
IITK-GSDMA GUIDELINES
for **STRUCTURAL USE**
of **REINFORCED MASONRY**
Provisions with Commentary and Explanatory Examples



Indian Institute of Technology Kanpur



Gujarat State Disaster Mitigation Authority

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The material presented in this document is to help educate engineers/ designers on the subject. This document has been prepared in accordance with generally recognized engineering principles and practices. While developing this material, many international codes, standards and guidelines have been referred. This document is intended for the use by individuals who are competent to evaluate the significance and limitations of its content and who will accept responsibility for the application of the material it contains. The authors, publisher and sponsors will not be responsible for any direct, accidental or consequential damages arising from the use of material content in this document.

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PREFACE

The use of plain or unreinforced masonry (URM) is limited in scope and structural capacity which is needed for large building systems, or when the lateral forces due to wind and earthquakes are significantly large. Use of reinforcement in masonry enhances its flexure and shear capacity which can overcome the limitations in the use of URM. Masonry units are being manufactured in shapes and sizes that make reinforcement embedding in masonry less cumbersome. With these developments, structural design of load bearing masonry buildings has been undergoing considerable modification as evidenced by changes that are taking place in the masonry design codes throughout the world.

Reinforced masonry is a construction system where steel reinforcement in the form of reinforcing bars or mesh is embedded in the mortar or placed in the holes and filled with concrete or grout. By reinforcing the masonry with steel reinforcement, the resistance to seismic loads and energy dissipation capacity can be improved significantly. In reinforced masonry, tension is developed in the masonry, but it is not considered to be effective in resisting design loads; reinforcement is assumed to resist all the tensile stresses.

The primary Indian masonry code, IS: 1905-1987 provides a semi-empirical approach to the design of unreinforced masonry, especially for stresses arising from vertical and moderate lateral loads, such as wind. The permissible stress values are not directly linked to masonry prism test values and do not address the strength and ductility of masonry members under large lateral loads due to earthquakes. During the IITK-GSDMA project, the IS:1905 has been modified to reflect the modern concept of engineered masonry. This modified version is referred as *Suggested Draft IS:1905* throughout this document and can be had from www.nicee.org.

The present IITK-GSDMA guidelines on the structural use of reinforced masonry is the next step in introducing the modern concept of engineered reinforced masonry so that benefits of economical and durable masonry construction can be continued to be enjoyed in contemporary structures, which place heavy demands on the resilience and ductility. The design provisions of these guidelines follow principles of allowable stress design, which is consistent with the design philosophy of IS:1905. However, the recent trend world over has been towards the development of limit/ultimate strength design for masonry. It is believed that at this stage in the country wherein the first time the engineered approach to design of reinforced masonry is being introduced; a more reliable, familiar and time-tested allowable stress design procedure is followed.

The material contained in this document also presented in two workshops held in Ahmedabad and New Delhi for practicing structural engineers. The feedback received on these occasions has been appropriately addressed while revising the document. It is hoped that these guidelines will encourage the design of reinforced masonry especially those structures which are expected to resist large earthquake loads.

The comprehensive comments received from the reviewers, Prof. Alok Madan of IIT Delhi and Prof. P Dayaratnam, Ex-Vice Chancellor, Jawaharlal Nehru Technical University, Hyderabad, have been extremely useful in improving these guidelines. Mr Sandeep Pal, Mr Ashish Narayan, and Mr Ankur Singh provided significant assistance in research and analysis of a large number of documents available on the subject. Mr Dipti Ranjan Sahoo helped in developing explanatory examples for the guidelines. Later Mr Kamlesh Kumar and Samaresh Paikara joined them for additional help in editing and revising. Their work on this project has been very critical and is gratefully acknowledged.

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IITK-GSDMA GUIDELINES

for **STRUCTURAL USE**

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Provisions with Commentary and Explanatory Examples

PART 1: GUIDELINES AND COMMENTARY

IITK-GSDMA
***Guidelines for Structural Use of
Reinforced Masonry***

PROVISIONS

COMMENTARY

0. – Foreword

0. – Foreword

0.1 –

This document is primarily concerned with the use of reinforcement in masonry for enhanced load carrying capacity in general and improved seismic performance. The guidelines addresses the requirements related to the reinforcement in masonry and its detailing as well as those for the seismic design and detailing of masonry shear walls for various level of performance.

C0.1 -

This document addresses reinforced masonry only. However, general provisions of unreinforced masonry contained in IS:1905-1987 (Re-affirmed 1992) will be referred to as mentioned in this document.

During the *IITK-GSDMA Program on Building Code*, the IS:1905 has been modified to reflect the modern concept of engineered masonry. This modified version is referred as *Suggested Draft IS:1905* throughout this document and can be had from www.nicee.org.

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 –

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

0.4 –

This document is prepared as a project entitled "Review of building codes and Handbook" awarded to Indian Institute of Technology Kanpur by GSDMA, Gandhinagar through World Bank Finances.

0.5 –

In the development of these guidelines reference is made to the following documents.

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- 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.
- g) AS 1640-1974 - SAA Brickwork Code. Standards Association of Australia.
- h) National Building Code of Canada, 1977. National Research Council of Canada.
- i) DIN 1053/I Code on brick calculation and performance. Deutsches Institut für Normung.
- j) CP111: Part2: 1970 Structural recommendations for load bearing walls with amendments up to 1976. British Standards Institution.
- k) BS 5628: Part 1: 1992 & BS 5628: Part 2: 2000 Code of practice for structural use of masonry, Part 1 Unreinforced masonry, Part 2 Reinforced masonry. British Standards Institution.
- l) CP 12 1: Part 1: 1973 Code of practice for walling, Part 1 Brick and block masonry. British Standards Institution.
- m) Recommended practice for engineered brick masonry. Brick Institute of America, 1969.
- n) Masonry Designer's Guide (Third Edition), The Masonry Society.

PROVISIONS

1. – Scope

1.1 –

This document gives recommendations for structural design aspect of 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.

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.

PROVISIONS

2. – Terminology

For the purpose of this document, the definitions given in IS: 2212-1962*, IS:1905-1987 (Re-affirmed 1992)** and suggested draft IS: 1905*** and the following shall apply.

*Code of practice for brickwork

**Code of practice for structural use of unreinforced masonry

*** Code of practice for structural use of unreinforced masonry (Draft suggested by IITK-GSDMA Program on Building Codes which is available from www.nicee.org).

2.1 – 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

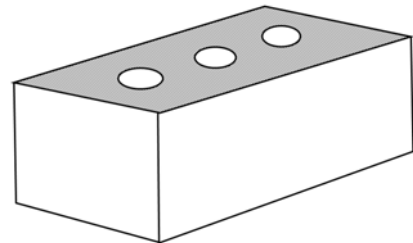
COMMENTARY

C2 – Terminology

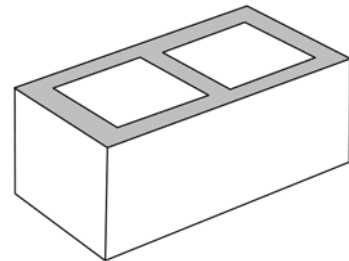
For the consistent use of this document, various terms are assumed to have certain meaning in this document. Many terms as defined in this commentary not always correspond to their meaning in ordinary usage.

C2.1– 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 C1 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.



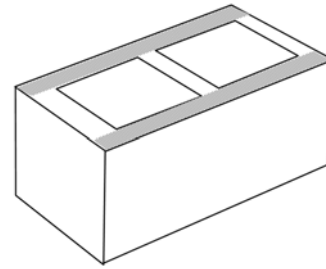
(a) Brick more than 75% solid. Net area equals gross area



(b) Hollow Unit: Full Mortar Bedding

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(Requires alignment of cross webs)

(c) **Hollow unit: Face Shell bedding**

Figure C1: Cross sectional area of masonry unit

2.2 – Grout

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

2.3 – Grouted Masonry

2.3.1 – Grouted Hollow-Unit Masonry

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

2.3.2 – Grouted Multi-Wythe Masonry

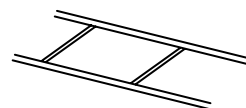
That form of grouted masonry construction in which the space between the wythes is solidly or periodically filled with grout.

2.4 – Joint Reinforcement

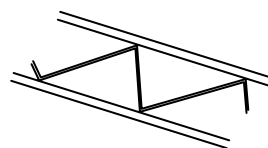
A prefabricated reinforcement in the form of lattice truss which has been hot dip galvanized after fabrication and is to be laid in the mortar bed joint.

C 2.4 – Joint Reinforcement

Joint reinforcements are usually arrangement of reinforcing bars as chords and web. Common configurations are ladder type or truss type arrangement as shown in Figure C2 below.



Ladder type joint reinforcing



Truss type joint reinforcing

Figure C2: Joint reinforcement

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2.5 – 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.

C2.2 – Pier

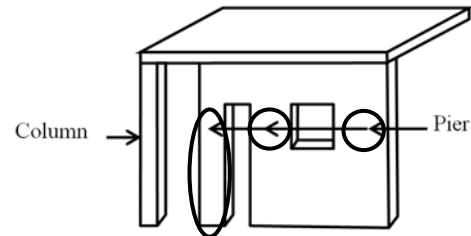


Figure C3- Piers and columns

2.6 – Prism

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

2.7 – Reinforced Masonry

Masonry which is reinforced to the minimum requirements of this document and grouted so that the two materials act together in resisting forces.

2.8 – Grouted Cavity Reinforced Masonry

Two parallel single leaf walls spaced at least 50 mm apart, effectively tied together with wall ties. The intervening cavity contains steel reinforcement and is filled with infill concrete so as to result in common action with masonry under load.

2.9 – Pocket type Reinforced Masonry

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

2.10– Quetta Bond Reinforced Masonry

Masonry at least one and half units thick in which vertical pockets containing reinforcement and mortar or concrete infill occur at intervals along its length.

2.11 – Specified Compressive Strength of Masonry

Minimum Compressive strength, expressed as force per unit of net cross- section area, required of the

PROVISIONS

COMMENTARY

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.

2.12 – Wall Tie

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

2.13 – Wythe

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

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3. – Materials

C3 – Materials

3.1 – Masonry Units

C3.1 – Masonry Units

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. The minimum compressive strength of masonry units used for reinforced masonry shall be 7 MPa.

C3.1.1 –

This arbitrary limit is to restrict the reinforced masonry construction only with good quality bricks of adequate strength and durability. This requirement is same as specified in the British Standard BS: 5628.

Strength of bricks in India varies from region to region depending on the nature of available soil and technique adopted for moulding and burning. It is possible to manufacture of bricks of improved quality from soils such as black cotton and moorum, which ordinarily give bricks of very low strength.

The following gives a general idea of the average strength of bricks in MPa available in various parts of India, employing commonly known methods for moulding and burning of bricks:

Delhi and Punjab	7 to 15
Uttar Pradesh	10 to 25
Madhya Pradesh	3.5 to 5
Maharashtra	5
Gujarat	3 to 10
Rajasthan	3
West Bengal	10 to 25
Andhra Pradesh	3 to 7
Assam	3.5

Recently machine-made bricks are now being produced, which give compressive strengths varying between 17.5 and 25 MPa. Bricks with hollow cores should be encouraged which makes the placement of reinforcement rather easy.

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, development of the bond of the grout to both reinforcement and masonry units and avoidance of corrosion of reinforcement.

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.

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3.2 – Mortar

3.2.1 –

For reinforced masonry, only high strength mortars (H1 and H2) shall be used to ensure transfer of internal forces from steel reinforcement to masonry.

3.3 – Reinforcement

Steel Reinforcement of grade Fe 415 (see IS: 1786-1985) or less only shall be used. Deformed bar shall be used.

3.4 – Material Properties

3.4.1 – General

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

3.4.2 – Elastic modulus

For steel reinforcement,
 $E_s = 200 \text{ GPa} = 2.0 \times 10^5 \text{ MPa}$

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 of suggested draft IS: 1905.

3.4.3 – Shear modulus

COMMENTARY

C3.2 – Mortar

C3.2.1 –

For selection and use of mortar reference should be made to suggested draft IS: 1905.

C3.4 – Material Properties

C3.4.1 – General

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

C3.4.2 - 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. 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 C4 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 C4: Chord Modulus of Elasticity

C3.4.3 – Shear modulus

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

COMMENTARY

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.

4. – DESIGN CONSIDERATIONS

4.1 – General

Design considerations of IS:1905-1987 in Section 4 shall apply alongwith the provisions of this Section.

4.2 – Structural Continuity

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

C4 – DESIGN CONSIDERATIONS

C4.1 – General

Certain issues are common for unreinforced and reinforced masonry construction and should be satisfied by the designer.

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4.3 – Effective span

C4.2 – Effective span

4.3.1 –

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

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

C4.3.2 –

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

4.3.2 –

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.

4.4 – Slenderness Ratio

C4.4 – Slenderness Ratio

4.4.1 – Wall

C4.4.1 – Wall

For reinforced masonry members such as walls subjected to out-of-plane bending and a beam as a part of wall subjected to bending in the plane of the wall, may have maximum slenderness ratio as given in Table 1.

A higher value of slenderness ratio is permitted for reinforced masonry than unreinforced masonry for its increased ability to resist flexural tension.

End Condition	Ratio of span to effective depth
i. Simply supported	35
ii. Continuous	45
iii. Spanning in two directions	45
iv. Cantilever	18

4.4.2 – Columns

C4.4.2 – Columns

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

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.

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4.5 Minimum Design Dimensions

4.5.1 – Minimum Thickness of Load Bearing Walls Columns

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

4.5.2 – Parapet Wall

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

4.6 Eccentricity in Columns

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

C4.6 – Eccentricity in columns

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.

5. – Requirements Governing Reinforcement and Detailing

C5 – Requirements Governing Reinforcement and Detailing

5.1 – General

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

The provisions of this section are modeled after IS: 456, the code for the structural use of plain and reinforced concrete. Some requirements have been simplified and many which are not suitable for masonry have been dropped.

C 5.1 – General

The provisions of section 5 apply to detailing of reinforcement and design of anchorage, development and splices. Provisions cover minimum bend radii, and covers. The provisions for development include deformed and plain bars and wire and bundled bars.

PROVISIONS

5.1.1 –

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

$$A_t = A_b + m A_s$$

where

A_t = Total transformed cross-sectional area of the member

A_b = Cross sectional area of brick

A_s = Cross sectional area of reinforcement

m = modular ratio of steel reinforcement and brick

5.1.2 –

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

COMMENTARY

C 5.1.1 –

In reinforced masonry, materials of different elastic properties are involved and considering the composite action of the members, for strains compatibility different stresses will develop in the component materials. To compute these strains, an easy method is to consider transformed section with respect to the axis of resistance. The transformed section is obtained by increasing area of the weaker element by the ratio of elastic moduli of stronger element to the weaker element (modular ratio). Stresses computed based on the transformed section, are then multiplied by modular ratio to obtain actual stresses.

C5.1.2 –

Cracking in masonry significantly reduces the stiffness of masonry component and strictly speaking cracked section properties should be used for those members which have reached cracked state. The determination of members in cracked or uncracked state is an iterative procedure and can be quite elaborate which can not be justified for many structures. The document permits stiffness to be computed on the basis of uncracked sections' properties for analysis purposes.

5.2 – Steel Reinforcement- Allowable Stresses

5.2.1 – Tension

Tensile stress in reinforcement shall not exceed the following:

MS Bars conforming to IS 432 (Part I)	140 MPa for diameter ≤ 20 mm 130 MPa for diameter > 20 mm
HYSD Bars (IS 1786)	230 MPa

5.2.2 – Compression

Compressive stress in reinforcement shall not exceed the following:

MS Bars conforming to IS 432 (Part I)	130 MPa
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PROVISIONS

HYSB Bars (IS 1786)	190MPa
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5.3 – Size of Reinforcement

- The maximum size of reinforcement used in masonry shall be 25 mm diameter bars and minimum size shall not be less than 5 mm.
- The diameter of reinforcement shall not exceed one-half the least clear dimension of the cell, bond beam, or collar joint in which it is placed.

5.4 – Spacing of Reinforcement

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

5.5 – Anchorage

5.5.1 – Development Length of Bars

The development length L_d for deformed bars conforming to IS 1786 shall be given by the following equation but shall not be less than 300 mm.

$$L_d = 0.25d_b F_s$$

Where,

d_b = nominal diameter of bar (mm)

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

COMMENTARY

C5.3 – Size of Reinforcement

The 25 mm diameter bar limit is arbitrary. Though it has not been shown conclusively that more number of small bars provide better performance than fewer large bars because of better bond, it certainly helps in avoiding unreasonable cover distance, anchorage length, etc.

In order to develop good bond, free flow of grout is necessary. This limitation on diameter of reinforcing bar with respect to core area has been shown to ensure good bond.

C5.4 – Spacing of Reinforcement

With the limitation on spacing limits, mortar/grout readily flows into spaces between bars without honeycombing, and concentration of bars on a line that might result in shear or shrinkage cracking, is prevented.

C5.5 – Anchorage

C5.5.1 – Development Length of Bars

For proper anchorage of the reinforcement, except for use of mechanical anchors, the development length L_d provided in a member must be sufficient to develop the design stress in steel without exceeding the design value for bond stress. If sufficient development length is not provided, hooks may be added or a greater number of smaller diameter bars can be substituted. The expression for development length can be derived by using an allowable bond stress of 1.0 MPa for deformed bar embedded in the mortar or grout.



$$\Delta T = \pi d_b u L_d$$

$$\pi d_b^2 F_s / 4 = \pi d_b u L_d$$

$$L_d = d_b F_s / (4 u) = d_b F_s / 4 (1 \text{ MPa})$$

$$= 0.25 d_b F_s (\text{mm})$$

Where,

F_s = calculated stress in the steel at some location

u = allowable bond stress

d_b = bar diameter

PROVISIONS

5.5.2 –

For MS bars and epoxy coated bars L_d shall be increased by 60%.

5.5.3 – Standard Hooks

a) Standard hooks shall be formed by one of the following methods:

- (i) 180 degree turn plus extension of at least 4 bar diameters but not less than 64mm at free end of bar.
- (ii) A 90 degree turn plus extension of at least 12 bar diameters at free end of bar.
- (iii) For stirrup and tie anchorage only a 90 degree or a 135 degree turn plus an extension of at least 5 bar diameters at the free end of the bar.

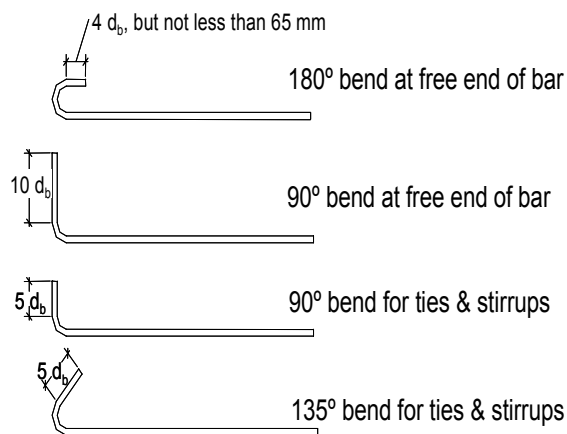


Figure 1: Standard hooks

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

(c) Inside diameter of bend for 12 mm diameter or smaller stirrups and ties shall not be less than 4 bar diameters. Inside diameter of bend for 16 mm diameter or larger stirrups and ties shall not be less than that given in section 5.5.3b.

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

COMMENTARY

L_d = development length

C5.5.2 –

This provision is due to the absence of ribs in MS bars, which is consistent with the provisions of IS 456. Ribs help in developing better bond by providing more surface area as well as providing mechanical resistance against pullout.

C5.5.3 – Standard Hooks

Any deficiency in the required development length can be made up by anchoring the reinforcing bars suitably. Plain bars should preferably end in hooks, as there may be some uncertainty regarding the full mobilization of bond strength through adhesion and friction. On the other hand, deformed bars have superior bond properties due to mechanical bearing and therefore the provision of hooks is not absolutely essential.

Development length of a bend/hook bar is equal to the length of the reinforcement between the point of tangency of the bend/hook and the critical section plus the anchorage value of bend/hook given in Sec. 5.5.3

Bend in reinforcing bars is easier to measure when it is described in terms of inside diameter of the bend rather than in terms of the bend radius. The minimum diameter of bends is primarily based on the practical consideration of bending the bars without breakage. Also these minimum bend diameters have shown to work well without crushing of masonry mortar or grout.

PROVISIONS

continuous or restrained beams.

5.5.4 – Anchorage of Reinforcing Bars

- a) Deformed bars may be used without end anchorages provided development length requirement is satisfied. Hooks should normally be provided for plain bars in tension.
- b) Bends and hooks shall conform to IS: 2502
 - (i) Bends-The anchorage value of bend shall be taken as 4 times the diameter of the bar for each 45° bend subject to a maximum of 16 times the diameter of the bar.
 - (ii) Hooks-The anchorage value of a standard U-type hook shall be equal to 16 times the diameter of the bar.
- c) For stirrups and transverse ties complete development length and anchorage shall be deemed to have been provided when the bar is provided with standard hook as described in Section 5.5.3

5.6 – Lap Splices

5.6.1 –

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

5.6.2 –

Lap length including anchorage value of hooks for bars in flexural tension shall be L_d (see section 5.5.1) or $30d_b$ whichever is greater and for direct tension shall be $2L_d$ or $30d_b$ whichever is greater. The straight length of the lap shall not be less than $15d_b$ or 200 mm.

The following provisions shall also apply:

Where lap occurs for a tension bar located at:

- a) Top of a section as cast and the minimum cover is less than twice the diameter of the lapped bar, the lap length shall be increased by a factor of 1.4.

COMMENTARY

C5.5.4 – Anchorage of Reinforcing Bars

Any deficiency in the required development length can be made up by anchoring the reinforcing bars suitably. Plain bars should preferably end in hooks, as there may be some uncertainty regarding the full mobilization of bond strength through adhesion and friction. Deformed bars, on the other hand, have superior bond properties owing to mechanical bearing, and therefore provision of hooks is not absolutely essential.

C5.6 – Lap Splices

C5.6.1 –

Adequate lap splices should be provided to develop more than the yield strength of steel for ductility. If not, a member may be subject to sudden splice failure when the yield strength of the steel is reached.

C5.6.2 –

The length of lap depends on the bond stress that is capable of being developed between the reinforcing steel and grout. From testing it has been seen that bond stress failure (or pull-out of reinforcing steel) is only one possible mode of failure for lap splices with other failure modes

being rupture of the reinforcing steel and longitudinal splitting of masonry along the length of the lap.

PROVISIONS

- b) Corner of a section and the minimum cover to either face is less than twice the diameter of the lapped bar or where the clear distance between adjacent laps is less than 75 mm or 6 times the diameter of lapped bar, whichever is greater, the lap length should be increased by a factor of 1.4.

