Analysis and Design of Earthquake Resistant Masonry Building

A Project Submitted
In Partial Fulfillment of the Requirements
For the Degree of

Bachelor of Technology
In Civil Engineering

By

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&
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DEPARTMENT OF CIVIL ENGINEERING
NATIONAL INSTITUTE OF TECHNOLOGY ROURKELA
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Under the Guidance of
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NATIONAL INSTITUTE OF TECHNOLOGY
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CERTIFICATE

This is to certify that the project entitled "**ANALYSIS AND DESIGN OF EARTHQUAKE RESITANT MASONRY BUILDINGS"** submitted by Miss Amrapali Bhowmik [Roll no. 10401003] and Miss Swarna Prava Mohanty [Roll no. 10401026] in partial fulfillment of the requirements for the award of bachelor of technology degree in Civil engineering at the National Institute of Technology Rourkela (deemed University) is an authentic work carried out by them under my supervision and guidance.

To the best of my knowledge the matter embodied in the project has not been submitted to any other university/institute for the award of any degree or diploma.

Date: 09-05-2008

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CONTENTS

LIST OF FIGURES

TABLES

ABSTRACT

Masonry buildings are the most common type of construction used for all housing around the world. Masonry buildings of brick and stone are superior with respect to durability, fire resistance, heat resistance and formative effects. Because of the easy availability of materials for masonry construction, economic reasons and merits mentioned above this type of construction is employed in the rural, urban and hilly regions up to its optimum, since it is flexible enough to accommodate itself according to the prevailing environmental conditions. Although this type of construction is most oftenly preferred and most frequently employed yet it is not completely perfect regard to seismic efficiency. The post earthquake survey has proved that the masonry buildings are most vulnerable to and have suffered maximum damages in the past earthquakes. A survey of the affected areas in past earthquakes (Bhuj 2001; Chamoli 1999; Jabalpur, 1997; Killari 1993; Uttarkashi 1991 and Bihar- Nepal 1988) has clearly demonstrated that the major losses of lives were due to collapse of low strength masonry buildings. Thus this type of construction is treated as non-engineered construction and most casualties are due to the collapse of these constructions in earthquake. Moreover the plight is that even after gaining knowledge of earthquake engineering since last three decades, neither a proper method have been developed for seismic analysis and design of masonry buildings nor the topic is fairly covered in the Indian curriculum in spite of the fact that 90% of the population lives in masonry buildings. The present work is a step towards with regard to illustrate a procedure for seismic analysis and design of a masonry building. The paper gives detail procedure of the seismic analysis and design of a three stoyered masonry Residential building. The procedure is divided into several distinctive steps in order to create a solid feeling and confidence that masonry buildings may also be designed as engineered construction.

CHAPTER 1

INTRODUCTION

INTRODUCTION

Occurrences of recent earthquakes in India and in different parts of the world and the resulting losses, especially human lives, have highlighted the structural inadequacy of buildings to carry seismic loads. There is an urgent need for assessment of the building for its present condition of its components and strength of materials. Further, seismic demand on critical individual components is determined using seismic analysis methods described in IS 1893 (Part1) for lateral forces prescribed for existing buildings in terms of seismic resistance. Masonry buildings in India are generally designed on the basis of IS 1905. The procedure for seismic analysis and design of masonry buildings has still not received adequate attention in India in spite of the fact that single-most important factor of contributing maximum damage and causalities in past earthquake is the collapse of masonry buildings. The aim of this work is to illustrate a simple procedure for design of masonry building. The procedure has been presented by considering each clause as mentioned in IS 1905 and IS 4326:1993 with the help of a work out example of a three storeyed residential masonry building. The procedure is divided into several distinctive steps in order to create a solid feeling and confidence that masonry buildings may also be designed as engineered construction.

