 0.25 \times 2.0 = 3.54$ m

Slenderness ratio in the direction of height is given by,

$$= \frac{3.54}{0.17} = 21$$

Slenderness ratio in the direction of length is given by,

$$= \frac{4.05}{0.17} = 8$$

Determination of grade of mortar:

As per Draft Code IS: 1905 Table 9, stress reduction factor, $k_{sb} = 0.59$

(for slender ratio of 21.0)

Area of portion of wall in plan

= 0.5 × 0.17

= 0.085 m^2

Area reduction factor,

k_a=0.7+1.5A =0.7+1.5×0.085 =0.83

Shape modification factor is taken as unity.

Basic compressive stress required for masonry,

$$f_b = \frac{0.45}{0.59 \times 0.83} = 0.93\,\text{MPa}$$

Obviously stress in Wall 'b' will govern the design. So bricks of 10.0 MPa strength are required (Refer to Table 10). For these bricks, from Table 12 of Draft Code IS:1905, the shape modification factor is given by $k_p = 1.1$.

Basic compressive stress of required masonry,

$$f_b = \frac{f_{bcb}}{k_p} = 0.846 \text{ MPa}$$

So, the grade of mortar for the masonry to be used is M1 (Refer to Table 10).

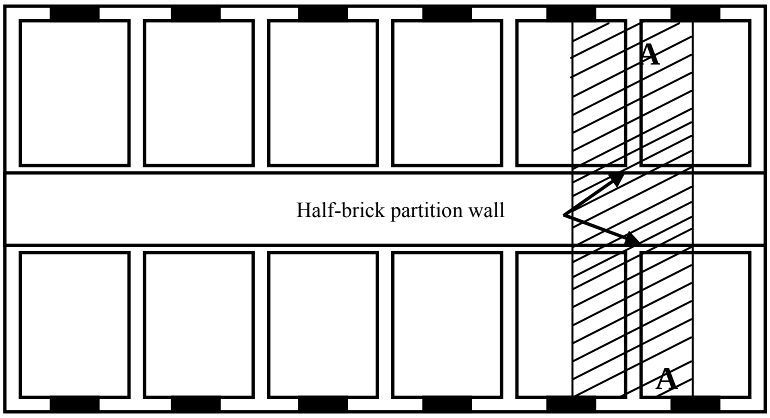
Remarks:

It may be mentioned that if there is only a small portion of wall which is carrying high stress, it may be possible to effect economy in cost by using a lower grade masonry for walls which do not have large openings and to use the masonry we have calculated only for the portion of wall 'b' which has openings in both sides. For that purpose stresses on other walls should also be calculated and masonry design accordingly. It should however be kept in view that if in one storey of a building, bricks and mortar of different strength/grades are to be used a very close supervision is required in order to avoid mistakes.

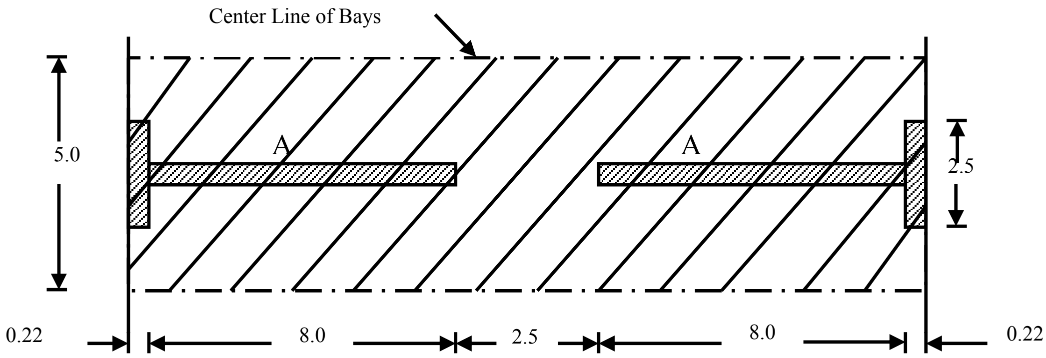
Example 4 – DESIGN OF UNREINFORCED CROSS WALL FOR WIND LOAD

Problem Statement:

A 3-storyed building as shown in Figure 4.1 has load bearing cross walls of 230 mm thickness. The building is subjected to a wind pressure of 1.32 kN/m^2. External longitudinal walls are also 230 mm thick while internal corridor walls are ½ brick thick. All walls are plastered both sides. Design the masonry is the cross walls of first floor. Assume roof and floor loads (RCC slab) to be 7 kN/m^2. The building is without any parapet over the roof. Center to center height between floors is 3 m.