Where both condition (1) and (2) apply, the lap length should be increased by a factor of 2.0.

NOTE- Splices in tension members shall be enclosed in spirals made of bars not less than 6 mm diameter with pitch not more than 100 mm.

5.7 – Curtailment of Tension Reinforcement

- (i) In any member subjected to bending, every reinforcing bar should extend, except at end supports, beyond the point at which it is no longer needed, for a distance equal to the effective depth of the member or 12 times the diameter of the bar, whichever is the greater. The point at which reinforcement is theoretically no longer needed is where the design resistance moment of the section, considering only the continuing bars, is equal to the applied design moment. However, reinforcement should not be curtailed in a tension zone unless at least one of the following conditions is satisfied for all arrangements of design load considered:
- (i) the reinforcing bars extend at least the anchorage length appropriate to their design strength from the point at which they are no longer required to resist bending;
 - (ii) the design shear capacity at the section where the reinforcement stops is greater than twice the shear force due to design loads, at that section;
 - (iii) the continuing reinforcing bars at the section where the reinforcement stops provide double the area required to resist the bending moment at that section.

Where there is little or no end fixity for a member in bending, at least 25% of the area of the tension reinforcement required at mid-span should be carried through to the support. This reinforcement may be anchored in accordance with 5.5, or by providing:

- (i) an effective anchorage length equivalent to 12 times the bar diameter beyond the centre line of the support, where no bend or hook begins before the centre of the support, or
- (ii) an effective anchorage equivalent to 12 times the bar diameter plus $d/2$ from the

COMMENTARY

C5.7 – Curtailment of Tension Reinforcement

The document prohibits the cutting-off of a bar at the theoretically determined point for a number of reasons:

- a) The bar should extend to a distance further from the theoretical cut-off point, in order to avoid stress concentration which may lead to moment cracks even at working loads.
- b) The recommended extension provides for slight deviations due to inaccuracies in the modeling and analysis. For example, the loading may well not be absolutely uniformly distributed, in which case the shape of the bending moment diagram will be different from that assumed.
- c) Termination of tensile reinforcement gives rise to a sharp discontinuity in the steel, causing early opening of flexural cracks which in turn may change into a diagonal crack prematurely.
- d) There could be some secondary bending moment developed either due to boundary conditions or eccentricity.

PROVISIONS

face of the support, where d is the effective depth of the member, and no bend begins before $d/2$ inside the face of the support.

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

5.8– Members Subjected to Flexure and Axial Forces

5.8.1 –

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

5.8.2 – Beams

Reinforcement in masonry designed as beam should be provided over a support where the masonry is continuous, whether the beam has been designed as continuous or not. Where this occurs, an area of steel not less than 50% of the area of the tension reinforcement required at mid-span should be provided in the top of the masonry over the support and anchored in accordance with section 5.5. In all cases at least one quarter of the reinforcement required at mid-span should be carried through to the support and similarly anchored.

5.8.3 – Columns

Design of reinforced column shall meet the requirements of this section.

5.8.3.1 –

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

5.8.3.2 – Lateral Ties

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

- (a) Longitudinal reinforcement shall be enclosed by lateral ties of at least 6mm diameter. Vertical spacing of ties along the length of column shall be lesser of
 - (i) 16 times diameter of longitudinal bar,

COMMENTARY

C5.8.3.1 –

Minimum vertical reinforcement is required in masonry columns to prevent brittle collapse. The maximum percentage limit is based on experience. Four bars are required so ties can be used to provide a confined core of masonry.

C5.8.3.2 – Lateral Ties

Lateral ties provide adequate lateral support to longitudinal bars to prevent instability. From theoretical considerations, it has been shown that the critical unsupported length for buckling of compression bars will be in excess of 16 times the bar diameter.

Lateral tie provides resistance to diagonal tension for column acting in shear. Ties may be located in

PROVISIONS

- (ii) 48 times diameter of lateral tie,
 - (iii) Least dimension of the column.
- (b) Arrangement of lateral ties is such that every corner and alternate longitudinal bar shall have lateral support provided by the corner of a lateral tie with an included angle of not more than 135°

COMMENTARY

mortar joint.

5.9 – Members Subjected to Shear

5.9.1 –

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

C5.9.1 –

5.9.2 –

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

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

Where,

- V = total applied shear force
- s = spacing of the shear reinforcement
- d = distance from extreme compression fiber to centroid of tension reinforcement
- F_s = permissible stress in steel reinforcement as defined in section 5.2.

C5.9.2 –

The required minimum shear reinforcement can be obtained by assuming a 45 degree shear crack originating from the extreme fiber in compression to the centroid of the tension steel at a distance *d*. Dowel action of the longitudinal steel is ignored and forces are summed in the direction of shear reinforcement.

For shear walls without shear reinforcement and shear force acting parallel to wall, actual depth of masonry in direction of shear can be used in place of effective depth.

In reinforced shear walls, shear reinforcement is usually horizontal and the use of actual depth is more justified than the effective depth, as the assumed 45 degree crack passes through more horizontal bars. However, this is not always conservative and in some cases the use of maximum reinforced length is more appropriate.

5.9.3 –

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

C5.9.3 –

This ensures that a potential shear crack (inclined crack) is crossed by at least one stirrup.

5.9.4 –

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

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

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5.10 – Reinforcement Detailing

5.10.1 – General

Reinforcement shall be located such that it acts compositely with the masonry and various ways in which it can be used in reinforced masonry are shown in Fig.2.

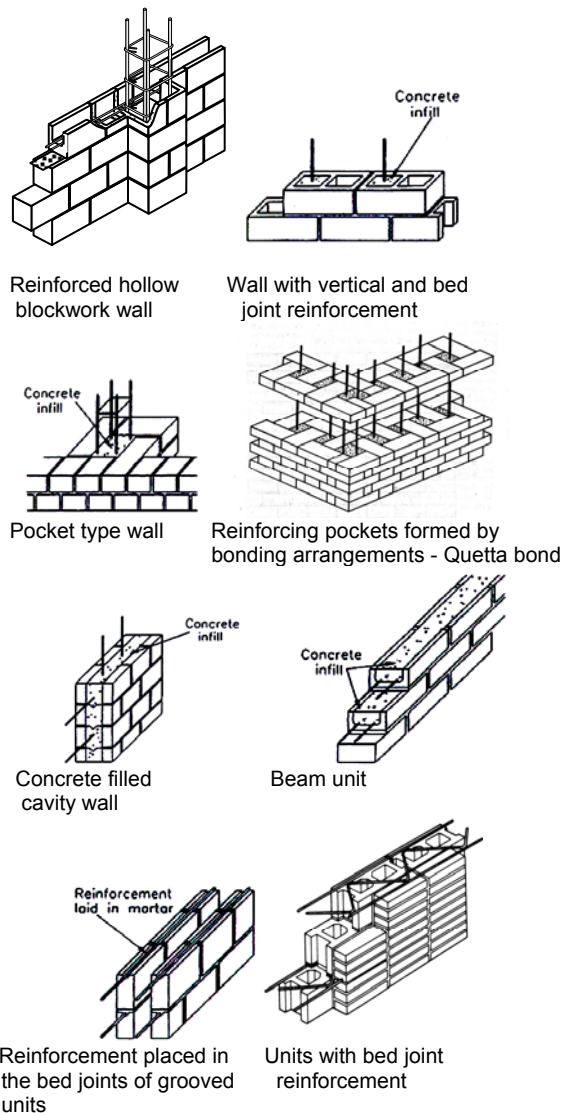


Figure 2: Reinforcement Details

5.10.2 – Protection of Reinforcement

C5.10.2 – Protection of Reinforcement

5.10.2.1 –

Where steel reinforcing bars are embedded in filled cavity (or pockets) or special bond construction, the bars shall have the minimum clear cover of 10 mm

C5.10.2.1 –

Traditionally, reinforcing bars are not galvanized. Corrosion of the steel is delayed by the masonry cover. Cover is measured from the exterior

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in mortar or a minimum clear cover 15 mm or bar diameter whichever is more in cement concrete (grout) so as to achieve good bond and corrosion resistance.

5.10.2.2 –

For the reinforcement steel placed in mortar bed joint, the minimum depth of mortar cover from the reinforcing steel to the face of masonry should be 15 mm as shown in Fig. 3. Also mortar cover above and below reinforcement placed in bed joints should not be less than 2 mm as shown in Fig 3.

Figure 3: Cover to Reinforcing Steel in Bed joints.

5.10.2.3 –

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

6. – Structural Design

6.1 – Permissible Compressive Force

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

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

where,

A_n = Net area

A_{st} = Area of steel

F_s = Permissible steel tensile stress

k_s = stress reduction factor as in Table 9 of IS:1905-1987.

6.1.1 – Effective Compressive Width for Locally Concentrated Reinforcement

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

- (a) centre-to-centre bar spacing, and
- (b) 6 times the wall thickness

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masonry surface to the outer-most surface of the steel. Masonry cover includes the thickness of masonry units, mortar and grout.

C5.10.2.2 –

Masonry cover available for the corrosion protection of joint reinforcement is much less compared to the reinforcing bars placed in masonry cores. Therefore, special attention is given for adequate cover for corrosion protection.

C6. – Structural Design

C6.1 – Permissible Compressive Force

The compressive force at the section from axial loads or from the axial component of combined loads is calculated separately as is limited to the permissible values. A coefficient of 0.25 provides a factor of safety of about 4.0 against crushing of masonry. The coefficient of 0.65 was determined from tests on masonry.

C6.1.1 – Effective Compressive Width for Locally Concentrated Reinforcement

Very limited data is available on the effective width of the compressive area for each reinforcing bar. The center to center spacing limit is to avoid overlapping areas of the compressive stress.

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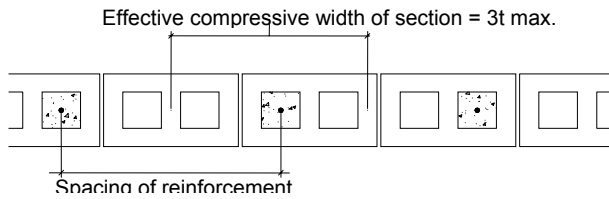


Figure 4: Effective compressive width

6.1.2 – Combined Permissible Axial and Flexural Compressive Stress

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

C6.1.2 – Combined Permissible Axial and Flexural Compressive Stress

The interaction equation specified for unreinforced masonry is not applicable for reinforced members. Interaction equations can be developed for reinforced sections, using principles of engineering mechanics for the composite behaviour of reinforced masonry and considering resistance of both uncracked and cracked state of the member section under axial and flexure stresses. It is an iterative procedure and one such method is included in Appendix A.

6.1.3 – Permissible Tensile Stress

Provisions of Section 5.4.2 of IS:1905-1987 shall apply.

6.1.4 - Permissible Shear Stress

6.1.4.1 For Reinforced masonry members

a) For Flexural members

i. Without Web Reinforcement

$$F_v = 0.083\sqrt{f_m} \text{ but not greater than } 0.25 \text{ MPa}$$

ii. With Web Reinforcement

$$F_v = 0.25\sqrt{f_m} \text{ but not greater than } 0.75 \text{ MPa}$$

b) For Walls

The allowable shear stress for reinforced masonry walls shall be according to Table 2.

C6.1.4 – Permissible Shear Stress

For reinforced masonry, the shear resistance is offered by both masonry and reinforcement, however, their effect is not taken additive in this document, like many other masonry codes. This has basis in an experimental research by Priestley and Bridgemen which concluded that the shear reinforcement in masonry is effective in providing resistance when it is designed to carry the full shear load. If calculated shear stress exceeds the allowable shear stress for masonry, full shear load has to be resisted by the reinforcement alone which is placed parallel to the applied shear force direction. The amount of shear reinforcement can be obtained as specified in Sec. 5.9.2.

	M/Vd	F_v , MPa	Maximum Allowable MPa
Without Web Reinforcement	<1	$\frac{1}{36}\left(4 - \frac{M}{Vd}\right)\sqrt{f_m}$	$\left(0.4 - 0.2 \frac{M}{Vd}\right)$
	>1	$0.083\sqrt{f_m}$	0.2
With Web Reinforcement	<1	$\frac{1}{24}\left(4 - \frac{M}{Vd}\right)\sqrt{f_m}$	$\left(0.6 - 0.2 \frac{M}{Vd}\right)$

Experiments have shown that shear resistance of the reinforced shear walls are function of the aspect ratio of the wall (i.e., ratio of height to length), besides distribution of reinforcement and masonry strength. Shear walls with lower aspect ratio have been provided with higher allowable shear stress. The aspect ratio of the wall is expressed by M/Vd term and is explained in Figure C5.

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Reinforcement	>1	$0.125\sqrt{f_m}$	0.4
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This M/Vd ratio is very much associated with the shear carrying capacity of the masonry walls, because change in this value controls the amount of flexural stress applied to the wall and finally coupled with the shear stress. If M/Vd ratio is small, then less amount of flexural stress will be associated with the shear stress, as a result the shear carrying capacity of the section will be increased. The experimental results also justify this observation. It shows that shear resistance increases with decrease in M/Vd ratio; however up to h/d of 1.0 the rate of increase is very small. But for values less than 1.0, rate of increase is almost proportional to the decrease in h/d ratio.

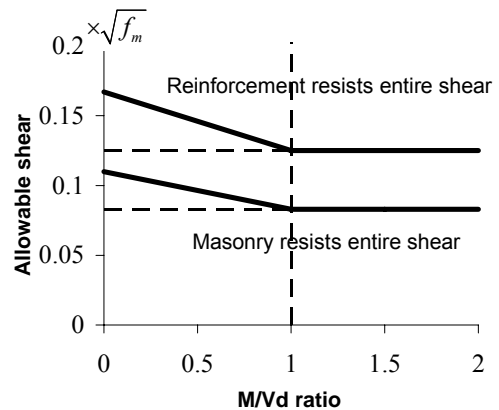


Figure C5: Allowable Shear for Reinforced Walls

A significant increase in shear strength is observed with introduction of web reinforcement. The difference between the shear resistances between the section having web reinforcement and section not having web reinforcement is more dominant when the M/Vd ratio is less. Like the previous case of no web reinforcement, with web reinforcement the shear resistance increases with decrease in aspect ratio.

6.1.5 –

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

C6.1.5 –

If there is tension in any part of a section of masonry, that part is likely to be cracked and thus cannot be depended upon for resisting any shear force. The clause is based on this consideration. This situation is likely to occur in masonry elements subjected to bending.

The shearing stress in reinforced masonry flexural member can be computed by the following formula:

$$f_v = V/(bjd)$$

or simply,

$$f_v = V/(bd) , \text{ which is an approximation of}$$

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maximum shear stress below neutral axis because the reinforced member will be cracked below neutral axis and therefore the classical maximum shear stress calculation that is valid for unreinforced, uncracked section required needs to be modified.

7. – Seismic Design Requirements

C7 – Seismic Design Requirements

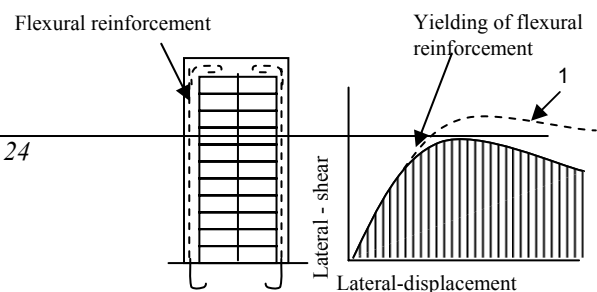
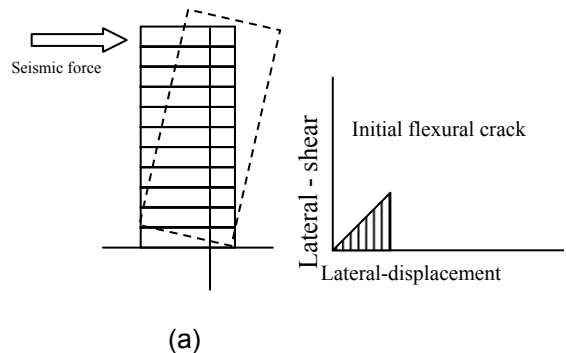
7.1 – Scope

The requirements of this section apply to the design and construction of reinforced masonry to improve its performance when subjected to earthquake loads. These provisions are in addition to general requirements of IS: 1893 (Part 1) – 2002.

C7.1 – Scope

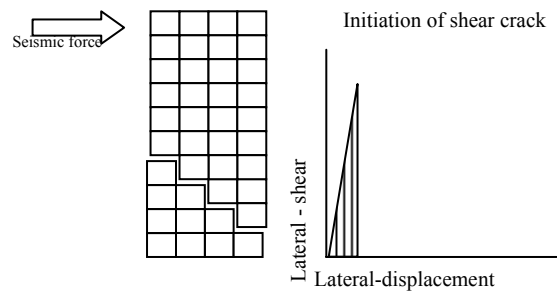
Bearing wall without reinforcement usually fail in brittle manner once flexural or shear cracking in the wall has initiated. Propagation of cracking and crack width can be controlled by providing wall reinforcement and wall can develop higher strength and ductility. This means that masonry walls can be designed and constructed so as to become earthquake resistant structural system, by using proper amount of reinforcement, enhancing its flexural as well as shear resistance as illustrated in Figure C6.

Use of reinforcement in bearing walls increases the energy absorption capacity of each wall significantly. An adequate amount of reinforcement significantly alters seismic behavior of masonry walls.

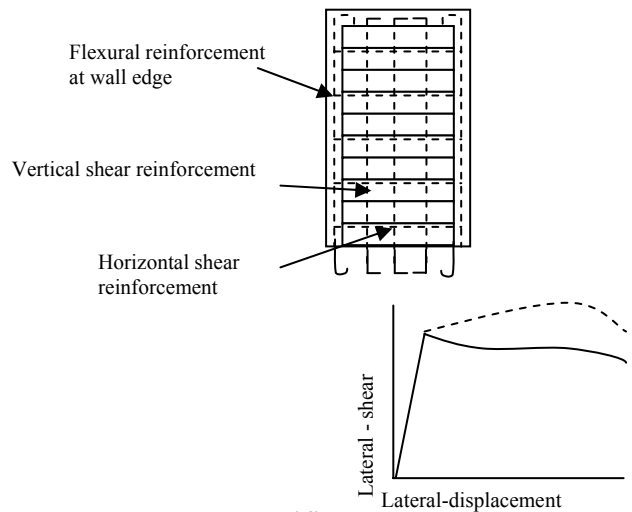


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(c)



(d)

Figure C6: Role of wall reinforcement in Bearing walls

7.2 – Different Performance Levels of Masonry Shear Walls

Masonry buildings rely on masonry shear walls for the lateral load resistance and can be detailed for

C7.2 – Different Performance Levels of Masonry Shear Walls

The lateral force resisting system in most masonry buildings are shear walls, however, sometimes concrete or steel frames with masonry infill are

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the following three levels of seismic performance, which can be appropriately chosen for a building considering its importance, location and acceptable degree of damage, etc. Table three summarizes the requirement for these shear walls and recommended R values for use with IS:1893.

7.2.1 – Detailed Unreinforced Masonry Shear Walls (Type A wall)

Design of detailed unreinforced masonry shear walls shall comply with requirements of unreinforced masonry wall of the suggested draft IS:1905 and shall comply with the following requirements:

7.2.1.1 – Minimum Reinforcement Requirements

Vertical reinforcement of at least 100 mm^2 in cross-sectional area shall be provided at a maximum spacing of 3 m on center at critical sections:

- a) Corners
- b) Within 400 mm of each side of openings,
- c) Within 200 mm of the end of the walls

7.2.1.2 –

Reinforcement around openings need not be provided for opening smaller than 400 mm in either direction.

7.2.1.3 –

Horizontal reinforcement shall consist of at least two

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also used. In this document, three types of shear walls are defined, which differ in their capacity to resist earthquakes by inelastic behavior and energy dissipation.

As a minimum, shear walls shall meet the requirements of ordinary unreinforced masonry shear walls and for acceptable response in higher seismic regions should be designed as detailed unreinforced, ordinary reinforced or special reinforced masonry shear wall with the increasingly improved seismic response. Ordinary unreinforced shear wall with no reinforcement and R value 1.5 is generally recommended for seismic zone II.

C7.2.1 – Detailed Unreinforced Masonry Shear Walls

These shear walls are designed as unreinforced masonry but contain minimum reinforcement in the horizontal and vertical directions. Because of this reinforcement, these walls have more improved inelastic response and energy dissipation potential. As a result, the seismic design parameters, including structure response factors are more favorable than ordinary unreinforced masonry shear walls. Because of presence of minimum reinforcement, these walls can accommodate larger deformation than similarly configured unreinforced masonry shear walls. These walls can be used from low to moderate seismic risk regions (i.e., Zone II and III).

C7.2.1.1 – Minimum Reinforcement Requirements

The requirement of minimum amount of reinforcement is based on judgment which has been also confirmed by experimental investigations wherein they have shown to perform adequately in resisting seismic loads. These reinforcements are schematically shown in Figure C7.

C7.2.1.2 –

In case of smaller openings, the development of diagonal strut for lateral load resistance is not affected as much as in a larger opening. As a result, the provision of reinforcement can be relaxed for smaller openings.