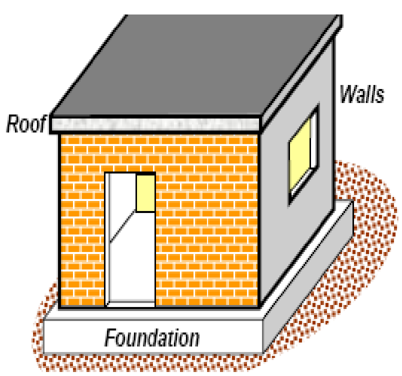
1.1 OBJECTIVE OF THE PROJECT

1. To make a study about the seismic behaviour of the masonry buildings during past earthquake and about the failure modes in case of masonry structures.
2. To make a study about the guidelines for the earthquake resistant masonry buildings according to the IS code.
3. To know about the earthquake design philosophy for an economical and safe design of a building.
4. To perform step-by-step procedure for lateral load analysis of a three storeyed masonry building. This analysis includes the determination of lateral loads by equivalent static load method, distribution of lateral loads in case of rigid and flexible diaphragms, pier analysis of shear walls with torsional effects and increase of axial load in piers of shear wall due to overturning.

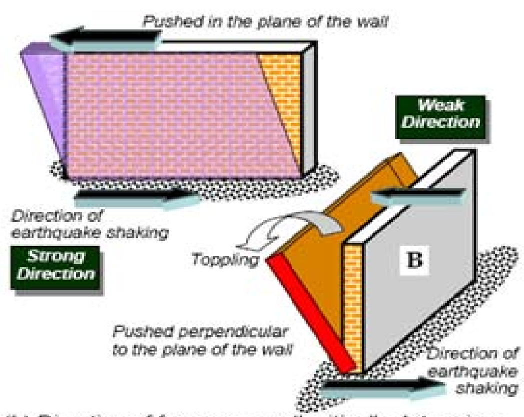
5. Design of the shear walls is done for the axial loads, moments and shear. The seismic design of the masonry building also includes the determination of vertical steel at corners and openings of shear wall for resisting the compression and flexure forces and design of lintel bands for resisting the shear forces in piers of shear walls.

1.2 LESSONS LEARNT FROM PAST EARTHQUAKE

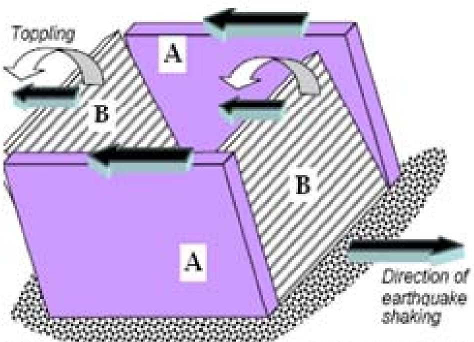
1. Ground vibrations during earthquakes cause inertia forces at locations of mass in the building. These forces travel through the roof and walls to the foundation. The main emphasis is on ensuring that these forces reach the ground without causing major damage or collapse. Lack of structural integrity is one of the principle sources of weakness responsible for severe damage leading to collapse. The failure of the connection between two walls, between walls and roofs as well was walls and foundation has been observed. Of the three components of a masonry building (*roof*, *wall* and *foundation)* the walls are most vulnerable to damage caused by horizontal forces due to earthquake. A wall topples down easily if pushed horizontally at the top in a direction perpendicular to its plane.



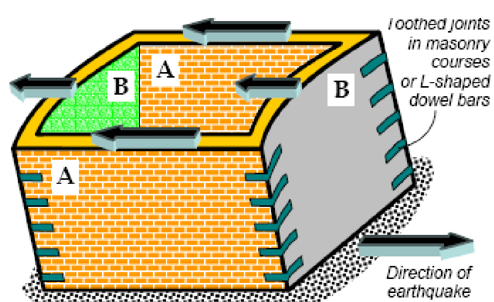
(a) Basic components of a masonry building



(b) Direction of force on a wall critically determines its earthquake performance



(a) For the direction of earthquake shaking shown, wall B tends to fail



(b) Wall B properly connected to Wall A (Note: roof is not shown): Walls A (loaded in strong direction) support Walls B (loaded in weak direction)

Damage in walls consisted of nearly vertical cracking of overhead of the openings, diagonal tension, cracking of piers between adjacent openings, separation of orthogonal walls, and partial out of plane collapse of second storey walls. Walls that are inadequately anchored to the floor diaphragm can exhibit large diagonal cracks in the piers due to in plane loads. If the mortar joints are weak, as in lime mortar, the cracks follow the joint.