GENERAL PLAN



TYPICAL PLAN OF CROSS WALL 'A'

(All dimensions are in meters)

Figure 4.1: General Plan of Building and Cross Wall under Consideration

Solution:

Design Data:

Number of stories = 3

Number of bays = 6

Width of building =18.5 m

Thickness of cross wall = 0.23 m

Bay length = 30 m

Thickness of external wall = 0.23 m

Thickness of internal corridor wall = 0.115 m

Centre to centre height of a wall, h = 3.0 m

Roof and floor load = 7 kN/m^2

Wind pressure = 1.32 kN/m^2

Unit weight of masonry = 20×10^3 N/m^3

Length of cross wall = 8.0 m

Depth of raking = 0.01 m

Thickness of plaster = 0.03 m

Height to Width Ratio of Building:

We will consider the design of atypical cross wall of first floor marked AA on the plant. Roof and floor loads borne by the typical cross walls is shown shaded on the plan in Figure 4.1

Height to width of the building equals

$$\frac{3\times 3}{(20.5+0.44)}=0.43$$

Since the spacing of cross wall spaced 5 meter apart, (limit is 6.0 meter for 20 cm wall), it is not necessary to work out wind stresses. However, wind stress in the transverse direction has been calculated in this context for sake of illustration.

Calculation of Loads:

Assume that roof and floor slabs are 150 mm thick. Since the building is only 3 storeyed, we could ignore the live load reduction factor for the sake of simplicity. As structural system is based on cross wall construction, slabs are designed for one-way action so that the slab load is supposed to come only on the cross walls. Since walls are plastered, joints of masonry are assumed to be raked.

Roof / floor load per bay per floor,

$$P_r = 7\times 10^3 \times 5\times 18.5 = 648\,\text{kN}$$

Self weight of cross wall per bay per floor

$$P_{cw} = 2\times 20\times 10^3 \times (0.22+0.03)\times 8\times 3$$

$= 240\times 10^3$ N = 240 kN

Self weight of 2 corridor walls (1/2 brick thick) per floor per bay ignoring opeings

$= 2\times 20\times 10^3\times(0.105+0.03)\times 3\times 5 =$ 81 kN

Wind load per bay per floor

$= 1320\times 5\times 3$ N = 20 kN

Wind forces at different floor levels is shown in Figure 4.2.S

Calculation of Compressive Stress:

Total vertical load on cross walls *AA* of first floor at plinth level,

$= 3(240)+2\times 648+2\times 81$

$= 2.18\times 10^3$ kN

Area of cross walls in plane per bay assuming total depth of raking to be 20 mm

$= 2\times 8\times(0.22-0.02)$

$= 3.2\ \text{m}^2$

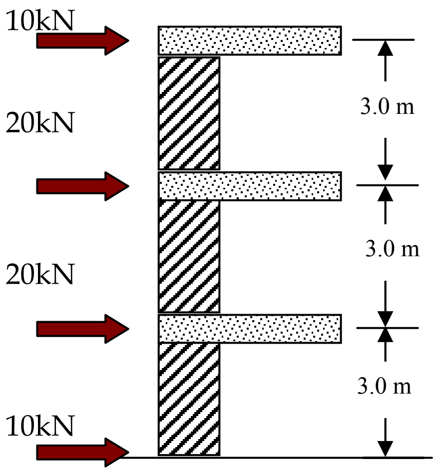


Figure 4.2: Wind forces in one bay

Direct compressive stress in masonry due to vertical loads,

$$=\frac{2.18\times 10^6}{3.2}=0.7\,\text{MPa}$$

Calculation of Bending Stress due to Wind Load:

Wind load normal to the main elevation per bay will be acting as shown in Fig. 4.2

Wind moments at plinth wall of building per bay,

$=10\times 9+20\times 6+20\times 3$

=270 kN

Total wind moments at plinth wall of building,

$= 270\times 6$

= 1620 kNm

Total wind moments will share by the cross walls and walls in the ratios of their respective stiffness.

Maximum projecting flange length being actual distance between windows openings and cross walls or 12 times the thickness of longitudinal wall or $H/6$; whichever is less

(Draft Code IS:1905 Section 4.2.2.5)

a) Distance between the openings and cross wall,

= 2.5-0.22 = 2.28 m

b) Least of following

(i) 12 times thickness of wall

= 12×0.22 = 2.64 m

(ii) $\frac{H}{6} = \frac{3\times 3}{6} = 1.5\,\text{m}$ (Governs)

Effective flange width of cross wall resisting bending moment is 1.5+0.22 = 1.72 m

Moment of inertia of cross wall *AA* inclusive of flanges, assuming two walls act integrally

$$= \frac{0.22\times 18.5^3}{12} - \frac{0.22\times 2.5^3}{12}$$

$$+ 2\times 0.22\times 1.72\times\left(\frac{18.5+0.22}{2}\right)^2$$

= 182.1 m^4

Moments of inertia of one end walls:

The projecting flange width is to be limited to 6t or H/16, whichever is less.