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

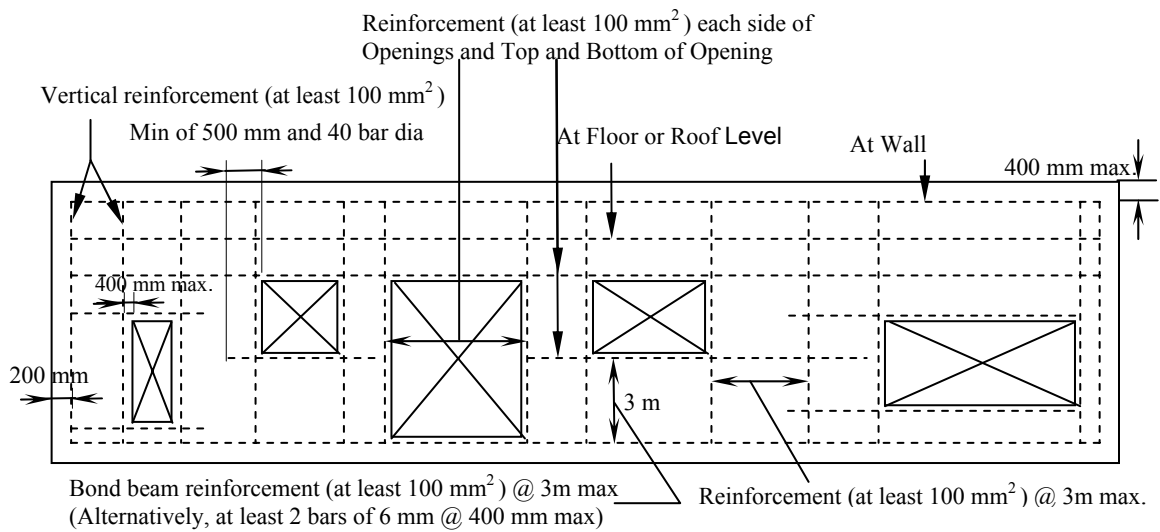


Figure C7 : Provision of minimum reinforcement as per clause 7.2.1.1, 7.2.1.2 and 7.2.1.3

Table 3 : Reinforcement and R values for different wall types in various seismic zones

Type of wall	Description	Reinforcement	Seismic Zone	R value
A	URM with minimum reinforcement	As per clause 7.2.1.1, 7.2.1.2, 7.2.1.3	II & III	2.25
B	RM with minimum reinforcement	As per clause 7.2.1.1, 7.2.1.2, 7.2.1.3 and requirements of ordinary Reinforced Masonry	IV & V	3.0
C	RM with minimum reinforcement and special reinforcement	As per clause 7.2.1.1, 7.2.1.2, 7.2.1.3 and requirements of special Reinforced Masonry	IV & V	4.0

* R value is given as per provision of IS:1893

7.2.2 – Ordinary Reinforced Masonry Shear Wall (Type B wall)

Design of ordinary reinforced masonry shear walls

C7.2.2 – Ordinary Reinforced Masonry Shear Wall

These shear walls are required to meet minimum

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shall comply with requirements of reinforced masonry wall as outlined in this document and shall comply with the requirements of section 7.2.1.1, 7.2.1.2 and 7.2.1.3.

7.2.3 – Special Reinforced Masonry Shear Wall (Type C wall)

Design of special reinforced masonry shear wall shall comply with the requirement of reinforced masonry as outlined in this document and the requirements of section 7.2.1.1, 7.2.1.2 and 7.2.1.3; and the following:

7.2.3.1 –

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

7.2.3.2 –

Maximum spacing of horizontal and vertical reinforcement shall be lesser of:

- a) one-third length of shear wall
- b) one-third height of shear wall
- c) 1.2 m.

7.2.3.3 –

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

7.2.3.4 –

Shear reinforcement shall be anchored around

COMMENTARY

requirements for reinforced masonry as noted in the referenced sections. As noted earlier, because of presence of steel reinforcement, they can be subjected to large inelastic deformation and loss of strength and stability of system and thus dissipating less amount of energy. Their expected performance is somewhat poorer than special reinforced masonry walls discussed in section 7.2.3. However, they can provide adequate safety in moderate to high seismic risk zone.

C7.2.3 – Special Reinforced Masonry Shear Wall

These shear walls are designed as reinforced masonry and are required to meet much restrictive reinforcement requirements, because these special reinforced masonry walls are meant to meet the seismic demands in regions of intense seismic activity and increased risk such as zone IV and V.

C7.2.3.1 –

This requirement of minimum amount of steel reinforcement is provided to improve the energy dissipation potential of the wall, and it has been in use in high seismic areas as a standard empirical requirement.

The minimum requirement of reinforcement are expressed as percentage of gross-sectional area and at least one-third of 0.2% of steel reinforcement in vertical direction is provided in horizontal direction to ensure better distribution of stress.

C7.2.3.3 –

The minimum requirement of reinforcement is meant for an appropriate distribution of strength in both direction and at least one-third of the minimum is required in vertical direction.

PROVISIONS

vertical reinforcing bars with a 135 or 180 degree standard hook.

COMMENTARY

Appendix A

(Provision 6.1.2)

TYPICAL ITERATIVE METHOD FOR WALLS SUBJECTED TO FLEXURE AND AXIAL LOAD

(Adapted from Masonry Designer's Guide)

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

Procedure 1

Figure A1 shows a diagram of a wall subjected to an axial load and bending moment. The same diagram is applicable to both in-plane and out-of-plane bending of walls. There are three possible conditions for the wall:

1. The wall is uncracked,
2. The wall is cracked with the steel in compression (the extent of the crack has not reached the steel), or
3. The wall is cracked with the steel in tension.

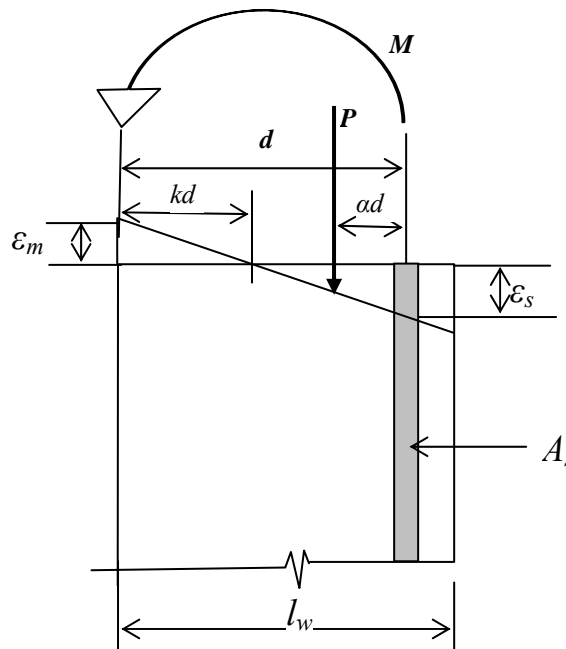


Figure A1: Flexure and Axial Wall Loading

Figure A2 shows the interaction between bending moment and axial load. Loading conditions beyond the limits of the diagram are beyond the allowable stresses. The condition of the wall can be quickly diagnosed by using the non-dimensional parameter M/Pd . This parameter represents a straight line radiating from the diagram's origin as shown in Figure A2. By summing moments, it can be shown that certain values of M/Pd divide the diagram into three regions. These values are given in Table A1. The allowable moments for Regions 1 and 2 can be obtained in closed form with simple equations. But in Region 3, it is complicated a bit. The process consists of initial assumption of thickness, length and other necessary parameters and determination of the amount of steel required.

For Region 3, the process starts with another assumption that the tension in steel controls. By making an initial guess about the location of the neutral axis, Equation A3 and A6 provide an iterative process that quickly converges. The equations are derived from the summation of forces and moments and by using linear stress-strain relationships. The equations should always converge for all values of P , l_w , M , and n . If convergence is not obtained, it is likely that the assumed initial value of a has exceeded d . The initial value of a is to be reduced and same procedure is to be repeated. The iteration will often result in a negative steel area. This signifies that the element does not need reinforcement to resist the loads within the allowable stresses, even though the section is cracked (Region 3).

Following convergence, a check is made of the initial assumption that tension in steel controls. If not valid, then the compression controls and a new set of equations that do not require iteration are used (Eqn. A8 and A9).

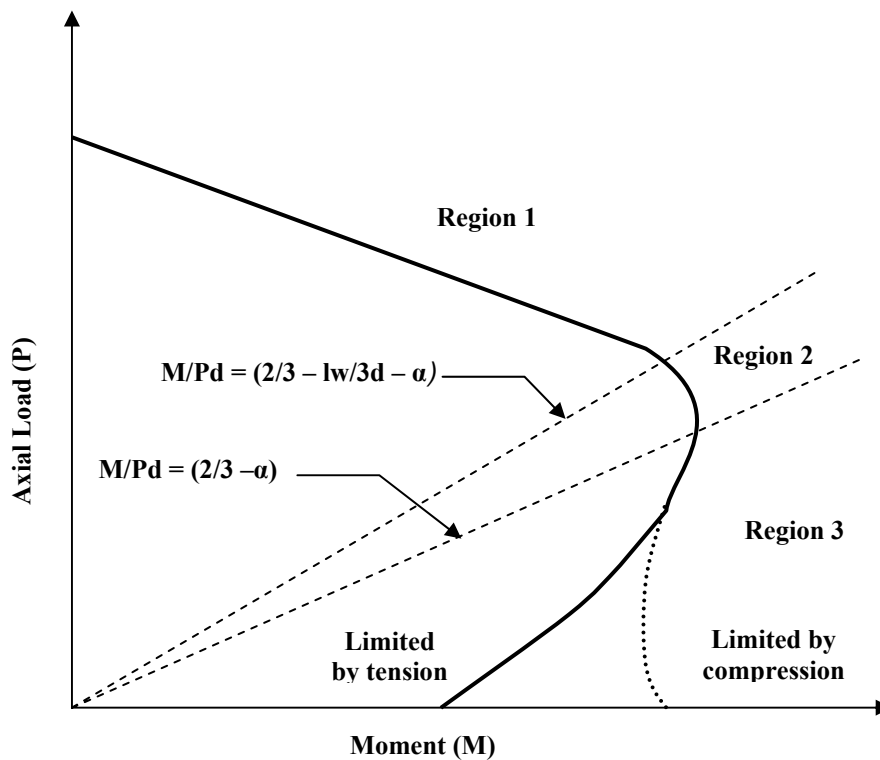
The following is a detailed step-by-step procedure of the analysis methodology described above.

Step 1:

Determine the wall condition:

1. Calculate α as the distance from the axial load to the centroid of the tension steel divided by d .
2. Calculate the quantity M/Pd .
3. Use Table A1 to determine the region for analysis. (Fig.A2)

Table A1: Flexure and Axial loading – Wall Analysis		
Region	Condition Of Wall	Test
1	Wall is in compression and not cracked	$\frac{M}{Pd} \leq \left[1 - \frac{l_w}{3d} - \alpha \right]$
2	Wall is cracked but steel is in compression	$\frac{M}{Pd} \leq \left[\frac{2}{3} - \alpha \right]$
3	Wall is cracked and steel is in tension	$\frac{M}{Pd} > \left[\frac{2}{3} - \alpha \right]$



FigureA2: Interaction diagram

Step 2:

Calculate the allowable moment.

REGION 1:

The moment is limited by flexural compression in the masonry.

$$M_m = \frac{bl_w^2}{6} F_b - P \frac{l_w}{6} \tag{A1}$$

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

REGION 2:

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

$$M_m = P(l - \alpha)d - \frac{2}{3} \left[\frac{P^2}{F_b b} \right] \quad (A2)$$

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

REGION 3:

The moment may be limited by either the compression in the masonry or tension in the steel. An iterative approach is required.

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

$$M_p = P \left(\frac{l_w}{2} - a \right) \quad (A3)$$

$$A_s = \frac{M - M_p}{F_s (d - a)} \quad (A4)$$

$$\zeta = \frac{(P + A_s F_s) n}{F_s b} \quad (A5)$$

$$a_2 = \frac{\sqrt{\zeta^2 + 2\zeta d} - \zeta}{3} \quad (A6)$$

Where:

- a Estimate of internal compression load centroid distance from the extreme compression fiber.
- P Applied axial load at the center of the wall
- M Applied moment
- A_s Area of trim or edge steel
- M_p Moment of applied axial load with respect to the centroid of internal compression force
- M_p Width of the wall for in-plane bending (thickness of wall for out of plane bending)
- l_w Distance from the extreme compression fiber to the steel centroid
- d Allowable steel tension stress
- F_s Width of the wall for out-of-plane bending (thickness of wall for in-plane bending)
- b

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

Step 3:

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

$$a = \frac{d}{3 \left(1 + \frac{F_s / F_b}{n} \right)} \quad (A7)$$

Step 4:

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

$$a = \frac{d}{2} - \sqrt{\frac{d^2}{4} - \frac{2(P\alpha d + M)}{3F_b b}} \quad (A8)$$

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

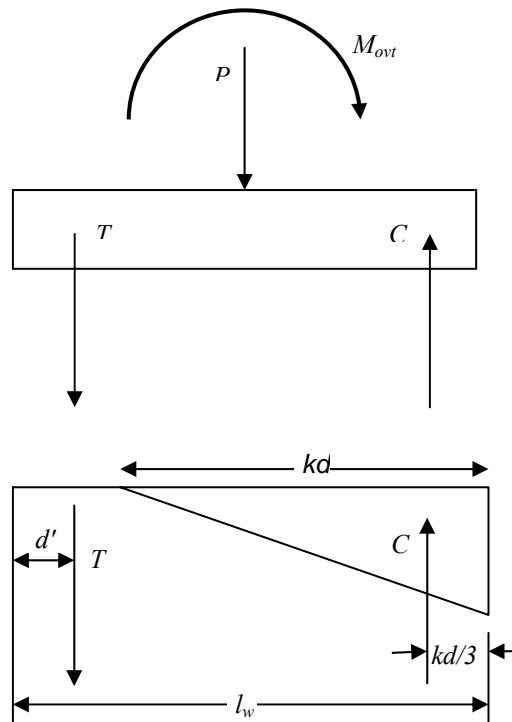
Increase b or F_b or both. The steel area is:

$$A_s = \frac{\left(\frac{3F_b ab}{2} - P\right)}{\left[nF_b \left(\frac{d}{3a} - 1\right)\right]} \quad (A9)$$

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

Procedure 2

The following is another procedure to determine the state of stress in wall elements subjected to combined action of bending and axial compression. Free body diagram of a wall with axial load, bending moment, and tensile and compressive forces is given in Fig.A3. The location of C is based on the neutral axis location kd . The location of T is at the centroid of the reinforcement.



FigureA3: Free body diagram of a wall

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

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

$$C \left[(l_w - d') - \left(\frac{kd}{3}\right) \right] = P \left(\frac{l_w}{2} - d' \right) + M \quad (A10)$$

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

$$\frac{1}{2}F_m bkd \left(d - \frac{kd}{3} \right) = P \left(\frac{l_w}{2} - d' \right) + M \quad (A11)$$

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

$$kd^2 - \frac{(kd)^2}{3} = \frac{P\left(\frac{l_w}{2} - d'\right) + M}{\frac{1}{2}F_m b} \quad (A12)$$

Multiplying by -3 and rearranging terms:

$$(kd)^2 - 3d(kd) + \frac{3\left[P\left(\frac{l_w}{2} - d'\right) + M\right]}{\frac{1}{2}F_m b} = 0 \quad (A13)$$

$$kd = \frac{3d - \sqrt{(-3d)^2 - 4\left[\frac{3\left[P\left(\frac{l_w}{2} - d'\right) + M\right]}{\frac{1}{2}F_m b}\right]}}{2} \quad (A14)$$

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

Having solved for kd one can calculate C .

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

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

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

Having solved for kd one can calculate C .

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

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

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

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

$$A_s = T/F_s$$

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

$$(A_s)_{eff} = A_s + \frac{P}{F_s} \quad \left(\text{or } \frac{T+P}{F_s}\right) \quad (A15)$$

$$\rho_{eff} = \frac{(A_s)_{eff}}{bd} \quad (A16)$$

$$k = \left[(n\rho)^2 + 2n\rho \right]^{\frac{1}{2}} - n\rho \quad (A17)$$

$$j = 1 - \frac{k}{3} \quad (A18)$$

The steel stress may be checked using

$$f_s = \frac{M'}{(A_s)_{eff} jd} \quad (A19)$$

where,

$$M' = P \left(\frac{l_w}{2} - d' \right) + M \quad (\text{A20})$$

$$T_{eff} = (A_s)_{eff} \times f_s \quad (\text{A21})$$

$$T = T_{eff} - P \quad (\text{A22})$$

The masonry stress may be checked by using

$$f_m = \frac{2M'}{bjkd^2} \quad (\text{A23})$$

If $f_s > F_s$, a second iteration may be required with a larger bar size or an extra bar. In a shear wall if an extra bar is added, the value of d' , and the moments $P \left(\frac{l_w}{2} - d' \right) + M$ will need to be re-evaluated. Then

one can reiterate from $(A_s)_{eff} = A_s + \frac{P}{F_s}$ to reduce f_s so that it is less than F_s .

Appendix B

(Provision 7.1)

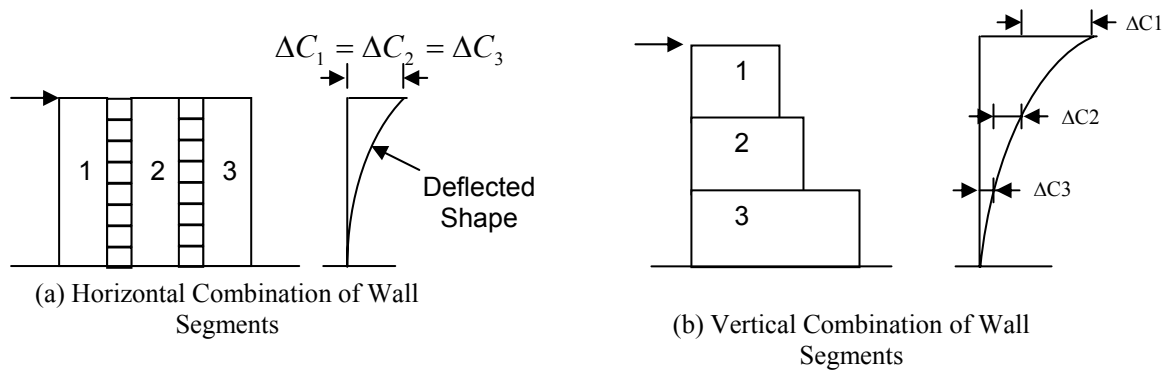
DISTRIBUTION OF LATERAL FORCES IN THE PIERS OF A MASONRY WALL

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

In any kind of placing of opening the wall can be represented as a horizontal and vertical combination of piers with their respective end condition which will be used to find out their rigidities. Where large openings occur, it is difficult to obtain effective coupling of the wall segments or piers.

If the wall is analyzed as a horizontal combination of piers as shown in the figure B1 (a) and the combined rigidity will be $R = R_{c1} + R_{c2} + R_{c3}$ (B1)

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



FigureB1: Wall Combinations for calculating rigidities of walls with openings

If the segments are combined vertically, as shown in *figureB1(b)*, the combined rigidity can be calculated as

$$R = \frac{1}{\Delta C_1 + \Delta C_2 + \Delta C_3} = \frac{1}{\frac{1}{R_{c1}} + \frac{1}{R_{c2}} + \frac{1}{R_{c3}}} \quad (B2)$$

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

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

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

$$\Delta_c = \frac{Fh^3}{3EI} + \frac{1.2Fh}{GA} \quad (B3)$$

Where, F = Horizontal force applied to the pier

h = Height of the pier

$$I = \text{moment of inertia of pier} = \frac{tL^3}{12}$$

G = Modulus of Rigidity = $0.4E_m$

A = Area of the pier = Lt

L = Length of the pier

t = thickness of pier

$$\text{So displacement of the cantilever pier is } \Delta_c = \frac{1}{Et} \left[4 \left(\frac{h}{L} \right)^3 + 3 \left(\frac{h}{L} \right) \right] \quad (\text{B4})$$

The rigidity of a wall is proportional to the inverse of the deflection.

For cantilever walls the rigidity will be,

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

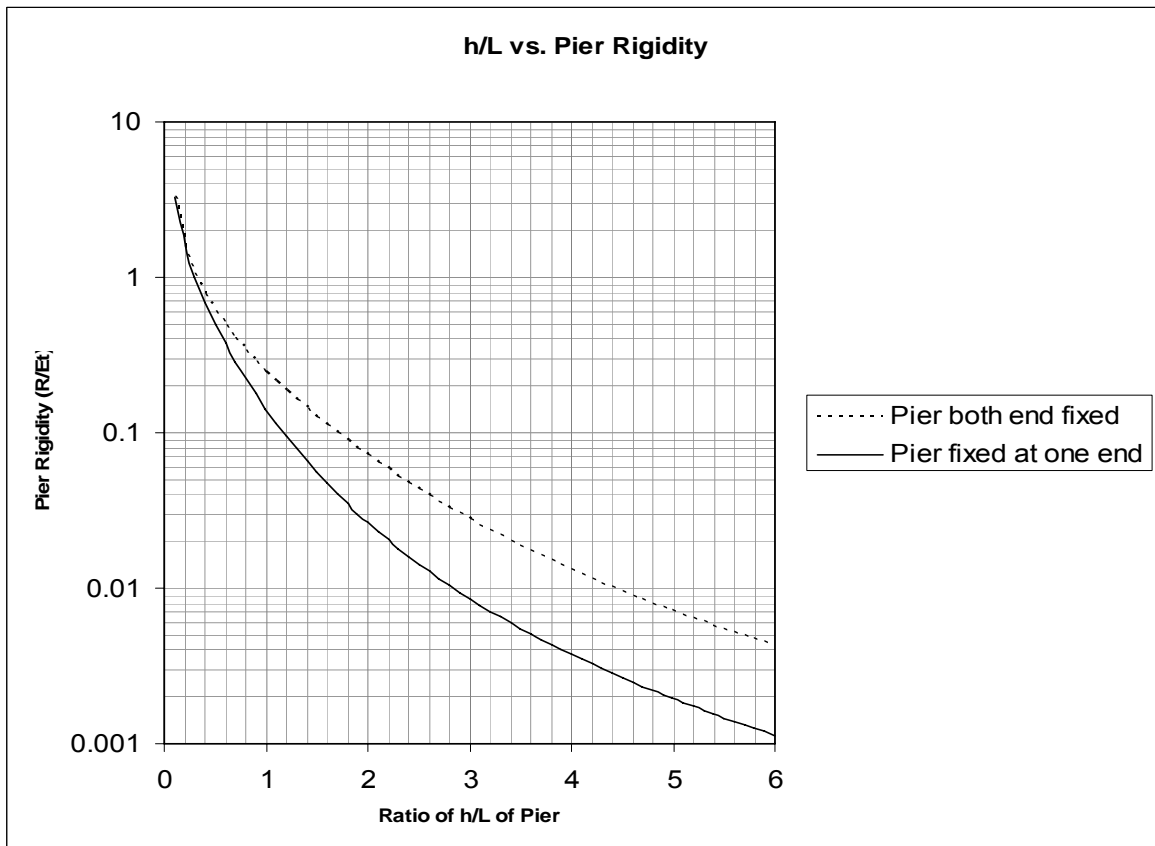
$$\Rightarrow \frac{R_c}{Et} = \frac{1}{\left[4 \left(\frac{h}{L} \right)^3 + 3 \left(\frac{h}{L} \right) \right]} \quad (\text{B5})$$

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

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

$$\Rightarrow \frac{R_f}{Et} = \frac{1}{\left[\left(\frac{h}{L} \right)^3 + 3 \left(\frac{h}{L} \right) \right]} \dots (\text{B6})$$

Values of $\frac{R}{Et}$ for different h/L ratio can be taken from the chart in the *figureB2*



FigureB2: Charts for calculating wall rigidities

From the relations given in equation (Eqn.B5) and (Eqn.B6) it is found that the relative contributions of the bending and shear deformation depend on the wall aspect ratio (h/L) and therefore, the rigidity varies over the height of the building. For high h/L ratios, the effect of shear deformation is very small and calculation of pier rigidities based on flexural stiffness is relatively accurate. For very squat walls (with $h/L < 0.25$), rigidities based on shear deformation are reasonably accurate, but for intermediate walls with h/L from 0.25 to 4, both components of relative rigidity should be considered.