2. The highest rate of damage in the buildings has been firstly, due to the failure of first and second floor projections, secondly, due to the failure of ornamental balconies and parapet walls, thirdly, due to the failure of arches over opening, and lastly, due to the collapse of improperly tied gable ends.
3. The potential out-of-plane failure of non-structural elements (parapet, veneers, gables, and unanchored walls) during earthquake constitutes the most serious life safety hazard for this type of construction. They must be given proper design consideration for lateral forces and should be braced or restrained.
4. In general, buildings with irregular plans experience more damage than rectangular buildings. The damage is often concentrated at corners due to lack of a detailed analysis that included the effects of odd shaped plans.

1.3 FAILURES MODES OF MASONRY

Vibrations caused by earthquakes generate additional loading. Shear stresses develop which cause damage to structural elements. Since masonry, which can be stressed relatively high in compression, is weak in resisting bending and shear, collapse is often the result. The different failures modes of masonry are:

a) Sliding shear failure

It results in a building sliding off its foundation or on one of the horizontal mortar joints. It is caused by low vertical load and poor mortar. If the building is adequately anchored to the foundation, the next concern is for adequate resistance of the foundation itself, in the form of some combination of horizontal sliding friction and lateral earth pressure. The dislocation of a lightly attached roof is also an example of this type of failure. A wall with poor shear strength, loaded predominantly with horizontal forces can exhibit this failure mechanism. Aspect ratio for such walls is usually 1:1 or less (1:1.5).

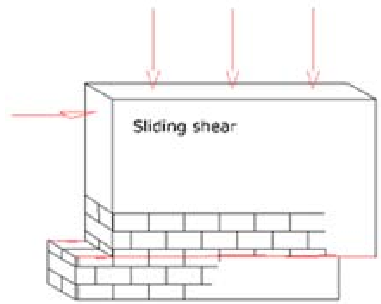


Fig 1.1 Sliding shear failure

b) Diagonal cracks

Diagonal cracks in masonry walls when the tensile stresses, developed in the wall under a combination of vertical and horizontal loads, exceed the tensile strength of the masonry material.

c) Nonstructural failure

While structural elements of a building should be the prime concern for earthquake resistance, everything in the building construction should resist forces generated by earthquakes. Nonstructural walls, suspended ceilings, window frames and fixtures should be secure against movement during the shaking actions. Failure here may not lead to building collapse, but it still constitutes danger for occupants and requires costly replacements or repair. Interior partitions, curtain walls, wall finishes, windows and similar building elements are often subjected during earthquakes to shear stresses, for which they do not have sufficient resistive strength. The most common damage resulting from this is breakage of window panes and cracks in internal plaster and external rendering. A possible remedy for the former is to isolate the window frames from the surrounding walls by the introduction of flexible joints; the latter can be avoided by reinforcing the plaster or to pre crack it by introducing control joints (groves).

Fig 1.2 Parapet failure (Non-structural failure)

d) **Failure due to overturning**

The critical nature of the overturning effect has much to do with the form of the building's vertical profile. A wall that is too tall or too long in comparison to its thickness is particularly vulnerable to shaking in its weak direction. Thus the tendency of a wall to topple when pushed in the weak direction can be reduced by limiting its length-to-thickness and height to- thickness ratios.