(i) 6 times thickness of wall =6×0.22 m = 1.32 m

(ii) $\frac{H}{16} = \frac{3\times 3}{16} = 0.56\,\text{m}$ (Governs)

Moments of inertia of one end wall:

$$= \frac{0.22\times 18.5^3}{12} - \frac{0.22\times 2.5^3}{12}$$

$$+2\times 0.22\times 0.78\times\left(\frac{18.5+0.22}{2}\right)^2$$

= 145.8 m^4

Total moment of inertia of five cross walls and two end walls,

I= 5×182.1+2×145.8 m^4

= 1202.1 m^4

Bending moment borne by one inner cross wall

$$= \frac{1620\times 10^3\times 182.1}{1202.1} = 245.4\,\text{kNm}$$

In view of wind/earthquake load, the permissible stress should be increased by 33% i.e., the load level should be reduced to 75%.

Bending stress on cross wall due to wind moment:

$$= \pm\frac{0.75My}{I}$$

$$= \pm\frac{0.75\times 245.4\times 10^3}{182.1}\times\left(\frac{18.5+0.44}{2}\right)$$

= 0.0096 MPa

Overall compressive stress on cross wall

= 0.7±0.0096

= 0.7096 MPa or 0.6904 MPa

Hence, when height/width ratio of a building is small, the bending stress can be neglected and the design could be based on direct compressive stress only.

Design of Cross wall:

Slenderness ratio of wall, assuming plinth level is 1.0m above top of footing

$$= \frac{h}{t} = \frac{0.75(3.0+1.0)}{0.22-0.02} = 15$$

Stress reduction factor, $k_s = 0.76$

(IS: 1905 Draft Code Table 11)

Shape modification factor, $k_{sh} = 1$

Basic compressive stress

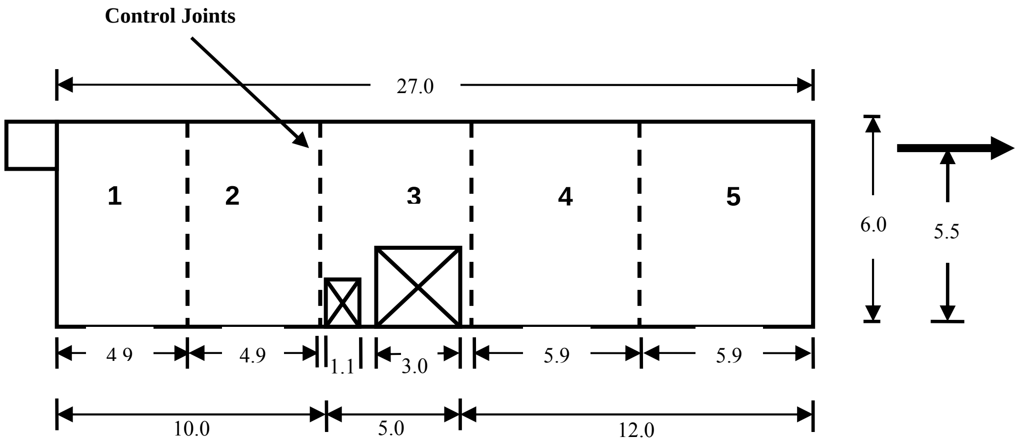
$$= \frac{f_c}{k_s} = \frac{0.711}{0.76} = 0.93\,\text{MPa}$$

Referring to Table 10 of IS Code IS:1905, bricks should be 10 MPa and M1 mortar.

Example 5 – DESIGN OF UNREINFORCED SHEAR WALL FOR IN-PLANE FLEXURE AND SHEAR

Problem Statement:

Design the pier 1 of the shear wall of a shopping center shown in Figure 5.1. Pier 1 is subjected to a shear load of 10 kN from the diaphragm due to wind load and 30 kN due to seismic load, applied at the roof height of 5.5 m. Assume, compressive strength of masonry, f_m = 10 MPa



(All dimensions are in meters)

Figure 5.1: Shear wall of shopping center

Solution:

Load Combinations:

The loading on pier 1 is shown in the figure 5.2. It is assumed that there is no gravity roof load applied to this wall pier. Only two load combinations i.e. (i) 0.75(D+W) and (ii) 0.75(D+E) are checked, for illustration.