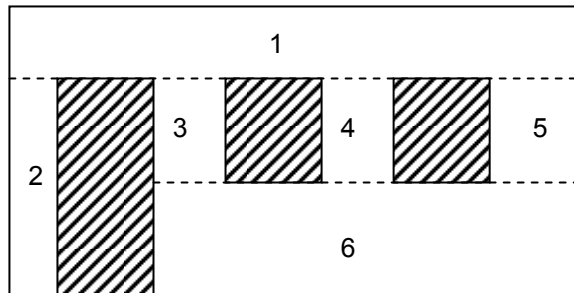
The method is explained further explained by one illustration. For the wall as shown in Fig.B3,

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

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

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

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



FigureB3: Wall with openings (calculation of rigidity and load distribution)

Appendix C

NOTATIONS, SYMBOLS AND ABBREVIATIONS

C-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
d	Effective depth
d_b	Nominal diameter of bar (mm)
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
$H1, H2$	High strength mortars
k_s	Stress reduction factor
L	Actual length of wall
L_d	Development length
P	Total horizontal load
s	Spacing of shear reinforcement
SR	Slenderness ratio
t	Actual thickness
V	Total applied shear force

***IITK-GSDMA* GUIDELINES**

for **STRUCTURAL USE**

of **REINFORCED MASONRY**

Provisions with Commentary and Explanatory Examples

PART 2: EXPLANATORY EXAMPLES

Example 1 – DESIGN OF REINFORCED MASONRY SHEAR WALL

Problem Statement:

Design the shear wall of a hotel building in Seismic Zone V shown in Figure 1.1 for seismic loads using reinforced single wythe hollow clay masonry units. Roof and floor construction consists of 200 mm precast hollow planks with 50 mm thick normal weight concrete topping.

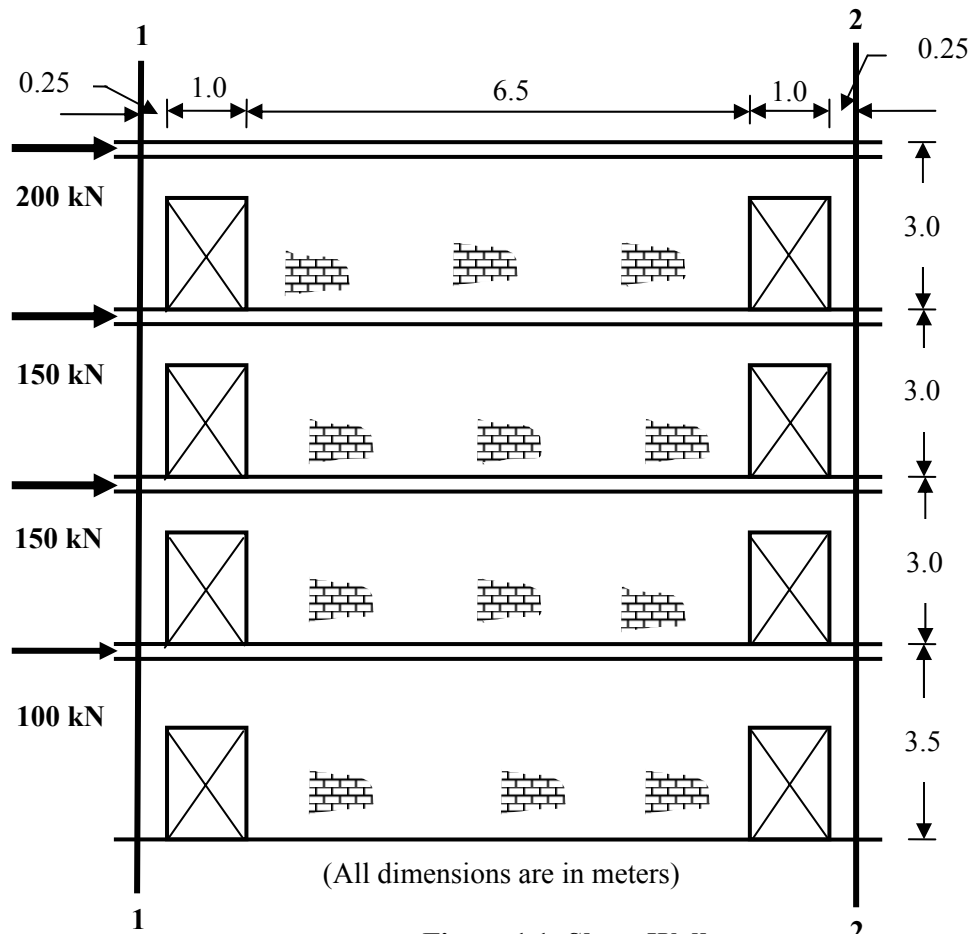


Figure 1.1: Shear Wall

Solution:

Design Data/ Assumptions:

Prism strength of masonry, $f_m = 15$ MPa

Permissible tensile stress of steel bars,

$$f_s = 230 \text{ MPa}$$

Young's Modulus of steel,

$$E_s = 2 \times 10^5 \text{ MPa}$$

Width of roof under consideration = 1.2 m

Width of corridor = 1.2 m

Dead Load:

Roof load = 4000 N/m²

Corridor Load = 5000 N/m²

Wall Load = 3000 N/m²

Live Load:

Roof Load = 950 N/m²

Floor Load = 4500 N/m²

Calculation of Loads:

Axial loads

Dead Loads per meter run of the wall:

$$\text{Roof} \quad 4000 \times 1.2 = 4800 \text{ N}$$

$$\text{Corridor} \quad 5000 \times 1.2 = 6000 \text{ N}$$

$$\text{Wall} \quad 3000 \times 3.0 = 9000 \text{ N}$$

$$3000 \times 3.5 = 10500 \text{ N}$$

Tributary length for the gravity loads on shear wall,

$$L_r = 6.5 + 1.0 = 7.5 \text{ m}$$

Length of wall, $L_w = 6.5 \text{ m}$

Total dead load at different floor levels are calculated in Table 1.1.

Table 1.1: Calculation of dead load

Element	Dead Load (kN)	Cumulative Dead Load (kN)
Roof	36.0	36.0
Wall	58.5	94.5
4 th	45.0	139.5
Wall	58.5	198.0
3 rd	45.0	243.0
Wall	58.5	301.5
2 nd	45.0	346.5
Wall	68.3	414.8

Live Loads

$$\text{Roof} \quad 950 \times 1.2 = 1.14 \text{ kN/m}$$

$$\text{Floor} \quad 4500 \times 1.2 = 5.4 \text{ kN/m}$$

Total live loads at different floor levels are shown in Table 1.2.

Table 1.2: Calculation of live load

Element	Live Load (kN)	Cumulative Live Load (kN)
Roof	8.5	8.5
4 th	40.5	49.0
3 rd	40.5	89.5
2 nd	40.5	130.1

Lateral loads are as shown in Figure 1.2.

Distribution of shear force and bending moment over the height of the building is shown in Figure. 1.2 and 1.3, respectively.

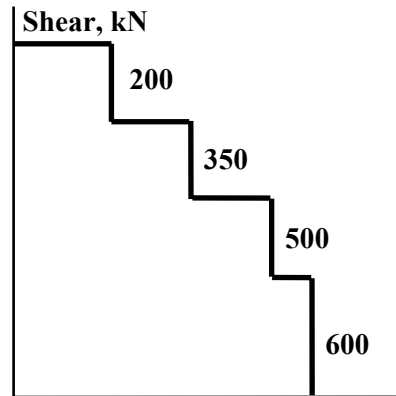


Figure 1.2: Shear at different floor levels

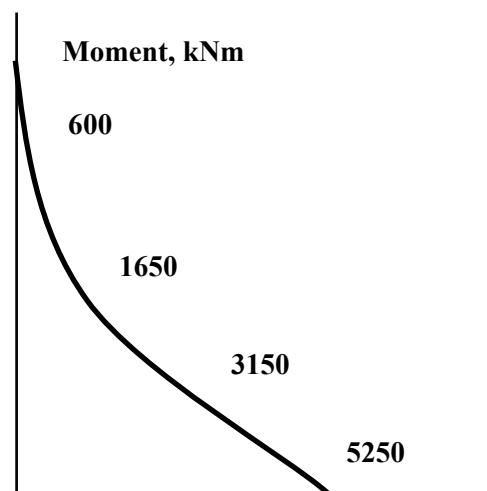


Figure 1.3: Bending moment at different floor levels

Flexure Consideration:

The vertical steel bars are placed at both ends of a shear wall.

From inspection, load combination $0.75(D+E)$ controls the design.

First Floor

$$P = 0.75D = 0.75 \times 414.8 \text{ kN} = 311.1 \text{ kN}$$

$$M = 0.75 \times 5250 \text{ kNm} = 3938 \text{ kNm}$$

Let's use 200 mm thick wall.

$$\text{Effective thickness} \quad b = 0.19 \text{ m}$$

$$\text{Area of masonry unit, } A_n = 0.19 \text{ m}^2$$

$$\text{Radius of gyration, } r = 0.055 \text{ m}$$

Assume 6 bars of 25 mm dia @150 mm c/c with a cover of 75 mm.

Area of steel provided, $A_s = 471 \text{ mm}^2$

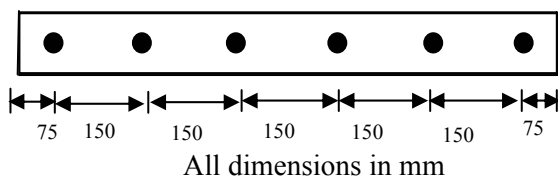


Figure 1.4: Steel bars along shear wall

First Estimate:

Effective depth of masonry,

$$d = 6.5 - 0.45 \text{ m} = 6.05 \text{ m}$$

Assume masonry stresses control.

Taking $\sum M$ about centroid of tension steel as shown in figure 1.4

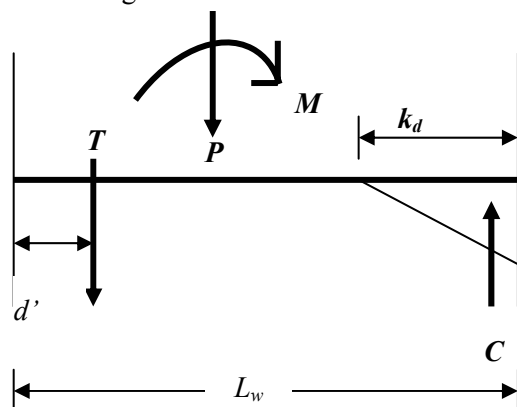


Figure 1.5: Forces acting on the wall

Bending stress in masonry,

$$f_b = 1.25 \times 0.25 \times f_m = 4.7 \text{ MPa}$$

Total compressive force in the section is given by

$$C = \frac{1}{2} f_b b k d = \frac{1}{2} \times 4.7 \times 10^6 \times 0.19 \times k \times 6.05$$

$$= 2.7 \times 10^6 k$$

Since sum of all moments acting on the section is zero, i.e., $\sum M = 0$

$$C \times \left(d - \frac{k d}{3} \right) - P \times \left(\frac{L_w}{2} - d' \right) - M = 0$$

$$2.7 \times 10^6 \times k \times \left(6.05 - \frac{6.05 \times k}{3} \right)$$

$$- 311.1 \times 10^3 \times \left(\frac{6.5}{2} - 0.45 \right) - 3938 \times 10^3 = 0$$

Solving the above equation, we get, $k = 0.331$

Putting the value of k ,

Total compressive force, $C = 894.1 \text{ kN}$

Total tensile force,

$$T = C - P = 894.1 - 311.1 = 583.0 \text{ kN}$$

Young's modulus of masonry is given by

$$E_m = 550 \times f_m = 8250 \text{ MPa}$$

(As per Section 3.4.2)

Modular ratio is given by

$$n = \frac{E_s}{E_m} = 24$$

From strain compatibility, stress in steel is given by

$$f_s = \frac{(1-k) \times n \times f_b}{k} = 228 \text{ MPa} (< 230 \text{ MPa})$$

Hence, masonry stress controls.

Effective tensile force is given by

$$T_{eff} = 228 \times 471 \text{ N} = 107.4 \text{ kN}$$

$$C_1 = T_1 + P = 107.4 + 311.1 \text{ kN} = 418.5 \text{ kN}$$

$$f_{b1} = \frac{2 \times 418.5 \times 10^{-3}}{0.19 \times 6.05 \times 0.331} = 2.2 \text{ MPa} (< f_b) < 4.7 \text{ MPa}$$

Since the calculated stress in masonry is less than the allowable stress, the design is OK for flexure.

Shear Consideration:

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.

Shear force, $V = 0.75 \times 600 \text{ kN} = 450 \text{ kN}$

Shear stress is given by

$$f_v = \frac{V}{b \times d} = \frac{450 \times 10^3}{0.19 \times 6.05} = 0.391 \text{ MPa}$$

Providing web shear reinforcement

$$\frac{M}{V \times d} = 1.085 (> 1.0) \text{ (As per Section 6.1.4)}$$

Allowable and maximum shear stress is given by

$$F_v = 0.125 \times \sqrt{f_m} = 0.125 \times \sqrt{15} = 0.484 \text{ MPa}$$

$$F_{v \max} = 0.4 \text{ MPa}$$

Hence, allowable shear stress is 0.4 MPa. Since, calculated shear stress is less than the allowable shear stress, the design is safe in shear with web reinforcement.

Design of shear reinforcement:

Provide 8 mm dia. steel bars. (say)

Maximum spacing of bars in both horizontal and vertical direction shall be lesser of following:

- a) one-third of length of shear wall = 2.2 m
- b) one third of height of shear wall = 1.0 m
- c) 1.2 m

(As per section 7.2.3.1)

Provide 8 mm dia. bars @1.0 m c/c.

Percentage of shear reinforcement provided

$$= \frac{50}{1000 \times 190} \times 100 = 0.03 \%$$

Minimum shear reinforcement area in each

direction shall not be less than 0.07% of gross cross sectional area.

(As per section 7.2.3.1)

Provide 10 mm dia. bars @500 mm c/c in horizontal directions.

Percentage of steel in horizontal direction

$$= \frac{78}{500 \times 190} \times 100 = 0.08\%$$

Percentage of steel in vertical direction

$$= \frac{78}{190 \times 150} \times 100 = 0.27\%$$

Sum of percentage of steel in both direction

$$= 0.08 + 0.27 = 0.35\%$$

As per Sec 7.2.3.1, sum of reinforcement area in both directions shall be at least 0.2% of gross area. Hence, the design is OK.

Example 2 – DESIGN OF REINFORCED SHEAR WALL FOR IN-PLANE FLEXURE AND SHEAR

Problem Statement:

Design the shear wall of 6.5 m length using 200 mm hollow concrete masonry units (CMU) for unreinforced and reinforced option with the following building geometry, material and loading data.

Total height of building: 6.0 m

Roof height: $h = 5.5$ m

Length of shear wall, $l_w = 6.5$ m

Live load = 23.12 kN

Bedded width, $b_b = 6$ cm

Prism strength of masonry: $f_m = 10$ MPa

Grade of steel: HYSD $f_{st} = 230$ MPa

Grade of mortar: M2

Compressive strength of masonry units: 15 MPa

Unit weight of masonry: 20 kN/m^3

Axial load from beam on wall: 48.65 kN

Lateral seismic load causing in-plane flexure in the wall are 55 kN (reinforced option) and 146 kN (unreinforced option).

Solution :

Unit weight of masonry assuming 50% hollow area = $20 \times 0.2 \times 0.5 \text{ kN/m}^2 = 2 \text{ kN/m}^2$

- i) $0.75(0.9DL+1.0E)$, i.e.,
 $0.67DL+0.75E$
- ii) $0.75(DL+1.0E+LL)$

Flexural Design Considerations:

Bending moment at base of wall:

Reinforced option:

$$M_{cr} = V_{cr} \times h = 55 \times 5.5 \text{ kN-m} = 302.5 \text{ kN-m}$$

Unreinforced option:

$$M_{cur} = V_{cur} \times h = 146 \times 5.5 \text{ kN-m} = 803 \text{ kN-m}$$

Beam load on wall: $DL_1 = 48.65 \text{ kN}$

Self weight on wall:

$$DL_s = 2 \times 6.5 \times 5.5 \text{ kN} = 71.5 \text{ kN}$$

Total dead load:

$$DL = DL_1 + DL_s = 48.65 + 71.5 \text{ kN} = 120.15 \text{ kN}$$

Live load: $LL = 93.12 \text{ kN}$

In view of wind/earthquake load, permissible stress level should be increased by 33% which is equivalent to reducing load level to 75% with 100% permissible stress level.

Following load combinations are adopted in this example.

Load cases:

Unreinforced concrete masonry:

First Case

Axial Load:

$$P_1 = 0.67 \times DL = 0.67 \times 120 = 80.5 \text{ kN (Governs)}$$

Bending Moment: $M_1 = 803 \text{ kNm}$

Second Case

Axial Load:

$$P_2 = 0.75 \times DL + 0.75 \times LL = 0.75(120 + 93.12) = 160 \text{ kN}$$

Bending Moment: $M_2 = M_{cur} = 803 \text{ kN}$

Bedded width: $b_b = 6$ cm

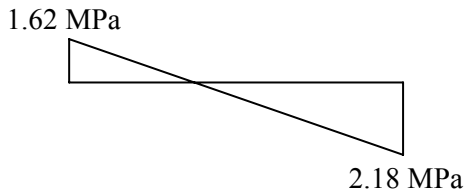
Net bedded area: $A = b_b \times b = 0.39 \text{ m}^2$

Moment of inertia, $I = 1.373 \text{ m}^4$

Computed stresses are given by

$$f_1 = \frac{-P_1}{A} + \frac{M_1 \times 6.5}{2 \times I} = 1.62 \text{ MPa (Tension)}$$

$$f_2 = \frac{-P_2}{A} - \frac{M_1 \times 6.5}{2 \times I} = -2.18 \text{ MPa (Comp.)}$$


Figure 2.1: Stress Distribution

No tension is allowed for in-plane loading by the code for un-reinforced shear wall. Hence, the wall must be reinforced.

Reinforced Concrete Masonry:

Allowable compressive stress in masonry:

$$F_a = 0.25 \times f_m$$

$$= 0.25 \times 10 \text{ MPa} = 2.5 \text{ MPa}$$

Allowable bending stress in masonry,

$$F_b = 1.25 \times 2.5 \text{ MPa}$$

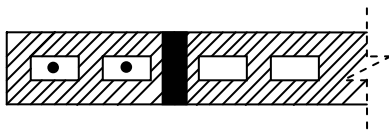
$$= 3.13 \text{ MPa}$$

Allowable tensile stress in steel, $f_s = 230 \text{ MPa}$

Assume steel bars are located in last cells of wall at both ends. Distance between the center lines of bars from end of wall, $d_e = 0.20 \text{ m}$

Effective depth: $d_{eff} = 6.5 - d_e = 6.3 \text{ m}$

Thickness of CMU carrying compression: 6 cm


Figure 2.2: Wall Cross Section

First Case:

$$\frac{M_1}{P_1 d_{eff}} = 1.583$$

Elastic modulus of masonry is given by,

$$E_m = 550 \times f_m = 5.5 \times 10^3 \text{ MPa}$$

Elastic modulus of steel = $2.1 \times 10^5 \text{ Mpa}$

Modular ratio is given by, $m = 36$

Distance of axial load from edge of wall

$$\alpha = \frac{\frac{l_w}{2} - d_e}{d_{eff}}$$

$$= \frac{6.5}{2} - 0.2$$

$$= \frac{3.1}{6.3} = 0.484$$

$$\frac{2}{3} - \alpha = 0.183 < \frac{M_1}{P_1 d_{eff}} \quad (\text{Region 3 applies})$$

(Refer to Appendix on P - M interaction curve of masonry)

Assumed distance of compression centroid from edge,

$$a = 1.9 \text{ m (Assumed)}$$

First Iteration

$$M_p = P_1 \left(\frac{l_w}{2} - a \right) = 80.4 \left(\frac{6.5}{2} - 1.9 \right)$$

$$M_p = 108.7 \text{ kNm}$$

$$A_s = \frac{M_1 - M_p}{f_s (d_{eff} - a)}$$

$$= \frac{803 - 108.7}{230(6.3 - 1.9)} = 0.000686 \text{ m}^2$$

$$\zeta = \frac{(P_1 + A_s f_s) n}{0.06 f_s} = \frac{(80.4 + 6686 \times 0.230) 36}{0.06 \times 230 \times 1000}$$

$$= 0.628 \text{ m}$$

$$a = \frac{\sqrt{(\zeta^2 + 2 \times d_{eff} \times \zeta)} - \zeta}{3} = 0.75 \text{ m}$$

(against 1.9 m)

Repeating the iteration with the calculated value of 'a' the value of 'a' converges as:

$$a = 0.682 \text{ m}, A_s = 0.000460 \text{ m}^2$$

Using 2-20 mm HYSD bars, area of steel provided = 628 mm^2

Second Case:

$$\frac{M_2}{P_2 d_{eff}} = 0.797$$

Distance of axial load from edge of wall

$$\alpha = \frac{\frac{l_w}{2} - d_e}{d_{eff}} = 0.484$$

$$\frac{2}{3} - \alpha = 0.183 < \frac{M_1}{P_1 d_{eff}} \quad (\text{Region 3 applies})$$

Assumed distance of compression centroid from edge,

$$a = 0.70 \text{ m (Assumed)}$$

First Iteration

$$M_p = P_2 \left(\frac{l_w}{2} - a \right) = 160 \left(\frac{6.5}{2} - 0.7 \right)$$

$$= 408.0 \text{ kNm}$$

$$A_s = \frac{M_1 - M_p}{f_s (d_{eff} - a)}$$

$$= \frac{803 - 408}{230(6.3 - 0.7)} = 0.00031 \text{ m}^2$$

$$\zeta = \frac{(P_2 + A_s f_s) n}{0.06 f_s}$$

$$= \frac{(160 + 310 \times 0.230) 36}{0.06 \times 230 \times 1000} = 0.668 \text{ m}$$

$$a = \frac{\sqrt{(\zeta^2 + 2 \times d_{ef} \times \zeta)} - \zeta}{3}$$

$$a = 0.77 \text{ m}$$

Iterating the above equations, value of 'a' converges as:

$$a = 0.77 \text{ m}$$

Using 2-16mm HYSD bars, area of steel provided,

$$A_s = 402 \text{ mm}^2$$

Shear Design Considerations

Actual shear stress is given by

$$f_v = \frac{V_{cr}}{d_{eff} \times 0.06} = 0.139 \text{ MPa}$$

Providing web shear reinforcement

$$\frac{M_{cr}}{V_{cr} d_{eff}} = 0.873 \quad (< 1.0)$$

As per Section 6.1.4, allowable shear stress is given by

$$F_v = \frac{1}{24} \left[4 - \frac{M_{cr}}{V_{cr} d_{eff}} \right] \sqrt{f_m}$$

$$= \frac{1}{24} \left[4 - \frac{289.55}{52.64 \times 6.3} \right] \sqrt{10}$$

$$= 0.412 \text{ MPa}$$

Maximum allowable shear stress

$$f_{vm} = \left(0.6 - 0.2 \frac{M_{cr}}{V_{cr} d_{eff}} \right)$$

$$= \left(0.6 - 0.2 \frac{289.55}{52.64 \times 6.3} \right)$$

$$= 0.425 \text{ MPa}$$

(As per Section 6.1.4)

Since, calculated shear stress is less than the allowable shear stress, the shear design is OK.