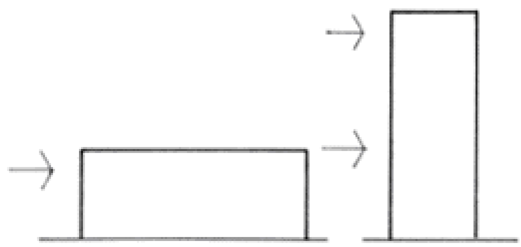


Fig 1.3 Failure due to overturning

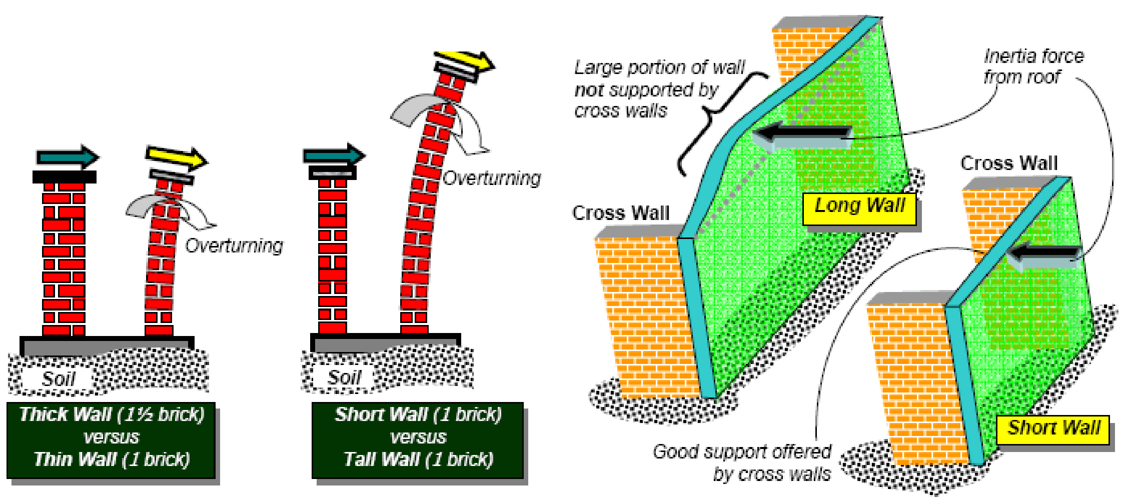


Fig 1.4 Slender walls are vulnerable – height and length to be kept within limits.

1.5 BASIC GUIDELINES OF MASONRY BUILDING

Material

Richer cement-sand mortar of 1:4 mixtures (1 part cement by 4 parts of sand) makes the masonry stronger against earthquake shaking as compared with 1:6 mortar by a factor of 2.5 to 3.0. Also 1:6 mortar is stronger than lime cinder or lime-surkhi mortar. Use of clay mud mortar produces the weakest masonry. Its strength in dry condition reduces to less than 50 percent when the walls get wet during rains. Hence, use of good plastering is essential to protect such masonry during rainy months

Building shape

Geometric irregularity of overall building shape in plan and elevation affects the seismic response of distribution, and it may also happen that certain parts of building may respond dynamically independent to the rest of the building. The plan configuration should always be symmetrical with respect to two orthogonal directions. It is also recommended that the plan should be compact and should not represent complex shapes, e.g. H, I, X, L etc. Geometrical irregularity concerned with here is the dimension of the lateral-force-resisting system, not the dimensions of the building envelope the structure by increasing ductility demands at a few locations. Also an irregular shape indicates an irregular mass. The best shapes for earthquake resistant buildings are regular shapes and preferably with two symmetry axes. In this case the centre of gravity and rigidity will be the same or close to each other and therefore there will not be any torsion in the building. Round buildings behaved particularly well during the 2001 earthquake of Gujarat, especially those that were built in adobe bricks. When it is not possible to have regular shapes, it is possible to improve the earthquake resistance by dividing the building in several parts. (Refer to Appendix 1.1)

Separation gap

Buildings with irregular and asymmetrical shapes are more fragile than simple ones. Hence they should be split into simpler shapes like shown above. These various parts will vibrate at a different frequency and amplitude under the reversible ground shakings. Therefore the y will hit each other and will be mutually damaged. A gap should be kept between them to a void collision. This gap can be filled with a crumbly material, which will be crushed under the shocks, or it can be left empty. In both cases, care should be taken for the waterproofing of the joint with a system that does not link again both parts. The separation gap must be minimum 25 mm for ground floor buildings and for higher ones the gap should be increased by 10 mm per storey more. (Refer to Appendix 1.3)

Rigidity distribution

The centre of gravity of the plan should also preferably be the centre of rigidity of the vertical masses. This would avoid torsion of the building.