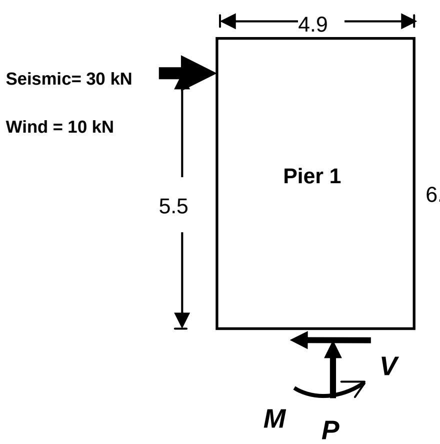


Figure 5.2: Loading on pier 1

Load Combination 0.75(D+W):

Assume one brick wall with raked joints to a depth of 1cm on both sides is used with unit weight of 20 kN/m^3. Nominal thickness of wall is 200 mm and effective wall thickness is 190 mm.

At the base of wall per meter length:

Factored axial load is given by

$P = 0.75\times20\times6\times0.19\times4.9$ kN = 83.79 kN

Factored bending moment is given by

$M = 0.75\times10\times5.5$ kN = 41.25 kN

Check for Tension

Area of wall is given by

$A = 4.9\times0.19$ m^2 = 0.931 m^2

Section modulus is given by

$$S = \frac{0.19\times4.9^2}{6} \text{ m}^3 = 0.76 \text{ m}^3$$

In-plane flexural considerations:

Max. tensile stress $= \frac{-P}{A} + \frac{M}{S}$

$= \frac{-83.79}{0.931} + \frac{41.25}{0.76} = -0.036$ MPa (Compression)

Therefore, no tensile stresses occur.

Check for Shear

Shear force due to wind = 0.75 × 10 kN = 7.5 kN

Max. shear stress for rectangular section

$$f_v = \frac{3}{2}\frac{V}{A} = \frac{3}{2}\left(\frac{7.5\times10^3}{0.931}\right)$$

$$f_v = 0.012 \text{ MPa}$$

Allowable shear stress is given by the least of the following:

(i) 0.5 MPa

(ii) $\left(0.1 + 0.2\times f_d\right)$

$= \left(0.1 + 0.2\times\frac{P}{A}\right)$ = 0.118 MPa

(iii) $0.125\times\sqrt{f_m} = 0.125\times\sqrt{10}$

= 0.395 MPa

Allowable shear =0.118 MPa > 0.012 MPa

(Draft Code IS: 1905: Sec 5.9.1.1)

Hence, the design of section is safe in shear.

Load Combination 0.75(D+E):

Assume 1 brick wall with raked joints to a depth of 10 mm on both sides is used. Nominal thickness of wall is 200 mm and effective wall thickness is 190 mm.

At the base of wall per meter length:

Axial load is given by

P = 0.75×20×6×0.19×4.9 kN = 83.79 kN

Bending moment is given by

M = 0.75×30×5.5 kN = 123.75 kN

Check for Tension

In-plane flexural considerations:

Max. tensile stress

$$= \frac{-P}{A} + \frac{M}{S} = \left[\frac{-83.79}{0.931} + \frac{123.75}{0.76}\right]\times10^{-3} = 0.07 \text{ MPa}$$

As no tension is allowed in un-reinforced masonry as Draft Code IS: 1905, the design should be modified. Assume two brick wall with raked joints to a depth of 10 mm on both sides is used. Nominal thickness of wall is 40 cm and effective wall thickness is 39 cm.

Area of wall section is given by A = 1.911 m^2

Section Modulus given by S = 1.56 m^3

Axial load is given by

P = 0.75×20×6×0.39×4.9 kN = 172 kN

Bending moment is given by

M = 0.75×30×5.5 kN = 123.8 kN

In-plane Flexural considerations:

Max. tensile stress =

$$\frac{-P}{A} + \frac{M}{S} = \left[\frac{-172}{1.911} + \frac{123.8}{1.56}\right]\times10^{-3} = -0.011 \text{ MPa}$$

(Compressive)

Since no tension occurs in the section, the design is OK

Note: Increase in masonry wall of 2 brick thick may increase seismic force (for 30 kN*) and should be recalculated. However, in this problem this increase in force is neglected.*

Check for Shear

Shear force due to earthquake = 0.75 × 30 kN =225 kN

Max. shear stress for rectangular section

$$f_v = \frac{3}{2}\frac{V}{A} = \frac{3}{2}\left(\frac{10\times10^3}{1.911}\right)$$

$$f_v = 0.018 \text{ MPa}$$

Allowable shear stress (F_v) is given by the least of the following:

(i) 0.5 MPa

(ii) $0.1 + 0.2\times f_d$ MPa

$= 0.1 + 0.2\times\frac{P}{A}$ = 0.129 MPa (Governs)

(iii) $0.125\times\sqrt{f_m} = 0.125\times\sqrt{10}$

= 0.395 MPa

Allowable shear =0.129 MPa > 0.018 MPa

(Draft Code IS: 1905: Sec 5.9.1.1)

Hence, the design of section is safe in shear.