Providing horizontal web reinforcement of 8mm dia, area of steel provided 50 mm²

Maximum spacing of bars in both horizontal and vertical direction shall be lesser of following:

- 1) one-third of length of shear wall = 2.2 m
- 2) one-third of height of shear wall = 2.0 m
- 3) 1.2 m

(As per Section 7.2.3.1)

Provide 8 mm dia. bars @500 mm c/c.

Percentage of shear reinforcement provided

$$= \frac{50}{500 \times 200} \times 100 = 0.05 \%$$

As per Sec 7.2.3.1, the minimum shear reinforcement area in each direction shall not be less than 0.07% of gross cross-sectional area and sum of reinforcement in both direction should not be less than 0.2% .

Provide 10 mm dia. bars @500 mm c/c.

Percentage of reinforcement in horizontal direction is 0.078% and percentage of steel in vertical direction is 0.018%. Since, the sum of percentage of steel in both direction is more than 0.2%, the design is OK.

Example 3 – ANALYSIS OF FLEXIBLE DIAPHRAGM OF ONE-STOREY REINFORCED CONCRETE MASONRY BUILDING

Problem Statement:

A simple four walled 15 m by 12 m building is a part of a warehouse facility located in seismic zone IV. It is constructed with a flexible timber roof on 3 m high reinforced concrete masonry unit (CMU) bearing walls in 200 mm thick. All walls are reinforced vertically and horizontally. The average weight of the roof diaphragm and mounted equipment is 900 N/m^2 . All connections between walls, roof and foundation are pinned. The building performance is being analyzed for an earthquake acting parallel to the short dimension. Disregard all openings in the walls.

- Find diaphragm-to-wall shear on line A-C for design of diaphragm-to-wall connection.
- Find the maximum chord force on the line C-D and design reinforcement for the chord (bond beam).
- Find the horizontal shear stress in wall A-C at a point 1.5 m above the foundation.
- Determine whether the wall thickness is adequate

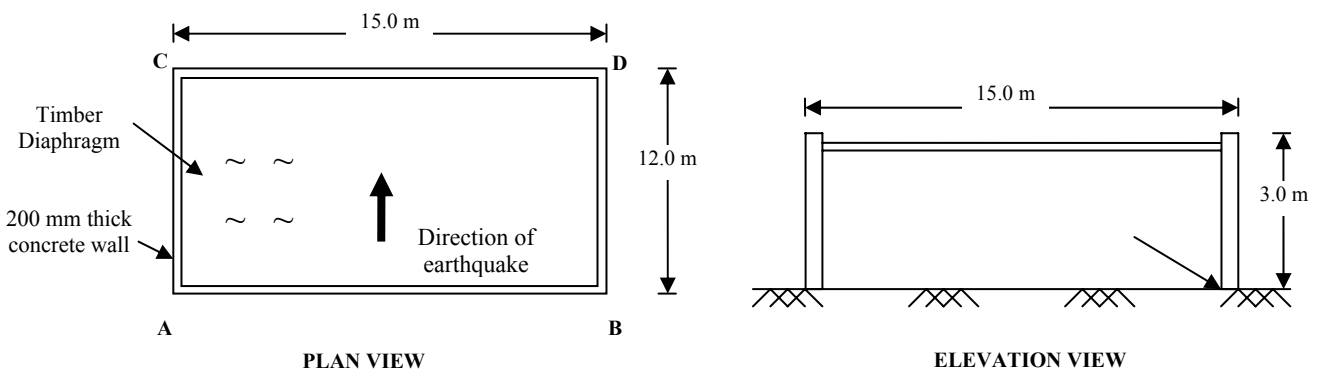


Figure 3.1: Plan and Elevation View of Building

Solution:

Length of, building $L = 15 \text{ m}$

Width of building, $B = 12 \text{ m}$

Prism strength of masonry, $f_m = 10 \text{ MPa}$

Unit weight of concrete = 25 kN/m^3

Calculation of Base Shear Coefficient:

Zone factor, $Z = 0.24$ (Seismic Zone IV)

Importance Factor, $I = 1.0$

Response reduction factor, $R = 3.0$

Design spectral acceleration, $S_a = 2.5g$

(Assuming it to be a short period structure)

Base shear coefficient,

$$A_h = \frac{ZIS_a}{2Rg} = \frac{0.24 \times 1.0 \times 2.5}{2 \times 3.0} = 0.104$$

Calculation of weight of diaphragm:

The weight being accelerated by the earthquake consists of the diaphragm weight and a portion of the wall weight. The weight of the diaphragm is

$$W_d = 900 \times 15 \times 12 \text{ N} = 162 \text{ kN}$$

Since the concrete has a density of 25 kN/m^3 , the weight of 1 m^2 of a 200 mm thick wall is

$$w_c = 0.2 \times 25 \text{ kN/m}^2 = 5.0 \text{ kN/m}^2$$

For purpose of determining the diaphragm force, the upper half of the perpendicular walls is used to calculate the wall weight. The remaining seismic force passes directly into the foundation without being carried by the wall-diaphragm connection. The weight of half perpendicular walls is

$$W_{pw} = 2 \times 1/2 \times 5 \times 15 \times 3 = 225 \text{ kN}$$

a) Diaphragm-to-wall Shear:

The roof diaphragm is to be designed to resist a portion of floor forces above it, as weighted by the floor weights. The diaphragm force must be within the values $0.35ZIW_p$ and $0.75ZIW_p$. With the given values of Z and I , the limits of diaphragm force are $0.105W_p$ and $0.225W_p$.

This is a simple one-story building with only one diaphragm, all the inertial load from the accelerating wall and roof masses must be carried by the wall-roof connection.

Checking, $0.105 < 0.104 < 0.225$

$$F_d = A_h \times W_p = 0.104 \times (162 + 225) = 40.25 \text{ kN}$$

Shear per meter length

$$V = \frac{40.25 \times 1000}{2 \times 12} = 1.68 \text{ kN/m}$$

$$C_c = 1.68 \times 12 \text{ kN} = 20.2 \text{ kN}$$

b) Design of Chord (bond beam) :

Loads on diaphragm and resulting forces in chord (bond beam) at roof level are shown in Figure 3.2. The distributed shear force across the face of diaphragm is

$$w_s = \frac{F_d}{L} = \frac{40.25 \times 10^3}{15} = 2.68 \text{ kN/m}$$

Chord force, C is given by

$$C = \frac{w_s \times L^2}{8 \times B} = \frac{2.68 \times 15^2}{8 \times 12} = 3.28 \text{ kN}$$

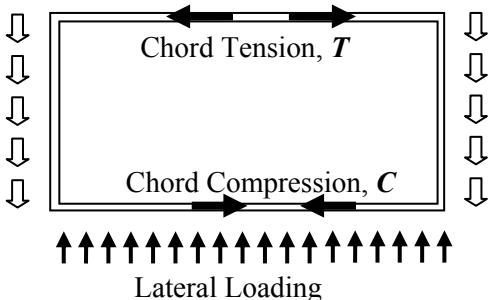


Figure 3.2: Diaphragm loading and chord forces

Using HYSD bars,
 Allowable tensile stress of bars = 230 MPa
 Area of reinforcement required

$$A_{st} = \frac{3.28 \times 10^3}{230} = 14 \text{ mm}^2$$

Provide 2-10 mm dia. bars in the bond beam.

c) Horizontal Shear Stress in wall A-C :

Half the weight of the parallel walls is

$$W_{sw} = 5000 \times 2 \times 12 \times 3/2 = 180 \text{ kN}$$

Total weight is sum of weight of parallel walls, half weight of perpendicular wall and weight of diaphragm.

$$\begin{aligned} \text{Total weight is } W &= W_d + W_{pw} + W_{sw} \\ &= 162 + 225 + 180 \text{ kN} \\ &= 567 \text{ kN} \end{aligned}$$

The seismic force is

$$V = 0.104 \times 567 \text{ kN} = 58.97 \text{ kN}$$

Since the perpendicular walls have no rigidity, all the seismic force is resisted by the two parallel walls. The shear stress is

$$f_v = \frac{58.97 \times 1000}{2 \times 0.2 \times 12} = 0.012 \text{ MPa}$$

d) Determination of Wall Thickness :

Maximum shear stress is 1.5 times average shear stress.

Maximum shear stress is

$$f_{vm} = 1.5 \times f_v = 1.5 \times 0.012 \text{ MPa} = 0.02 \text{ MPa}$$

As per Sec 6.1.4, allowable shear stress (F_v) is given by the least of the following:

- (i) $1.33 \times 0.5 \text{ MPa} = 0.665 \text{ MPa}$
- (ii) $1.33(0.1 + 0.2 \times f_d) \text{ MPa}$
 $= 1.33(0.1 + 0.2 \times \frac{567 \times 1000}{2 \times 200 \times 12000})$
 $= 0.165 \text{ MPa} \quad (\text{Governs})$
- (iii)
 $0.133 \times 0.125 \times \sqrt{f_m} \text{ MPa}$
 $= 0.133 \times 0.125 \times \sqrt{10}$
 $= 0.526 \text{ MPa}$

Allowable shear stress is 0.165 MPa. Since calculated shear stress is less than allowable shear stress, the section is safe in shear.

Example 4 – LATERAL LOAD ANALYSIS OF THREE-STOREY CMU MASONRY WALL BUILDING

Problem Statement:

The elevation of a three-storey perforated shear wall is as shown in Figure 4.1. It is assumed to be a masonry structure which is fully grouted and un-reinforced. The grade of masonry is given as $f_m = 8$ MPa, wall thickness is 200 mm and steel is used is HYSD bars having allowable tensile stress of 230 MPa. For this building determine the following.

1. Determine and check vertical compressive stresses, tensile stresses and shear stresses in the masonry as per IS:1905.
2. Design reinforcement for piers that may require it.

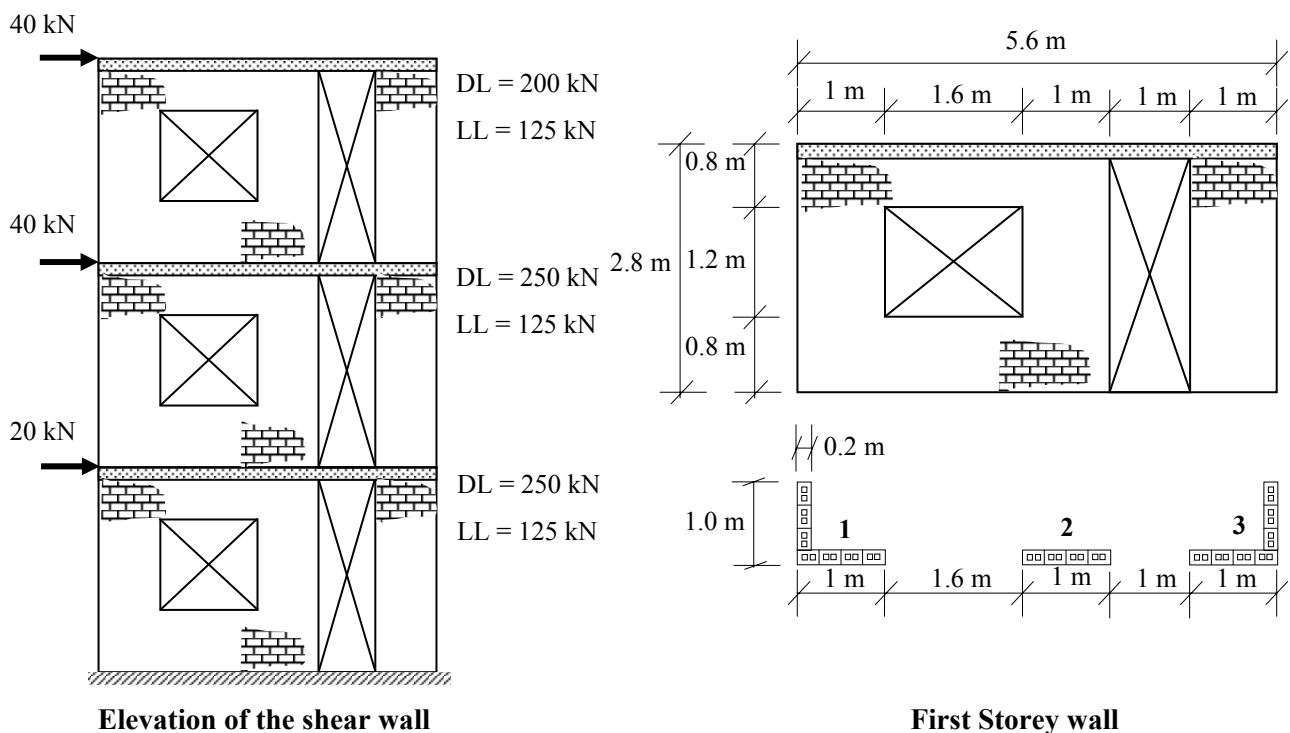


Figure 4.1: Details of the Shear Wall

Solution:

Calculation of Geometric Properties:

Pier Areas (cross sectional)

$$\begin{aligned} \text{Area of Pier 1} &= (800 \times 190 + 1000 \times 190) \text{ mm}^2 \\ &= 3.42 \times 10^5 \text{ mm}^2 \end{aligned}$$

$$\begin{aligned} \text{Area of Pier 2} &= (1000 \times 190) \text{ mm}^2 \\ &= 1.9 \times 10^5 \text{ mm}^2 \end{aligned}$$

$$\begin{aligned} \text{Area of Pier 3} &= (800 \times 190 + 1000 \times 190) \text{ mm}^2 \\ &= 3.42 \times 10^5 \text{ mm}^2 \end{aligned}$$

$$\text{Height of pier 1} = 1.2 \text{ m}$$

$$\text{Height of pier 2} = 1.2 \text{ m}$$

$$\text{Height of pier 3} = 2.8 \text{ m}$$

$$E_m = 550 \times 8 = 4400 \text{ MPa}$$

Moment of Inertia and Stiffness:

Pier 1:

Centroidal distance from left face of wall

$$y_1 = \frac{800 \times \frac{190^2}{2} + 190 \times \frac{800^2}{2}}{800 \times 190 + 1000 \times 190} \times 10^{-3} = 0.32 \text{ m}$$

Moment of inertia about centroidal axis

$$(I_g)_1 = \left[190^3 \times \frac{800}{12} + 190 \times 800 \times \left(320 - \frac{190}{2} \right)^2 + 190 \times 1000 \times (500 - 320)^2 \right] + \frac{190}{12} \times 1000^3$$

$$= 3.01 \times 10^{10} \text{ mm}^4$$

Stiffness = k_1

$$= \frac{1}{\left(\frac{1200^3}{12 \times E_m \times I_{g1}} \right) + \frac{5}{6} \times 190 \times 1000 \times 0.4 \times E_m}$$

$$= 1.855 \times 10^5 \text{ N/mm}$$

$$\frac{k_1}{E_m} = 42.15 \text{ mm}$$

Pier 2:

Centroidal distance from left face of wall

$$y_1 = (1000 + 1600 + 500) \times 10^{-3} = 3.1 \text{ m}$$

Moment of inertia about centroidal axis

$$(I_g)_2 = \frac{190 \times 1000^3}{12} = 1.583 \times 10^{10} \text{ mm}^4$$

Stiffness = k_2

$$= \frac{1}{\left(\frac{1200^3}{12 \times E_m \times I_{g2}} \right) + \frac{5}{6} \times 190 \times 1000 \times 0.4 \times E_m}$$

$$= 1.569 \times 10^5 \text{ N/mm}$$

$$\frac{k_2}{E_m} = 35.661 \text{ mm}$$

Pier 3:

Centroidal distance from left face of wall

$$y_3 = (5600 - 320) \times 10^{-3} = 5.28 \text{ m}$$

Moment of inertia about centroidal axis

$$(I_g)_1 = \left[190^3 \times \frac{800}{12} + 190 \times 800 \times \left(320 - \frac{190}{2} \right)^2 + 190 \times 1000 \times (500 - 320)^2 \right] + \frac{190}{12} \times 1000^3$$

$$= 3.01 \times 10^{10} \text{ mm}^4$$

Stiffness = k_3

$$= \frac{1}{\left(\frac{2800^3}{12 \times E_m \times I_{g3}} \right) + \frac{5}{6} \times 190 \times 1000 \times 0.4 \times E_m}$$

$$= 4.194 \times 10^4 \text{ N/mm}$$

$$\frac{k_1}{E_m} = 9.53 \text{ mm}$$

$$\sum \frac{k_i}{E_m} = 87.343 \text{ mm}$$

Distribution of Lateral Shear among Piers:

As the floor is assumed to be rigid the lateral forces will be distributed in proportion to the pier stiffness. $V_{ei} = V_b \frac{k_i}{\sum k_i}$ The distribution of shear

force among piers is shown in Table 4-1

Total base Shear = (40 + 40 + 20) = 100 kN

Table 4-1: Distribution of Lateral Shear among Piers

Pier No.	k_i (kN/m)	$\frac{k_i}{\sum k_i}$	$V_b \frac{k_i}{\sum k_i}$ (kN)
1	1.855×10^5	0.483	48.26
2	1.569×10^5	0.408	40.83
3	4.194×10^4	0.109	10.91

Distribution of Overturning Moment to Piers as Axial Forces

Location of Neutral axis

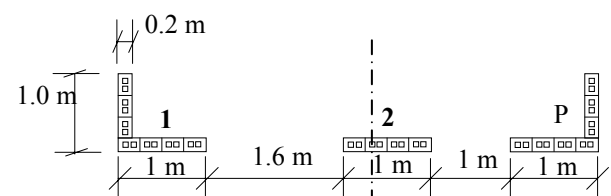


Figure 4.2: Piers in the wall

Table 4-2: Location of Neutral axis

Pier No.	y_i (m)	A_i (m ²)	$A_i \times y_i$ (m ³)
1	0.32	0.342	0.109
2	3.1	0.19	0.589
3	5.28	0.342	1.806
Σ		0.874	2.504

$$y_{NA} = \frac{\sum A_i \times y_i}{\sum A_i} = \frac{2.504}{0.874} = 2.865 \text{ m}$$

Distance of the central line of the piers from N.A.

Pier 1:

$$(y_b)_1 = y_{NA} - y_1 = 2.865 - 0.32 = 2.545 \text{ m}$$

Pier 2:

$$(y_b)_2 = y_{NA} - y_2 = 2.865 - 3.1 = -0.235 \text{ m}$$

Pier 3:

$$(y_b)_3 = y_{NA} - y_3 = 2.865 - 5.28 = -2.415 \text{ m}$$

Total Overturning Moment

$$= (40 \times 7 + 40 \times 4 + 20 \times 1) = 460 \text{ kNm}$$

Moment of inertia of the whole wall section

$$= I_{NA} = \sum (I_{NA})_i$$

where, $(I_{NA})_i = A_i (y_b)_i^2 + (I_g)_i$

Axial load due to overturning moment M

$$= (P_e)_i = (DF_m)_i \times M$$

where, $(DF_m)_i = \frac{A_i (y_b)_i}{\sum I_{NA}}$. The calculation is

shown in tabular form in Table 4-3 and Table 4-4

Table 4-3: Geometric properties of the piers in north wall

Pier No.	A_i (m ²)	$(y_b)_i$ (m)	$A_i (y_b)_i^2$ (m ⁴)	$(I_g)_i$ (m ⁴)	$(I_{NA})_i$ (m ⁴)
1	0.342	2.545	2.216	0.03	2.246
2	0.19	-0.235	0.01	0.016	0.026
3	0.342	-2.415	1.994	0.03	2.024
Σ	0.874				4.296

Table 4-4: Distribution of Overturning Moment to Piers as Axial Forces

Pier No.	$(I_{NA})_i$ (m ⁴)	$A_i \times (y_b)_i$ (m ³)	$(DF_m)_i$ (m ⁻¹)	$(P_e)_i$ (kN)
1	2.246	0.87	0.203	93.198
2	0.026	-0.045	-0.01	-4.776
3	2.024	0.826	-0.192	-88.422
Σ	4.296		0.00	0.00

Distribution of Direct Axial Compression:

Axial load due to gravity loads are distributed in proportion to the tributary wall length.

Total dead load =

$$P_d = (200 + 250 + 250) = 700 \text{ kN}$$

$$(P_d)_1 = 700 \times \frac{(0.8 + 1 + 0.8)}{7.2} = 252.78 \text{ kN}$$

$$(P_d)_2 = 700 \times \frac{(0.8 + 1 + 0.5)}{7.2} = 223.61 \text{ kN}$$

$$(P_d)_3 = 700 \times \frac{(0.5 + 1 + 0.8)}{7.2} = 223.61 \text{ kN}$$

Total live load =

$$P_l = (3 \times 125) = 375 \text{ kN}$$

$$(P_l)_1 = 375 \times \frac{(0.8 + 1 + 0.8)}{7.2} = 135.42 \text{ kN}$$

$$(P_l)_2 = 375 \times \frac{(0.8 + 1 + 0.5)}{7.2} = 119.79 \text{ kN}$$

$$(P_l)_3 = 375 \times \frac{(0.5 + 1 + 0.8)}{7.2} = 119.79 \text{ kN}$$

The loads transferred to individual piers are shown in tabular form in Table 4-5.

Table 4-5: Forces in Different piers due to different Loads

Pier No.	$(P_d)_i$ (kN)	$(P_l)_i$ (kN)	$(P_e)_i$ (kN)	$(V_e)_i$ (kN)	$(M_e)_i$ (kNm)
1	252.78	135.42	93.20	48.26	28.96
2	223.61	119.79	-4.78	40.83	24.50
3	223.61	119.79	-88.42	10.91	15.28
Σ	700.00	375.00	0.00	100.00	

Note : As the piers are assumed to be fixed at both

$$\text{ends } M_{ei} = V_{ei} \times \frac{h_i}{2}.$$

Load Combinations:

Three probable load cases have been considered to cause failure and each has some potential failure mode which will govern the failure in that particular load combination.