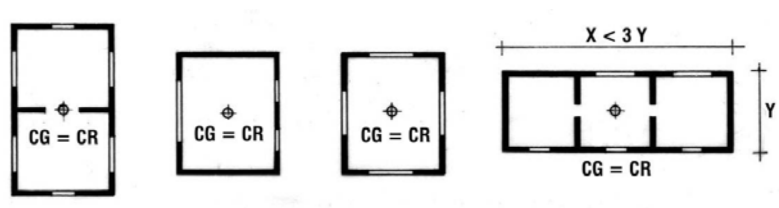
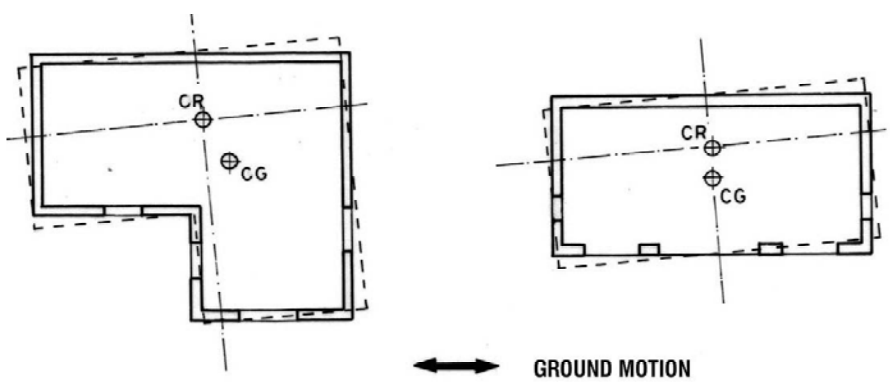


Fig 1.5 Proper distribution of Walls and openings



Wrong shape **Bad distribution of walls /openings**

Fig 1.6 Torsion due to Bad Design

Simplicity

Simplicity in the ornamentation is the best approach. Large cornices, vertical or horizontal cantilevered projections, cladding materials, etc. are dangerous during earthquakes. Thus they should be avoided. (Refer to Appendix 1.3)

Foundation

Certain types of foundation s are more susceptible to damage than others. Isolated footing of columns can easily be subjected to differential settlement, particularly when they rest on soft soils. Mixed foundations in the same building are also not suitable. In most of cases what works best are trench foundations. (Refer to Appendix 1.4)

Openings

Doors and windows reduce the lateral resistance of walls to shear. Hence, they should preferably be small and rather centrally located.

Height-to-thickness ratio

Masonry walls are slender because of their small thickness compared to their height and length. A simple way of making these walls behave well during earthquake shaking is by making them act together as a box along with the roof at the top and with the foundation at the bottom. A number of construction aspects are required to ensure this box action. Firstly, connections between the walls should be good. This can be achieved by (a) ensuring good interlocking of the masonry courses at the junctions, and (b) employing horizontal bands at various levels, particularly at the lintel level. Secondly, the sizes of door and window openings need to be kept small. The smaller the opening, the larger is the resistance offered by the wall. Thirdly, the tendency of a wall to topple when pushed in the weak direction can be reduced by limiting its length-to-thickness and height to-thickness ratios (Figure 1.4). Design codes specify limits for these ratios. A wall that is too tall or too long in comparison to its thickness is particularly vulnerable to shaking in its weak direction. Slender unreinforced masonry bearing walls with large height-to-thickness ratios (*h/t*) are more susceptible to damage from out-of-plane forces. Height `*h*` is unsupported height of unreinforced masonry wall, which is usually the storey height, unless bands (at lintel/sill level) are present. The band beams (sill/lintel) are assumed to provide necessary lateral support for the unreinforced masonry wall in out-of plane direction. The beams must be anchored into the return walls. (Refer to Appendix 1.5)