Table 4-6: Load Combinations

Load Case	Axial Comp. Force	Moment	Shear
1	$P1_i = (P_d)_i + (P_l)_i$		
	Results in large axial forces.		
2	$P2_i = 0.75 \times ((P_d)_i + (P_l)_i + (P_e)_i)$	$M2_i = 0.75(M_e)_i$	$V2_i = 0.75(V_e)_i$
	Results in large axial and bending force for lateral force.		
3	$P3_i = 0.75 \times (0.9 \times (P_d)_i - (P_e)_i)$	$M3_i = 0.75(M_e)_i$	$V3_i = 0.75(V_e)_i$
	Result in low axial load which causes low moment capacity.		

Considering the above combinations we are finding out the final loads on different piers and that is shown in Table 4-7.

Table 4-7: Loads on piers under different Load combinations

Pier No.	$P1_i$ (kN)	$P2_i$ (kN)	$P3_i$ (kN)	$M2_i$ (kNm)	$V3_i$ (kN)
1	388.19	361.04	99.46	21.72	36.19
2	343.40	261.13	146.24	18.37	30.62
3	343.40	323.87	83.50	11.46	8.19

Check for Pier Axial and Flexural Stresses:

Axial and flexural stresses in the piers are checked assuming the piers are un-reinforced and un-cracked.

Load case 1: (D+L)

Given, $f_m = 8$ MPa

Allowable basic compressive stress

$$f_{bc} = 0.25 f_m = 0.25 \times 8 = 2 \text{ MPa}$$

As per clause 4.6.1 of IS: 1905

$$\text{Slenderness Ratio} = SR_i = \frac{h_i}{\text{thickness}} = \frac{h_i}{0.19}$$

Stress reduction factor $((k_s)_i)$ will be corresponding to the corresponding slenderness ratio as per clause 5.4.1.1 and Table 9 of IS: 1905-1987..

Area reduction factor (k_a) will be taken as per clause 5.4.1.2 of IS: 1905-1987.

Shape modification factor (k_p) is taken as 1.

Allowable compressive stress

$$F_{ai} = (k_s)_i \times k_a \times k_p \times f_{bc} \text{ (clause 5.4.1 of IS: 1905-1987) as shown in Table 4-8.}$$

Table 4-8: Calculation of allowable compressive stress in different piers in direct compression

Pier No.	SR_i	$(k_s)_i$	k_a	k_p	$(F_a)_i$ (MPa)
1	6.316	0.992	1.00	1.0	1.984
2	6.316	0.992	0.985	1.0	1.954
3	14.737	0.762	1.00	1.0	1.524

The compressive stress developed in the piers and corresponding demand capacity ratio is checked in Table 4-9.

Table 4-9: Check of Piers in compression under load case 1

Pier No.	$P1_i$ (kN)	$f_a 1_i$ (MPa)	$(F_a)_i$ (MPa)	$\frac{f_a 1_i}{(F_a)_i}$
1	388.19	1.135	1.984	0.572
2	343.40	1.807	1.954	0.925
3	343.40	1.004	1.524	0.659

In each of the pier the demand capacity ratio is less than 1, hence piers are safe in axial compression under load case 1.

Load case 2: 0.75(D+L+E)

In this case compressive stress due to both direct compression and bending and their combined action is checked as shown in Table 4-10, 4-11, 4-12.

$$\text{Axial component} = f_a 2_i = \frac{P1_i}{A_i}$$

$$\text{Bending component} = f_a 2_i = \frac{M2_i}{(S_g)_i}$$

Where, $(S_g)_i = \frac{(I_g)_i}{(y_i)_{\max}}$ = section modulus of piers.

Table 4-10: Check of Piers in direct compression under load case 2

Pier No.	$P2_i$ (kN)	$f_a 2_i$ (MPa)	$(F_a)_i$ (MPa)	$\frac{f_a 2_i}{(F_a)_i}$
1	361.04	1.056	1.984	0.532
2	261.13	1.374	1.954	0.703
3	323.87	0.947	1.524	0.621

Allowable compressive stress in bending
 $= (F_b)_i = 1.25 \times (F_a)_i$

Table 4-11: Check of Piers in bending compression under load case 2

Pier No.	$(S_g)_i$ (m ³)	$M2_i$ (kNm)	$f_b 2_i$ (MPa)	$(F_b)_i$ (MPa)	$\frac{f_b 2_i}{(F_b)_i}$
1	0.044	21.72	0.49	2.48	0.20
2	0.032	18.37	0.57	2.44	0.23
3	0.044	11.46	0.26	1.91	0.14

Table 4-12: Check in combined action

Pier No.	$\frac{f_a 2_i}{(F_a)_i} + \frac{f_b 2_i}{(F_b)_i}$
1	0.732
2	0.933
3	0.761

In each of the pier the demand capacity ratio is less than 1, hence piers are safe in axial compression, bending compression and their combined action under load case 2.

Load case 3: 0.75(0.9D -E)

In this case minimum axial stress is checked against allowable tensile stress as shown in Table 4-13 and 4-14.

Allowable compressive stress in bending

$$= (F_t)_i = 0.07 \text{ MPa}$$

Table 4-13: Check of Piers in direct axial force under load case 3

Pier No.	$P3_i$ (kN)	$f_a 3_i$ (MPa)	$(F_a)_i$ (Mpa)	$\frac{f_a 3_i}{(F_a)_i}$
1	99.46	0.29	1.984	0.15
2	146.24	0.77	1.954	0.39
3	83.50	0.24	1.524	0.16

Considering that moment will cause axial tension and tensile stress of $f_b 3_i$ and axial force due to dead load will cause axial compression and their resultant will act upon the pier given in the table below. (+ sign denotes tensile stress).

$$f3_i = f_b 3_i - f_a 3_i$$

Table 4-14: Check of Piers in axial tension under load case 3

Pier No.	$(S_g)_i$ (m ³)	$M3_i$ (kNm)	$f_b 3_i$ (MPa)	$f3_i$ (MPa)	$\frac{f3_i}{(F_t)_i}$
1	0.044	21.72	0.49	0.2	2.86
2	0.032	18.37	0.57	-0.2	-2.86
3	0.044	11.46	0.26	0.02	0.29

In each of the pier except the pier 1 the demand capacity ratio is less than 1, hence those piers are safe in axial compression and combined action of axial compression and tension due to bending under load case 3.

So, reinforcement is to be provided in pier 1.

Check for Pier Shear Stresses:

The pier shear stresses will be critical under load case 3 when axial loads are minimum. The shear stress developed and allowable values for each pier are shown in Table 4-15.

Assuming piers are un-cracked and un-reinforced.

Web area of each pier

$$= A_{web} = 0.19 \times 1.0 = 0.19 \text{ m}^2$$

Shear stress developed in piers

$$= (f_v)_i = 1.5 \times \frac{V3_i}{(A_{web})_i}$$

$$\text{Stress due to dead load in piers} = (f_d)_i = \frac{P3_i}{A_i}$$

Allowable Shear stress $= (F_v)_i$ is minimum of

- 0.5 MPa
- $0.1 + 0.2(f_d)_i$
- $0.125\sqrt{f_m} = 0.353 \text{ MPa}$

Table 4-15: Check of Piers in shear under load case 3

Pier No.	$V3_i$ (kN)	$(f_v)_i$ (MPa)	$(f_d)_i$ (MPa)	$(F_v)_i$ (MPa)	$\frac{(f_v)_i}{(F_v)_i}$
1	36.19	0.29	0.29	0.16	1.81
2	30.62	0.24	0.77	0.25	0.96
3	8.19	0.07	0.24	0.15	0.47

In each of the pier except the pier 1 the demand capacity ratio is less than 1, hence those piers are safe in shear under load case 3.

So, shear reinforcement is to be provided in pier 1.

Design of vertical reinforcement for Pier 1:

Design forces:

$$V = V3_1 = 36.19 \text{ kN}$$

$$M = \frac{V \times h_1}{2} = 21.72 \text{ kNm}$$

$$P = P3_1 = 99.46 \text{ kN}$$

Assuming that the neutral axis is within the flange ($kd < 0.19\text{m}$) and tension controls the design.

$$b = 1000 \text{ mm}$$

$$d = (1 - 0.115) = 0.885 \text{ m}$$

$$F_s = 230 \text{ Mpa}$$

$$a = \frac{P}{b \times d \times F_s} = \frac{99.46 \times 10^3}{1000 \times 885 \times 230} = 4.886 \times 10^{-4}$$

$$1 \text{ no. } 12 \text{ mm } \phi \text{ bar is used, } A_s = 113 \text{ mm}^2$$

$$p = \frac{A_s}{b \times d} = \frac{113}{1000 \times 885} = 1.28 \times 10^{-4}$$

$$E_m = 550 \times 8 = 4400 \text{ MPa}$$

$$E_s = 2 \times 10^5 \text{ MPa}$$

$$n = \frac{E_s}{E_m} = \frac{2 \times 10^5}{4400} = 45.5$$

$$k^2 + 2 \times n \times (p + a) \times k - 2 \times n \times (p + a) = 0$$

$$\Rightarrow k^2 + 0.056 \times k - 0.056 = 0$$

$$\Rightarrow k = 0.21$$

$$\Rightarrow k = -0.27$$

$$\text{for } k = 0.21 \quad kd = 186 \text{ mm} < 190 \text{ mm}$$

Hence N.A is within flange.

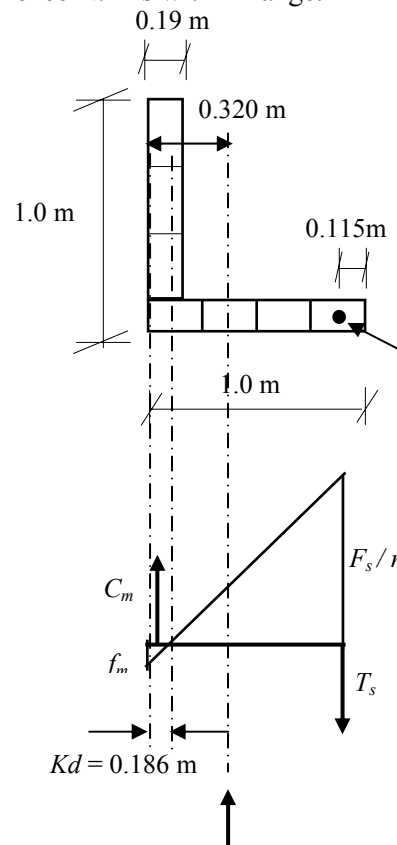


Figure 4.3

By similar triangle,

$$f_m' = \frac{kd}{d - kd} \times \frac{F_s}{n} = \frac{186 \times 230}{(885 - 186) \times 45.5}$$

$$= 1.345 \text{ Mpa}$$

$$\frac{f_m'}{f_{bc}} = \frac{1.345}{2.0} = 0.673 < 1$$

Hence tension controls.

$$C_m = 0.5 \times f_m' \times k \times d \times b = 125 \text{ kN}$$

$$T_s = A_s \times F_s = 26 \text{ kN}$$

$$C_m - T_s = 99 \text{ kN}$$

$$P = 99.46 \text{ KN}$$

Hence O.K.

$$M_{CG} = T_s \times (d - y_1) + C_m \times \left(y_1 - \frac{kd}{3}\right)$$

$$= 26 \times (1 - 0.32) + 125 \times \left(0.32 - \frac{0.186}{3}\right)$$

$$= 49.93 \text{ kNm}$$

$$\frac{M}{M_{CG}} = \frac{21.72}{49.93} = 0.435$$

Hence O.K.

Design of horizontal reinforcement for Pier 1:

Let, 10 mm ϕ bar is used, $A_v = 113 \text{ mm}^2$

$$s_v = \frac{A_v \times F_s \times d}{V} = 438 \text{ mm}$$

Providing 10 mm ϕ bar @ 200 mm c/c

$$V_s = \frac{A_v \times F_s \times d}{s_v} = \frac{78 \times 230 \times 885}{200} = 79.4 \text{ kN}$$

which is more than the applied shear.

Hence, O.K.

$$f_v = \frac{V}{t \times d} = \frac{36.19 \times 10^3}{190 \times 885} = 0.215 \text{ MPa}$$

$$\frac{M}{Vd} = \frac{21.72}{36.19 \times 0.885} = 0.678 < 1.0$$

Hence, allowable shear stress will be minimum of

$$a) \frac{1}{24} \left(4 - \frac{M}{Vd}\right) \sqrt{f_m'} = 0.392 \text{ MPa}$$

$$b) \left(0.6 - 0.2 \frac{M}{Vd}\right) = 0.46 \text{ MPa}$$

So, $F_v = 0.392 \text{ MPa} > f_v$

Hence O.K.

Check for provision of minimum reinforcement (as per section 7.2.3.1):

$$p_h = \frac{78}{190 \times 200} = 2.05 \times 10^{-3} > 0.0007$$

Hence O.K.

Area of vertical reinforcement required

$$A_{vert} = 0.0007 \times 342 \times 10^3 = 239.4 \text{ mm}^2$$

$$\text{No. of 12 mm } \phi \text{ bar required} = \frac{239.4}{113} = 2.11$$

So, provide 3 nos. 12 mm ϕ bar one at each end at corner.

$$p_v = \frac{3 \times 113}{342 \times 10^3} = 0.99 \times 10^{-3} > 0.0007$$

$$p_h + p_v = 3.04 \times 10^{-3} > 0.002$$

Hence, O.K.

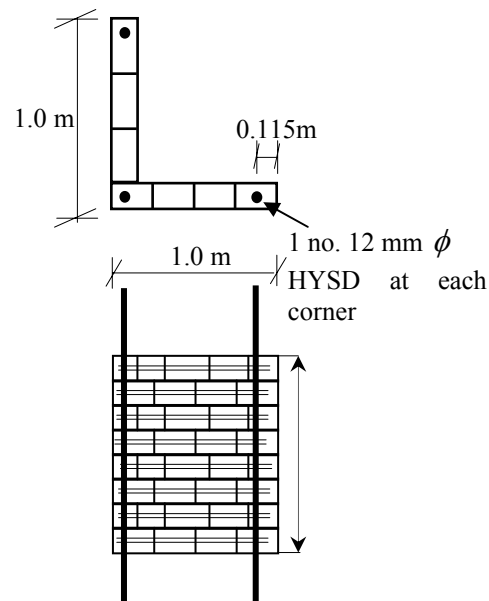


Figure 4.4: Details of reinforcement in Pier 1

Example 5 – SEISMIC ANALYSIS OF TWO-STOREY REINFORCED MASONRY BUILDING

Problem Statement:

The elevations of a two-story warehouse building located in seismic zone V are shown in Figure 5.1. It is assumed that the second-story walls and walls in the east-west direction have no openings. The wall thickness is 230 mm. The reinforced brick masonry building is to be designed to resist the lateral loads designated by the IS Codes. The grade of masonry is given as $f_m = 10$ MPa and steel is used is HYSD bars having allowable tensile stress of 230 MPa. In this example, earthquake loads in east-west direction is considered. This example illustrates the calculations pertaining to the determination of the following:

1. The total lateral force acting at roof and second floor levels.
2. The distribution of the diaphragm shears to the parallel shear walls.
3. The internal distribution to each pier within a wall of the total force brought to that shear wall.
4. The most severe axial load combination in conjunction with the in-plane bending moment and shear which each pier must be designed to resist.

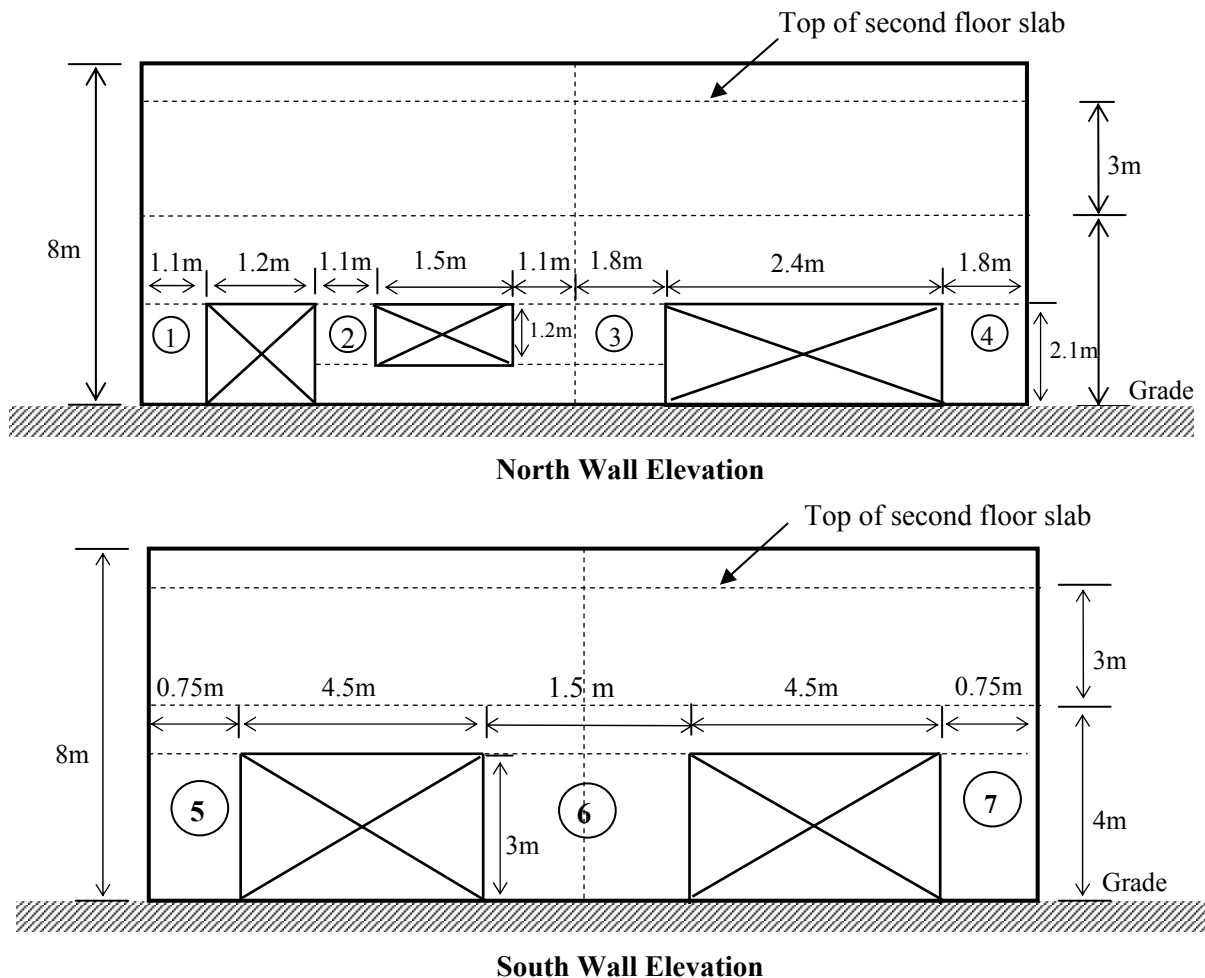


Figure 5.1: Two-Story Warehouse Building Elevation.

Solution:

Given data:

Length of building (N-S) $L = 18$ m

Breadth of building (E-W) $D = 12$ m

Thickness of the walls $t = 0.23$ m

Loading data:

Roof

Slab Dead Load $DL_2 = 5.0$ kPa

Slab Live Load $LL_2 = 1.5$ kPa

Floor

Slab Dead Load $DL_1 = 5.0$ kPa

Storage live Load $LL_1 = 5.0$ kPa

Wall Weights

Unit Weight of Masonry = 20 kN/m³

So, weight of wall per unit area

$$= 0.23 \times 1 \times 1 \times 20 = 4.6 \text{ kN/m}^2$$

Calculation of Loads:

Calculation of total loads

$$\begin{aligned} \text{Roof Dead load } DL'' &= DL_2 \times L \times D \\ &= 1080 \text{ kN} \end{aligned}$$

$$\begin{aligned} \text{Roof Live Load } RL_2 &= LL_2 \times L \times D \\ &= 324 \text{ kN} \end{aligned}$$

Wall Load (Upper Storey)

$$\begin{aligned} &= (1 + 3) \times (12 \times 2 + (18 - 2 \times 0.23) \times 2) \times 4.6 \\ &= 1087.1 \text{ kN} \end{aligned}$$

First Floor level loads

$$DL' = DL_1 \times L \times D = 1080 \text{ kN}$$

$$LL = LL_1 \times L \times D = 2160 \text{ kN}$$

Calculation of Seismic Weight

Due to Wall Load (Upper Storey)

$$\begin{aligned} WL_2 &= (1 + 1.5) \times (12 \times 2 + (18 - 2 \times 0.23) \times 2) \times 4.6 \\ &= 679.4 \text{ kN} \end{aligned}$$

Due to Wall Load (Lower Storey)

$$\begin{aligned} WL_1 &= (1.5 + 2) \times [(12 \times 2 + (18 - 2 \times 0.23) \times 2) \\ &\quad - 0.1 \times (1.2 + 1.5 + 2.4) - 1 \times (2 \times 4.5)] \times 4.6 \end{aligned}$$

$$= 907.4 \text{ kN}$$

Seismic weight on roof

$$= DL'' + WL_2 = 1080 + 679.4 = 1759.4 \text{ kN}$$

As per clause 7.3.1 of IS 1893: 2002, imposed load to be considered in seismic weight,

$$50 \% \text{ of imposed load} = 2.5 \text{ kPa}$$

Seismic weight on First Floor

$$\begin{aligned} &= DL' + 0.5 \times LL + WL_1 = 1080 + 0.5 \times 1080 + 907.4 \\ &= 2527.4 \text{ kN} \end{aligned}$$

Total seismic weight

$$W' = 1759.4 + 2527.4 = 4286.8 \text{ kN}$$

Calculation of Base Shear

$$Z = 0.36 \quad \text{(Seismic zone V)}$$

$$I = 1 \quad \text{(Occupancy importance factor)}$$

$$R = 3 \quad \text{(Response reduction factor)}$$

$$H_n = 7 \text{ m} \quad \text{(Height of a building)}$$

(Refer to the Figure 5.1)

The average response acceleration coefficient,

$\left(\frac{S_a}{g}\right) = 2.5$, assuming the building period to lie in the short period range of the design spectrum.

$$V_B = \frac{Z \times I}{2 \times R} \times \frac{S_a}{g} \times W' = 643 \text{ kN}$$

(IS: 1893 (Part 1):2002 Clause 6.4.2)

Design Lateral Forces at Each Floor Level

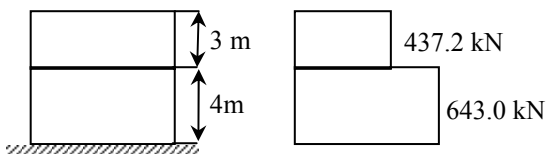
The distribution of shear force in the vertical direction is made as per provision of IS: 1893 and corresponding calculations are shown in the Table 5-1

Total height from Ground floor level to roof: $L_2 = 7$ m

Total height from Ground floor level to first floor $L_1 = 4$ m

Table 5-1: Lateral Force Distribution (clause 7.7.1 of IS 1893: 2002)

Story	W_i (kN)	h_i (m)	$W_i h_i^2$ ($\times 10^3$) (kNm ²)	$\frac{W_i h_i^2}{\sum W_i h_i^2}$	Lateral force at level i (kN)
Roof	1759.4	7	86.21	0.68	437.2
1 st	2527.4	4	40.44	0.32	205.8
Σ	4286.8		126.65		643.0


Figure 5.2: Lateral Force at Floor Levels
Distribution of Shears among different walls at the First Storey Level:
Location of the Center of Gravity

Because of symmetrical layout of east and west wall, the centre of gravity will be located on the mid line between east and west wall.