Seismic Response of Masonry building

Masonry buildings have dynamic properties (mass, stiffness, and strength) that affect how hard they shake in response to earthquake ground motion. Just like a tuning fork, each building has a natural tendency to vibrate at its fundamental frequency. If one of the frequency components of the ground motion is near the fundamental frequency of the building, accelerations (and forces) are amplified as the building is forced to resonate. As the ground motion changes direction, the forces within a building also change direction, causing shaking or vibrations in the building. A well-designed and well-built reinforced building has a reliable load path (see Figure 1.5) that transfers these forces through the structure to the foundation where the soil can resist them. Because the floor and roof elements (diaphragms) are relatively heavy, a large portion of the building mass is concentrated in these elements. For structural analysis purposes, the mass of other building components, including the walls, beams, columns, furniture, and other building contents, are normally presumed concentrated at the floor and roof levels. Horizontal earthquake forces are usually resisted by either walls or frame elements. At the base of wall and frame elements, foundation components transfer the earthquake forces to the earth. The diaphragms, walls, frames, and foundations of a building are the key elements along which engineers visualize a load path through the structure. The key links between these elements are also important components of the chain that makes up the horizontal and vertical load paths for the horizontal loads. The earthquake resistance of a building is only as strong as the weakest link in the load path.

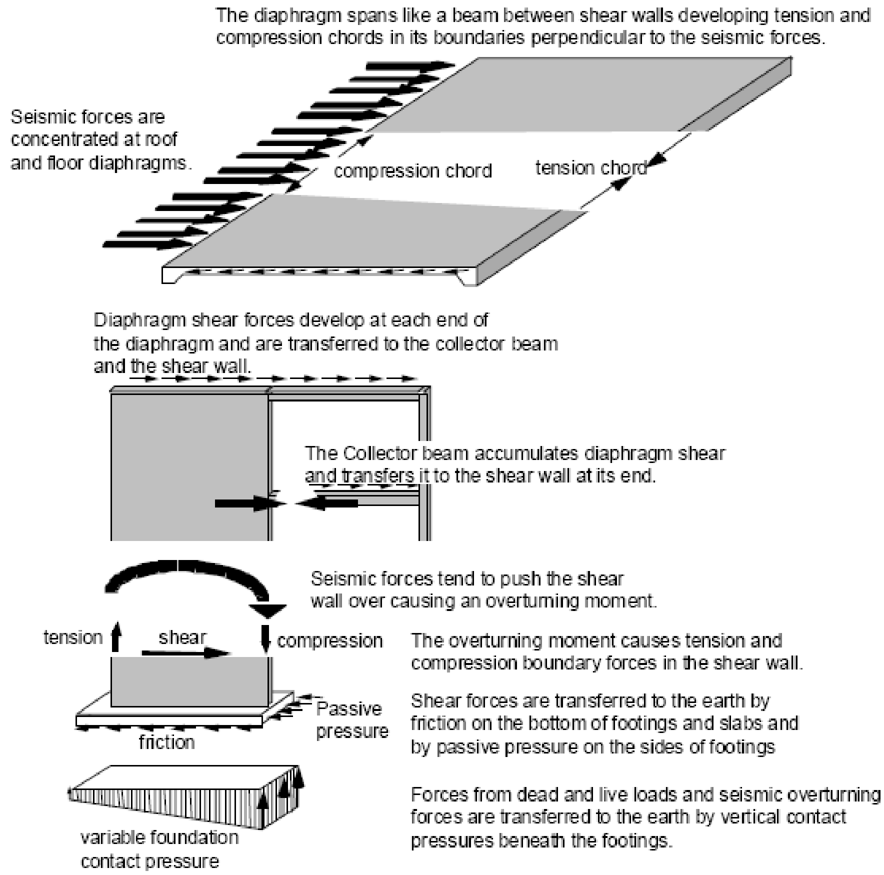


Fig 1.7 The load path for seismic forces in masonry buildings

Shear Wall Seismic Behaviour

Reinforced concrete and masonry shear walls are vertical seismic elements that resist lateral loads in their plane. They are like vertical diving boards extending upward from the foundation .The earthquake forces act horizontally in the plane of this vertical cantilever. After the diaphragm shear force has been transmitted into the shear wall, the shear wall behaves like an almost rigid diaphragm to resist these forces. In reinforced walls, the reinforcing bars (rebar) are usually laid out in a regular rectangular pattern, with bars running in both horizontal and vertical directions at uniform spacing. Shear walls develop bending forces as well as shears, and all forces are transmitted to the foundation elements, which resist the tendency of the seismic forces to push the wall over in its own plane (Figure 1.7). This moment, which wants to rotate the shear wall, is called an overturning moment. It increases from the top to the bottom of the building. This is why reinforced shear walls have extra vertical bars placed at the ends. This boundary reinforcing resists the bending