Area of north wall as shown in Figure 5.1

$$A_n = 8 \times 12 - (2.1 \times 1.2 + 1.5 \times 1.2 + 2.4 \times 2.1) \\ = 86.64 \text{ m}^2$$

Distance of north wall from the mid of north wall

$$D_n = 0 \text{ m}$$

Area of a south wall

$$A_s = 8 \times 12 - (2 \times 4.5 \times 3) \\ = 69 \text{ m}^2$$

Distance of south wall from the mid of north wall

$$D_s = 17.77 \text{ m}$$

The distance of C.G = C_g from the centre of the north wall is,

$$C_g = \frac{A_n D_n + A_s D_s}{A_n + A_s} \\ = \frac{80.64 \times 0 + 69 \times 17.77}{86.64 + 17.77} = 7.88 \text{ m}$$

Determination of the Stiffness of the exterior Walls:

$$k_F = \frac{E_m t}{\left[\left(\frac{H}{L} \right)^3 + 3 \frac{H}{L} \right]} \text{ for fixed wall or pier}$$

$$k_C = \frac{E_m t}{\left[4 \left(\frac{H}{L} \right)^3 + 3 \left(\frac{H}{L} \right) \right]} \text{ for cantilever wall}$$

The equations above represent the total stiffness (flexure + shear) of a masonry wall.

Here the piers and walls are assumed to be fixed at both ends and corresponding stiffness calculation is shown in the tabular form in Table 5.2

Modulus of elasticity

$$E_m = 550 \times f_m = 5.5 \times 10^3 \text{ MPa}$$

Table 5-2: Calculation of stiffness of piers and walls

Pier No.	Length L (m)	H (m)	H/L	k_i (MN/m)
1	1.10	2.10	1.91	99.72
2	1.10	1.20	1.09	276.74
3	2.90	1.20	0.41	964.01
4	1.80	2.10	1.17	248.63
5	0.75	3.00	4.00	16.64
6	1.50	3.00	2.00	90.36
7	0.75	3.00	4.00	16.64
East wall	18.00	4.00	0.22	1866.77
West wall	18.00	4.00	0.22	1866.77
North Wall	12.00	$k_1 + k_2 + k_3 + k_4$		1589.10
South wall	12.00	$k_5 + k_6 + k_7$		123.65

For several piers connected along their tops, stiffness of the wall is determined as

$$k = \sum_{i=1}^n k_i = k_1 + k_2 + \dots + k_n$$

Note: Pier stiffness is equal to the force required to produce a unit deflection.

Location of the Center of Stiffness:

Because of symmetrical layout of east and west wall and symmetrical configuration, the centre of stiffness will be located on the mid line between east and west wall and hence there will be no static eccentricity when load is in north-south direction.

The distance of C.S = C_s from the centre of the north wall is $C_s = \frac{k_n \times D_n + k_s D_s}{k_n + k_s}$

$$= \frac{1589.10 \times 0 + 123.65 \times 17.77}{1589.10 + 123.65} = 1.28 \text{ m}$$

Static eccentricity = $7.88 - 1.28 = 6.6 \text{ m}$

When load is in East-West direction design eccentricity = $1.5 \times 6.6 + 0.05 \times 18 = 10.8 \text{ m}$

When load is in North-South direction design eccentricity = $0.05 \times 12 = 0.6 \text{ m}$

(clause 7.9.2 of IS: 1893: (Part 1) 2002)

So, torsional moment developed due to eccentricity $M_T = V_B \times 10.8 = 643 \times 10.8$

$$= 6944.4 \text{ kNm}$$

The calculations for distribution of lateral shear to the walls are shown in the Table 5-3.

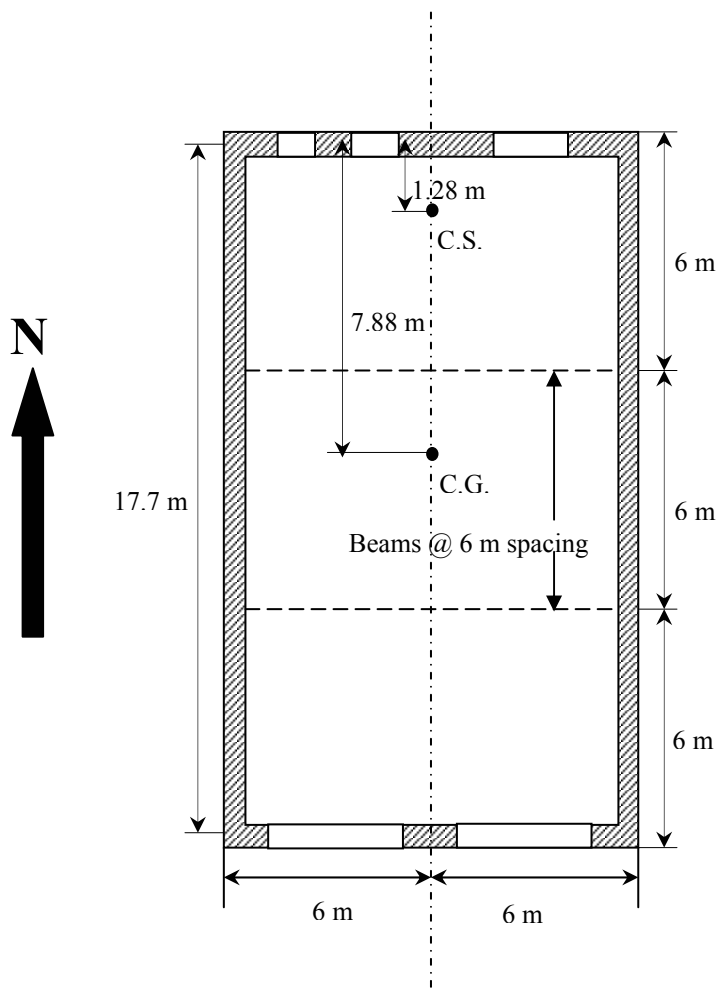


Figure 5.3: Plan view of the building at Ground Floor

Table 5-3: Distribution of Translational and Torsional shears at Ground Floor when load is in E-W direction

Wall	Stiffness k_i (MN/m)	D_y (m)	$k_i D_y$	$k_i D_y^2$	$V = \frac{k_i}{\sum k_i} \times V_B$ (kN)	$V_m = \frac{k_i D}{\sum k_i D^2} \times M_T$ (kN)	$V_T = V_m + V$ (kN)
North	1589.10	1.28	2034.05	2603.58	596.58	85.33	681.91
South	123.65	16.49	2038.93	33622.00	46.42	85.53	131.95
Σ	1712.75			36225.58	643.00		
		D_x (m)	$k_i D_x$	$k_i D_x^2$			
East	1866.77	5.89	10985.95	64652.31	0.00	460.89	460.89
West	1866.77	5.89	10985.95	64652.31	0.00	460.89	460.89
Σ	3733.54			129304.62			
			$\Sigma k_i D^2$	165530.20			

Calculation of Dead and Live Loads:

For this example purpose only the north wall is analyzed and dead load and live loads are calculated on the north wall only.

Figure 5.3 show that the building has beams in the lateral direction at a spacing of 6 m.

So, loads from $3 \times 12 = 36 \text{ m}^2$ area will be transferred to north and south walls.

Dead load from slabs on these walls

$$= 36 \times (5 + 5) = 360 \text{ kN}$$

Considering, a section in north wall at a height of 2.1 m from ground.

So, dead load from the wall above

$$= 5.9 \times 12 \times 4.6 = 325.7 \text{ kN}$$

So, total dead load on the North wall

$$= 360 + 325.7 = 685.7 \text{ kN}$$

Live load from slabs on these walls

$$= 1.5 \times 36 + 5 \times 36 = 234 \text{ kN}$$

Calculation of Geometric Properties:

The geometric property of the piers in the north wall are calculated corresponding to the figures given in Figure 5.1 and 5.3 and is given in the tabular form in Table 5-4.

Table 5-4: Geometric properties of the piers in north wall

Pier No.	Length L (m)	Height (m)	Area (m ²)	c_i (m)	$(I_g)_i$ (m ⁴)
1	1.10	2.10	0.25	0.55	0.03
2	1.10	1.20	0.25	0.55	0.03
3	2.90	1.20	0.67	1.45	0.47
4	1.80	2.10	0.41	0.90	0.11

Distribution of Lateral Shear among Piers:

As the floor is assumed to be rigid the lateral forces will be distributed in proportion to the pier stiffness $V_{ei} = V_b \frac{k_i}{\sum k_i}$. The distribution of shear among piers is shown in the Table 5-5.

Total base Shear to the north wall = 681.91 kN

Table 5-5: Distribution of Lateral Shear among Piers

Pier No.	k_i (kN/m)	$\frac{k_i}{\sum k_i}$	$V_b \frac{k_i}{\sum k_i}$ (kN)
1	1×10^5	0.06	40.91
2	2.77×10^5	0.17	115.92
3	9.64×10^5	0.61	415.97
4	2.49×10^5	0.16	109.11
Σ	15.9×10^5	1.00	681.91

Distribution of Overturning Moment to Piers as Axial Forces:

Location of Neutral axis

Table 5-6: Location of Neutral axis

Pier No.	y_i (m)	A_i (m ²)	$A_i \times y_i$ (m ³)
1	0.55	0.25	0.14
2	2.85	0.25	0.72
3	6.35	0.67	4.24
4	11.1	0.41	4.60
Σ		1.58	9.69

$$y_{NA} = \frac{\sum A_i \times y_i}{\sum A_i} = \frac{9.69}{1.58} = 6.11 \text{ m}$$

Distance of the central line of the piers from N.A.

Pier 1:

$$(y_b)_1 = y_{NA} - y_1 = 6.11 - 0.55 = 5.56 \text{ m}$$

Pier 2:

$$(y_b)_2 = y_{NA} - y_2 = 6.11 - 2.85 = 3.26 \text{ m}$$

Pier 3:

$$(y_b)_3 = y_{NA} - y_3 = 6.11 - 6.35 = -0.24 \text{ m}$$

Pier 4:

$$(y_b)_4 = y_{NA} - y_4 = 6.11 - 11.1 = -4.99 \text{ m}$$

Vertical distribution of lateral force will be according to the same distribution factor as in case of the whole building.

Lateral force applied at the roof level

$$= 0.68 \times 681.91 = 463.70 \text{ kN}$$

and at first floor level

$$= 0.32 \times 681.91 = 218.21 \text{ kN}$$

Total Overturning Moment

$$= (463.70 \times 7 + 218.21 \times 4) = 4118.74 \text{ kNm}$$

Moment of inertia of the whole wall section

$$= I_{NA} = \sum (I_{NA})_i$$

where, $(I_{NA})_i = A_i (y_b)_i^2 + (I_g)_i$

Axial load due to overturning moment M

$$= (P_e)_i = (DF_m)_i \times M$$

where, $(DF_m)_i = \frac{A_i (y_b)_i}{\sum I_{NA}}$

The calculations are shown in the tabular form in Table 5-7.

Table 5-7: Distribution of Overturning Moment to Piers as Axial Forces

Pier No.	A_i (m ²)	$(y_b)_i$ (m)	$A_i (y_b)_i^2$ (m ⁴)	$(I_g)_i$ (m ⁴)	$(I_{NA})_i$ (m ⁴)	$A_i \times (y_b)_i$ (m ³)	$(DF_m)_i$ (m ⁻¹)	$(P_e)_i$ (kN)
1	0.25	5.56	7.82	0.03	7.85	1.41	0.07	288.31
2	0.25	3.26	2.69	0.03	2.71	0.82	0.04	164.75
3	0.67	-0.24	0.04	0.47	0.51	-0.16	-0.01	-41.18
4	0.41	-4.99	10.31	0.11	10.42	-2.07	-0.10	-411.88
Σ	1.59		20.86	0.63	21.49	0.00	0.00	0.00

Distribution of Direct Axial Compression

Axial load due to gravity loads are distributed in proportion to the tributary wall length.

Total dead load = $P_d = 685.7 \text{ kN}$

$$(P_d)_1 = 685.7 \times \frac{(1.1 + 0.6)}{12} = 97.14 \text{ kN}$$

$$(P_d)_2 = 685.7 \times \frac{(0.6 + 1.1 + 0.75)}{12} = 140.0 \text{ kN}$$

$$(P_d)_3 = 685.7 \times \frac{(0.75 + 2.9 + 1.2)}{12} = 277.14 \text{ kN}$$

$$(P_d)_4 = 685.7 \times \frac{(1.2 + 1.8)}{12} = 171.42 \text{ kN}$$

Total live load =

$$P_l = 234 \text{ kN}$$

$$(P_l)_1 = 234 \times \frac{(1.1 + 0.6)}{12} = 33.15 \text{ kN}$$

$$(P_l)_2 = 234 \times \frac{(0.6 + 1.1 + 0.75)}{12} = 47.78 \text{ kN}$$

$$(P_l)_3 = 234 \times \frac{(0.75 + 2.9 + 1.2)}{12} = 94.57 \text{ kN}$$

$$(P_l)_4 = 234 \times \frac{(1.2 + 1.8)}{12} = 58.5 \text{ kN}$$

Summary of Pier Forces

The loads transferred to individual piers are shown in tabular form in Table 5-8.

Table 5-8: Forces in Different piers due to different Loads

Pier No.	$(P_d)_i$ (kN)	$(P_l)_i$ (kN)	$(P_e)_i$ (kN)	$(V_e)_i$ (kN)	$(M_e)_i$ (kNm)
1	97.14	33.15	288.31	40.91	42.96
2	140.00	47.78	164.75	115.92	69.55
3	277.14	94.57	-41.18	415.97	249.58
4	171.42	58.50	-411.88	109.11	114.57
Σ	685.7	234.00	0.00	681.91	

Note : As the piers are assumed to be fixed at both ends $M_{ei} = V_{ei} \times \frac{h_i}{2}$.

Load Combinations

Three probable load cases have been considered to cause failure and each has some potential failure mode which will govern the failure in that particular load combination.

Table 5-9: Load Combinations

Load Case	Axial Comp. Force	Moment	Shear
1	$P1_i = (P_d)_i + (P_l)_i$		
	Results in large axial forces.		
2	$P2_i = 0.75 \times ((P_d)_i + (P_l)_i + (P_e)_i)$	$M2_i = 0.75(M_e)_i$	$V2_i = 0.75(V_e)_i$
	Results in large axial and bending force for lateral force.		
3	$P3_i = 0.75 \times (0.9 \times (P_d)_i - (P_e)_i)$	$M3_i = 0.75(M_e)_i$	$V3_i = 0.75(V_e)_i$
	Result in low axial load which causes low moment capacity.		

Considering the above combinations we are finding out the final loads on different piers and that is shown in Table 5-10.

Table 5-10: Loads on piers under different Load combinations

Pier No.	$P1_i$ (kN)	$P2_i$ (kN)	$P3_i$ (kN)	$M2_i$ (kNm)	$V3_i$ (kN)
1	130.3	313.9	-151.1	33.2	30.7
2	187.8	264.4	-29.8	52.2	86.9
3	371.7	309.7	154.8	187.2	312.0
4	229.9	418.4	-194.1	85.9	81.8

Check for Pier Axial and Flexural Stresses

Axial and flexural stresses in the piers are checked assuming the piers are un-reinforced and un-cracked.

Load case 1: (D+L)

Given, $f_m = 10$ MPa

Allowable basic compressive stress

$$f_{bc} = 0.25 f_m = 0.25 \times 10 = 2.5 \text{ MPa}$$

As per clause 4.6.1 of IS: 1905-1987

$$\text{Slenderness Ratio} = SR_i = \frac{h_i}{\text{thickness}} = \frac{h_i}{0.23}$$

Stress reduction factor $((k_s)_i)$ will be corresponding to the corresponding slenderness ratio as per clause 5.4.1.1 and Table 9 of IS: 1905-1987.

Area reduction factor (k_a) will be taken as per clause 5.4.1.2 IS: 1905-1987.

Shape modification factor (k_p) is taken as 1.

Allowable compressive stress

$$F_{ai} = (k_s)_i \times k_a \times k_p \times f_{bc}$$

(clause 5.4.1 of IS: 1905-1987 as shown in Table 5-11.

Table 5-11: Calculation of allowable compressive stress in different piers in direct compression

Pier No.	SR_i	$(k_s)_i$	k_a	k_p	$(F_a)_i$ (MPa)
1	9.13	0.92	1.00	1.00	2.30
2	5.22	1.00	1.00	1.00	2.50
3	5.22	1.00	1.00	1.00	2.50
4	9.13	0.92	1.00	1.00	2.30

The compressive stress developed in the piers and corresponding demand capacity ratio is checked in Table 5-12.

Table 5-12: Check of Piers in compression under load case 1

Pier No.	$P1_i$ (kN)	$f_a 1_i$ (MPa)	$(F_a)_i$ (MPa)	$\frac{f_a 1_i}{(F_a)_i}$
1	130.3	0.51	2.30	0.22
2	187.8	0.74	2.50	0.30
3	371.7	0.56	2.50	0.22
4	229.9	0.56	2.30	0.24

In each of the pier the demand capacity ratio is less than 1, hence piers are safe in axial compression under load case 1.

Load case 2: 0.75(D+L+E)

In this case compressive stress due to both direct compression and bending and their combined action is checked as shown in Table 5-13, 5-14 and 5-15.

$$\text{Axial component} = f_a 2_i = \frac{P1_i}{A_i}$$

Table 5-13: Check of Piers in direct compression under load case 2

Pier No.	$P2_i$ (kN)	$f_a 2_i$ (MPa)	$(F_a)_i$ (MPa)	$\frac{f_a 2_i}{(F_a)_i}$
1	313.9	1.26	2.30	0.55
2	264.4	1.06	2.50	0.42
3	309.7	0.46	2.50	0.19
4	418.4	1.17	2.30	0.51

$$\text{Bending component} = f_a 2_i = \frac{M2_i}{(S_g)_i}$$

Where, $(S_g)_i = \frac{(I_g)_i}{(y_i)_{\max}}$ = section modulus of pier

Allowable compressive stress in bending = $(F_b)_i = 1.25 \times (F_a)_i$

Table 5-14: Check of Piers in bending compression under load case 2

Pier No.	$(S_g)_i$ (m ³)	$M2_i$ (kNm)	$f_b 2_i$ (MPa)	$(F_b)_i$ (MPa)	$\frac{f_b 2_i}{(F_b)_i}$
1	0.046	33.2	0.70	2.88	0.24
2	0.046	52.2	1.13	3.13	0.36
3	0.322	187.2	0.58	3.13	0.19
4	0.124	85.9	0.67	2.88	0.23

The Moment causes both of compression and tension in the piers but here only compressive value is checked in this load case as the effect of tension will be more in the load case 3. So here in Table 5-15 we are checking for simultaneous action of compressive force due to direct axial force and bending action.

Table 5-15: Check in combined action

Pier No.	$\frac{f_a 2_i}{(F_a)_i} + \frac{f_b 2_i}{(F_b)_i}$
1	0.79
2	0.78
3	0.38
4	0.74

In each of the pier the demand capacity ratio is less than 1, hence piers are safe in axial compression, bending compression and their combined action under load case 2.

Load case 3: 0.75(0.9D - E)

In this case minimum axial stress is checked against allowable tensile stress as shown in Table 5-16 and 5-17.

Allowable compressive stress in bending

$$= F_{ti} = 0.07 \text{ MPa}$$

Table 5-16: Check of Piers in direct axial force under load case 3

Pier No.	$P3_i$ (kN)	$f_a 3_i$ (MPa)	$(F_a)_i$ (Mpa)	$\frac{f_a 3_i}{(F_a)_i}$
1	-151.1	-0.60	-0.07	8.57
2	-29.8	-0.12	-0.07	1.71
3	154.8	0.23	2.50	0.09
4	-194.1	-0.47	-0.07	6.71

Considering that moment will cause axial tension and tensile stress of $f_b 3_i$ and axial force due to dead load will cause axial compression and their resultant will act upon the pier given in the table below. (+ Sign denotes tensile stress)

$$f 3_i = f_b 3_i - f_a 3_i$$

Table 5-17: Check of Piers in axial tension under load case 3

Pier No.	S_{gi} (m ³)	$M 2_i$ (kNm)	$f_b 3_i$ (MPa)	$f 3_i$ (MPa)	$\frac{f 3_i}{(F_t)_i}$
1	0.046	33.2	0.70	1.30	18.57
2	0.046	52.2	1.13	1.25	17.86
3	0.322	187.2	0.58	0.35	5.00
4	0.124	85.9	0.67	1.16	16.57

In each of the pier the demand capacity ratio is more than 1, hence piers are not safe in axial action and combined action of axial action and tension due to bending under load case 3.

So, reinforcement is to be provided in all piers.

This 3rd loading combination gives the highest value of demand capacity ratio among all the potential cases which may cause failure to the piers.

Example 6– SEISMIC ANALYSIS AND DESIGN OF CONCRETE MASONRY SHEAR WALL BUILDING

Problem Statement:

One story masonry building with wood framed roofs and masonry shear walls is shown in Figure 6.1 and 12.2. Concrete masonry units of 400×200×200 mm size are used.

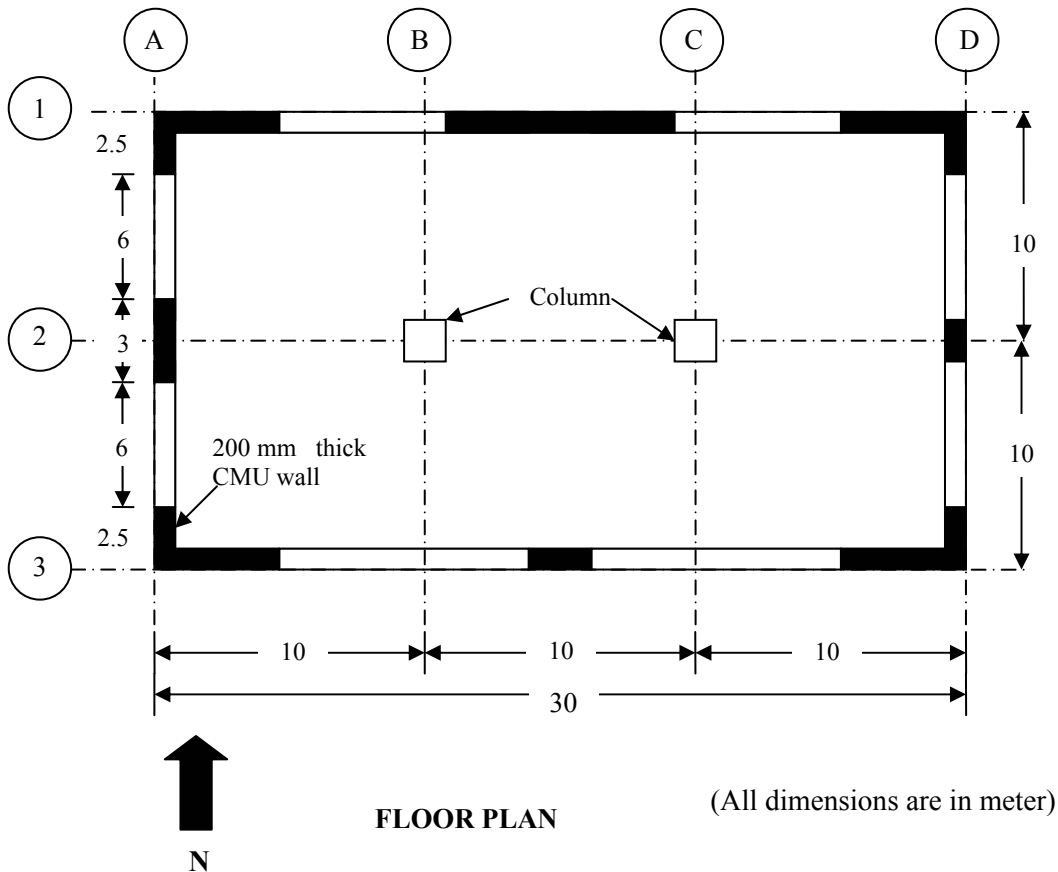
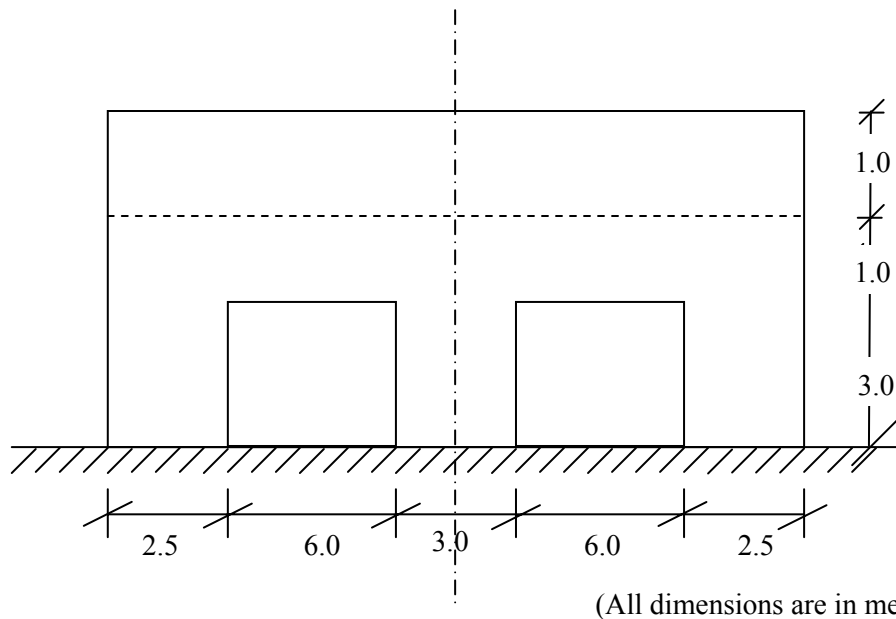


Figure 6.1: Plan of masonry building

This example will illustrate the following parts of the design process.

1. Design base shear coefficient.
2. Base shear in the transverse direction.
3. Shear in wall on line A.
4. Design 200 mm shear wall on line A for out-of-plane seismic forces.
5. Design 200 mm shear wall on line A for in-plane seismic forces.
6. Design 200 mm shear wall on line A for flexure and in-plane forces.
7. Chord (bond beam at roof level) design.


Figure 6.2: ELEVATION OF WALL ON LINE A

Solution:

Given Data:

Building dimensions:

Length along EW direction: $L = 30$ m

Length along NS direction: $B = 20$ m

Overall height of wall, $H = 5.0$ m

Roof Height, $h = 4.0$ m

Roof weights:

Roofing 360 N/m²

12.5mm plywood 70 N/m²

Roof framing 200 N/m²

Mechanical/electrical 70 N/m²

Insulation 70 N/m²

Total Dead load 800 N/m²

Roof live load 950 N/m²

Exterior 200 mm CMU walls

(Fully grouted, light-weight masonry)

Unit weight of masonry of 200 mm thick,

$$w_w = 20000 \times 0.2 = 4000 \text{ N/m}^2$$

Crushing Strength of masonry, $f_m = 15$ MPa

Permissible tensile stress of HYSD steel,

$$f_s = 230 \text{ MPa}$$

Seismic and site data:

Zone factor $Z = 0.36$ (Seismic Zone V)

Importance factor $I = 1.0$

Response reduction factor $R = 4.0$

(Special reinforced masonry wall)

Design spectral acceleration, $\frac{S_a}{g} = 2.5$

Design Base Shear Coefficient

As per IS: 1893, base shear coefficient is given by

$$A_h = \frac{Z \times I \times S_a}{2 \times R \times g} = \frac{0.36 \times 1.0 \times 2.5}{2 \times 4.0} = 0.113$$

Base Shear in Transverse Direction

The building has a flexible roof diaphragm and heavy CMU walls. The diaphragm spans as a simple beam between resisting perimeter walls in both directions and will transfer 50 percent of the diaphragm shear to each resisting wall. The building weight calculation is separated in three portions: the roof, longitudinal walls, and transverse walls for ease of application at a larger stage in the calculations. The reason to separate the CMU wall masses is because masonry walls that resist ground motion parallel to their in-plane direction resist their own seismic inertia without transferring seismic forces into the roof diaphragm.

Weight of roof is, $W_{roof} = 360 \times 30 \times 20 = 216 \text{ kN}$

For longitudinal wall weights (out-of-plane walls), the upper half of the wall weight is tributary to the roof diaphragm. Neglecting the openings in the top half of walls, weight of longitudinal walls is

$$W_{we} = 2 \times 4000 \times 30 \times \left(\frac{4}{2} + 1 \right) = 720 \text{ kN}$$

For forces in the transverse direction, seismic inertial forces from the transverse walls (lines A and D) do not transfer through the roof diaphragm.

Therefore, the effective diaphragm weight in the north-south direction is

$$W_{td} = W_{roof} + W_{we} = 216 + 750 = 936 \text{ kN}$$

The transverse seismic inertial force (shear force), which is generated in the roof diaphragm is calculated as follows:

$$V_{td} = A_h \times W_{td} = 0.113 \times 966 = 105.3 \text{ kN}$$

Seismic inertial force (shear force), which is generated in the transverse walls (in-plane walls) is calculated using full weight (and height) of the walls (with openings ignored for simplicity)

$$V_{tw} = 2 \times 0.113 \times 4000 \times 5 \times 20 = 90 \text{ kN}$$

Design base shear in the transverse direction is the sum of the shears from the roof diaphragm shear and the masonry walls in-plane forces.

$$V_{trans} = V_{td} + V_w = 108.7 + 90 = 195.3 \text{ kN}$$

Shear wall on line A

The seismic shear tributary to the wall on line A comes from the roof diaphragm (transferred at the top of the wall) and the in-plane wall inertia forces:

$$V_a = \frac{V_{td} + V_{tw}}{2} = \frac{108.7 + 90}{2} = 97.65 \text{ kN}$$

Design 200 mm shear wall on line A for out-of-plane seismic forces

Length of shear wall, $l_{sa} = 3.0 \text{ m}$

The shear wall 200 mm thick on line A is a bearing wall and must support gravity loads. It must be capable of supporting both gravity and out-of-plane seismic loads, and gravity plus seismic forces at different instants of time

depending on the direction of seismic ground motion.

Calculation of Vertical Loads

Gravity loads from roof framing tributary to the shear wall at line A is given by

$$P_{gdl} = 200 \times \frac{30}{3 \times 2} \times \frac{20}{2} = 10 \text{ kN}$$

Wall load on 3 m wall at mid height,

$$P_{wdl} = 4000 \times 3 \times \left(1 + \frac{4}{2} \right) = 36 \text{ kN}$$

Dead load from lintels,

$$P_{ldl} = 4000 \times (5 - 3) \times \frac{6}{2} = 24 \text{ kN}$$

Total dead load, $P_{dl} = P_{gdl} + P_{wdl} + 2 \times P_{ldl}$

$$\begin{aligned} \text{Total live load, } P_{ll} &= 950 \times \frac{30}{2 \times 3} \times \frac{20}{2} = 47.5 \text{ kN} \\ &= 10 + 36 + 2 \times 24 = 94 \text{ kN} \end{aligned}$$

Calculation of Seismic Loads

Out-of-plane seismic forces are calculated as the average of the wall element coefficients at the base of the wall and the top of the wall.

Seismic load at base ($h_{xb} = 0$)

$$\begin{aligned} F_{pb} &= 0.5 \times Z \times I \times \left(1 + \frac{h_{xb}}{h} \right) \times w_w \\ &= 0.5 \times 0.36 \times 1.0 \times \left(1 + \frac{0}{4} \right) \times 4000 = 720 \text{ N/m}^2 \end{aligned}$$

Seismic load at roof ($h_{xr} = 4.0 \text{ m}$)

$$\begin{aligned} F_{pr} &= 0.5 \times Z \times I \times \left(1 + \frac{h_{xr}}{h} \right) \times w_w \\ &= 0.5 \times 0.36 \times 1.0 \times \left(1 + \frac{4}{4} \right) \times 4000 = 1440 \text{ N/m}^2 \end{aligned}$$

Average seismic load,

$$F_p = \frac{F_{pr} + F_{pb}}{2} = \frac{720 + 1440}{2} = 1080 \text{ N/m}^2$$

Calculations of wall moments due to out-of-plane forces are done using standard beam formula for a propped cantilever.

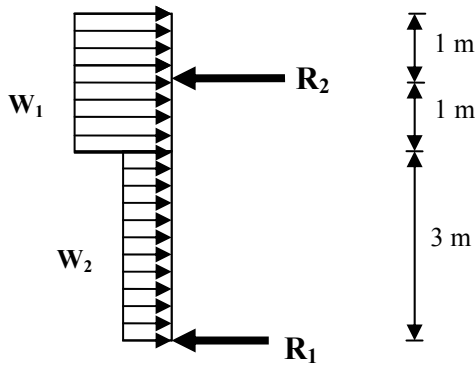


Figure 6.3: Propped cantilever loading diagram

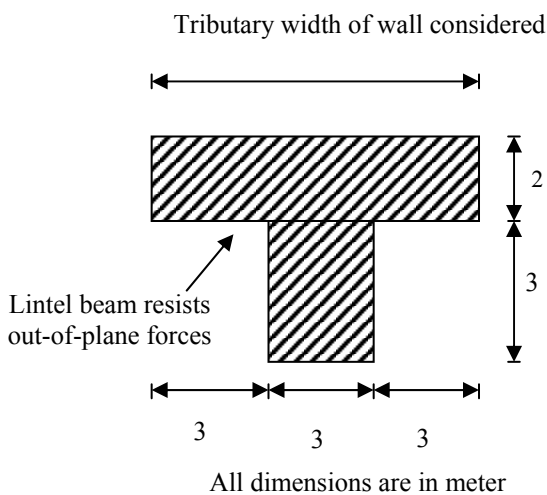


Figure 6.4: Tributary width of wall for out of plane seismic inertial force calculation

Figure 6.3 shows the loading diagram for wall out-of-plane loading and figure 6.4 for tributary widths walls used to determine the loading diagram.

$$W_1 = \left(\frac{6.0}{2} \times 2 + 3.0 \right) \times 1080 = 9.72 \text{ kN/m}$$

$$W_2 = (3.0) \times 1080 = 3.24 \text{ kN/m}$$

Maximum bending moment, $M_{out} = 5.74 \text{ kNm}$

On account of 33% in allowable stress level, the load level is reduced to 75%.

$$M_{out} = 0.75 \times 5.74 \text{ kNm} = 4.31 \text{ kNm}$$

Axial stress is given by

$$f_a = \frac{0.75 P d l}{Area} = \frac{0.75 \times 94 \times 10^3}{2 \times 3.0 \times (0.003)} = 3.92 \text{ MPa}$$

Allowable axial stress is given by

$$F_a = 0.25 \times 15 = 3.75 \text{ MPa}$$

Allowable bending stress of masonry is

$$F_b = 1.25 \times F_a = 1.25 \times 3.75 = 4.69 \text{ MPa}$$

Provide 10 mm bars at the center of wall section.

Effective depth, $d = 200/2 = 100 \text{ mm}$

Assume, $j = 0.9$

Area of steel required,

$$A_s = \frac{M_{out}}{f_s \times j \times d} = \frac{4.31 \times 10^6}{230 \times 0.9 \times 100} \text{ mm}^2$$

$$= 157 \text{ mm}^2$$

Number of bars, $n = 157/78 = 2.1$

Provide 4 bars of 10 mm.

Area of steel provided, $A_s = 314 \text{ mm}^2$

Spacing of bars, $s = 333 \text{ mm}$

Calculate Effective Width

As per Section 6.1.1, the effective width is given by the least of the following

- i. c/c spacing of bars = 333 mm (Governs)
- ii. 6 times wall thickness = $6 \times 200 = 1200 \text{ mm}$

Check for Stresses

Design moment, $M_{out} = 4.31 \text{ kNm}$

Young's modulus of steel, $E_s = 2 \times 10^5 \text{ MPa}$

Young's modulus of masonry,

$$E_m = 550 \times f_m = 550 \times 15 = 8.25 \times 10^3 \text{ MPa}$$

$$n = \frac{E_s}{E_m} = \frac{2 \times 10^5}{8.25 \times 10^3} = 24$$

Percentage of steel is given by

$$\rho = \frac{314}{3.0 \times 100} = 0.001$$

Solving for actual value of 'k'

$$k^2 + 2 \times \rho \times k \times n - 2 \times \rho \times n = 0$$

$$k = 0.205$$

$$j = 1 - k/3 = 1 - 0.205/3 = 0.93$$

Allowable tension flexural stress limit on moment is given by

$$M_t = A_s \times j \times d \times f_s$$

$$= 314 \times 0.93 \times 230 \times 100$$

$$= 6.72 \text{ kNm} (> 4.31 \text{ kNm})$$

Allowable compression flexural stress limit on moment is given by

$$\begin{aligned} M_c &= 0.5 \times l_{sa} \times j \times k \times d^2 \times F_b \\ &= 0.5 \times 3000 \times 0.93 \times 0.205 \times 100^2 \times 4.69 \\ &= 13.43 \text{ kNm} (> 5.74 \text{ kNm}) \text{ (OK)} \end{aligned}$$

Design the Shear Wall on Line a for In-Plane Seismic Forces

Shear Force Distribution

The shear force on line A must be distributed to three shear wall piers (2.5 m, 3.0 m, and 2.5 m width respectively) in proportion to their relative rigidities. This can be accomplished by assuming that the walls are fixed at the tops 3 m deep lintel. The relative rigidities of the piers are calculated as follows:

$$D_1 = 2.5 \text{ m}; \quad D_2 = 3.0 \text{ m}; \quad D_3 = 2.5 \text{ m}$$

$$H_1 = H_2 = H_3 = 5.0 \text{ m}$$

Thickness of masonry units resisting shear, $t_u = 0.003 \text{ m}$

$$\Delta_i = \frac{1}{E_m t_u} \left[\left(\frac{H_i}{D_i} \right)^3 + 3 \times \frac{H_i}{D_i} \right]; \quad k_i = 1/\Delta_i$$

$$k_{ri} = \frac{k_i}{\sum k_i}$$

The details of calculation of relative rigidity is shown in table 6-1

Table 6-1: Calculation of relative rigidity

Pier	Width of pier (m)	Shear deflection, Δ_i (mm)	Stiffness, k_i (1/mm)	Relative Rigidity, k_{ri}
1	2.5	57	1.77	0.29
2	3.0	39	2.57	0.42
3	2.5	57	1.77	0.29

Design shear force for pier 2 is given by,

$$V_d = k_{r2} \times V_a = 0.42 \times 97.65 \text{ kN} = 41.04 \text{ kN}$$

Shear stress is given by $f_v = 0.06 \text{ MPa}$

In view of 33% increase in allowable stress level due to wind/earthquake load, the load level is reduced to 75%.

Bending moment is given by

$$M = 0.75 \times V_d \times l_{sa} / 2$$

$$= 0.75 \times 41.04 \times 3.0 / 2 = 46.24 \text{ kNm}$$

$$\frac{M_1}{V_s \times l_{sa}} = \frac{46.24 \times 10^6}{41.04 \times 3000} = 0.375 (< 1.0)$$

Allowable shear stress is given by

$$F_v = \frac{1}{24} \left(4 - \frac{M_1}{V_s \times l_{sa}} \right) = 0.151 \text{ MPa}$$

Maximum allowable shear stress is given by

$$F_{vm} = \left(0.6 - \frac{0.2 \times M_1}{V_s \times l_{sa}} \right) = 0.525 \text{ MPa}$$

Since the calculated shear is less than the allowable shear, the wall section is safe in shear.

Design the Shear Wall on Line A for Combined Axial and In-Plane Bending Forces.

Following load combinations are adopted in this section with 100% permissible stress values which is equivalent of 33% increase in permissible stress values for wind/earthquake load combinations.

- i) 0.75(D+L+E) (floor live load = 0)
- ii) 0.75(0.9D+E) i.e., 0.67D-0.75E

(i) Load Case 1

Axial load,

$$P_1 = 0.75 \times P_{dl} = 0.75 \times 94 \text{ kN} = 70.5 \text{ kN}$$

Bending moment is given by

$$\begin{aligned} M_1 &= 0.75 \times V_s \times l_{sa} / 2 \\ &= 0.75 \times 41.04 \times 3.0 / 2 = 46.24 \text{ kNm} \end{aligned}$$

Assume 2 bars are provided at end section.

Effective depth is given by

$$d_e = 3.0 - 0.2 = 2.8 \text{ m}$$

$$M_1 / P_1 d_e = 0.234$$

Distance of axial load from edge of section,

$$\Delta = \frac{\frac{3000}{2} - 200}{2800} = 0.464$$

$$\frac{2}{3} - \Delta = \frac{2}{3} - 0.464 = 0.202 (< M_1 / P_1 d_e)$$

Hence, region 3 of P-M interaction curve applies. (Refer to appendix on P-M interaction curve of masonry)

Iteration Method:

Assume, $a = 500$ mm

$f_s = 230$ MPa

$$M_p = P_1 \times \left(\frac{l_{sa}}{2} - a \right)$$

$$= 70.5 \times \left(\frac{3000}{2} - 500 \right)$$

$$= 70.5 \text{ kNm}$$

$$A_s = \frac{M_p - M_1}{f_s \times (d - a)} = \frac{70.5 - 46.24}{230 \times (2800 - 500)}$$

$$= 45.87 \text{ mm}^2$$

$$\zeta = \frac{(P_1 + A_s f_s) n}{f_s b} = 1.08 \text{ m}$$

$$a = \frac{\sqrt{\zeta^2 + 2d\zeta} - \zeta}{3} = 534 \text{ mm } (>500 \text{ mm})$$

Iterating in similar manner, we get $a = 532$ mm

Provide 1- 8 mm HYSD bar.

Area of steel provided is 50 mm^2

(ii) Load Case 2

Axial load,

$$P_2 = 0.67 \times Pdl = 0.67 \times 94 \text{ kN} = 63 \text{ kN}$$

Bending moment is given by

$$M_2 = 0.75 \times V_s \times l_{sa} / 2$$

$$= 0.75 \times 41.82 \times 3.0 / 2 = 47.04 \text{ kNm}$$

Assume 2 bars are provided at end section.

Effective depth is given by

$$d_e = 3.0 - 0.2 = 2.8 \text{ m}$$

$$M_1 / P_1 d_e = 0.195$$

Distance of axial load from edge of section,

$$\Delta = \frac{\frac{3000}{2} - 200}{2800} = 0.464$$

$$\frac{2}{3} - \Delta = \frac{2}{3} - 0.464 = 0.202 \quad (> M_1 / P_1 d_e)$$

Hence, region 2 of P-M interaction curve applies.

$$M_w = P_2 (1 - \Delta) d + \frac{2}{3} \left(\frac{P_2^2}{1.25 F_a d} \right)$$

$$= 96.9 \text{ kNm} \quad (\text{OK})$$

Hence, the section is satisfactory

Design of Chords

Analysis of transverse roof diaphragm chords is determined by calculation of the diaphragm simple span moment by the diaphragm depth.

$$w_{dt} = \frac{V_{td} + V_{tw}}{L} = \frac{108.7 + 90}{30} = 6.51 \text{ kNm}$$

Diaphragm bending moment,

$$M_{diaph} = \frac{w_{dt} \times L^2}{8} = \frac{6.51 \times 30^2}{8}$$

$$= 732.38 \text{ kNm}$$

Axial force in the chords (Figure 6.5),

$$T_u = M_{diaph} / B = 732.38 / 20 = 36.62 \text{ kN}$$

Area of reinforcement required,

$$A_s = T_u / f_s = 36.62 \times 10^3 / 230 \times 1.33 = 159 \text{ mm}^2$$

Provide 2-10 mm dia. bars to resist chord forces.

Place the chord bars close to roof diaphragm.

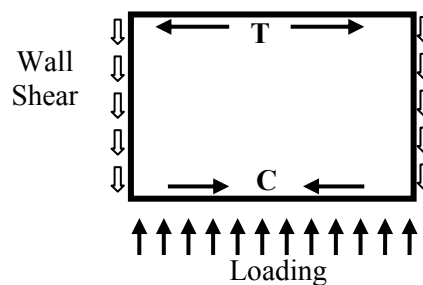


Figure 6.5: Diaphragm with loading