forces, alternating vertical tension and compression, in the wall. Bending forces can also develop around large openings in walls. This is why additional trim bars are added at the edges of wall openings. Horizontal construction joints in walls rely on shear transfer mechanisms such as built-in bumps or blocks, like the vertical joints in rigid floor diaphragms. Seismic forces tend to push the shear wall over causing an overturning moment .The overturning moment causes tension and compression boundary forces in the shear wall. Sometimes walls in the same plane are connected together with horizontal beams, called spandrels or coupling beams, at floor and roof levels. During earthquakes, these components can also sustain damage similar to that observed in walls. New construction standards require reinforcing patterns that favor the more desirable, ductile behavior. In new construction, all shear walls are required by code to be reinforced.

REINFORCEMENT DETAILING OF MASONRY BUIDINGS

The essential reinforced elements required to make a building earthquake safe are as given in Figure 1.8**.** They are provided in the form of

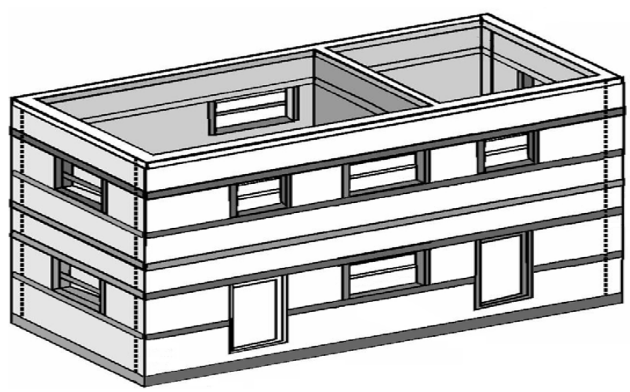


Fig 1.8 Reinforcement elements of masonry building

1. Lintel Band
2. Roof/ Floor Band
3. Vertical reinforcing bar at corner
4. Plinth Band
5. Window Sill Bands

HORIZONTAL SEISMIC BANDS

Horizontal bands are the most important earthquake-resistant feature in masonry buildings. The bands are provided to hold a masonry building as a single unit by tying all the walls together, and are similar to a closed belt provided around cardboard boxes. There are four types of bands in a typical masonry building, namely *gable band*, *roof band*, *lintel band* and *plinth band* named after their location in the building. The lintel band is the most important of all, and needs to be provided in almost all buildings. The gable band is employed only in buildings with pitched or sloped roofs. In buildings with flat *reinforced concrete* or *reinforced brick* roofs, the roof band is not required, because the roof slab also plays the role of a band. However, in buildings with flat timber or CGI sheet roof, roof band needs to be provided. In buildings with pitched or sloped roof, the roof band is very important. Plinth bands are primarily used when there is concern about uneven settlement of foundation soil. The lintel band ties the walls together and creates a support for walls loaded along weak direction from walls loaded in strong direction. This band also reduces the unsupported height of the walls and thereby improves their stability in the weak direction. During the 1993 Latur earthquake (Central India), the intensity of shaking in Killari village was IX on MSK scale. Most masonry houses sustained partial or complete collapse (Figure 2a). On the other hand, there was one masonry building in the village, which had a lintel band and it sustained the shaking very well with hardly any damage (Figure 1.9)

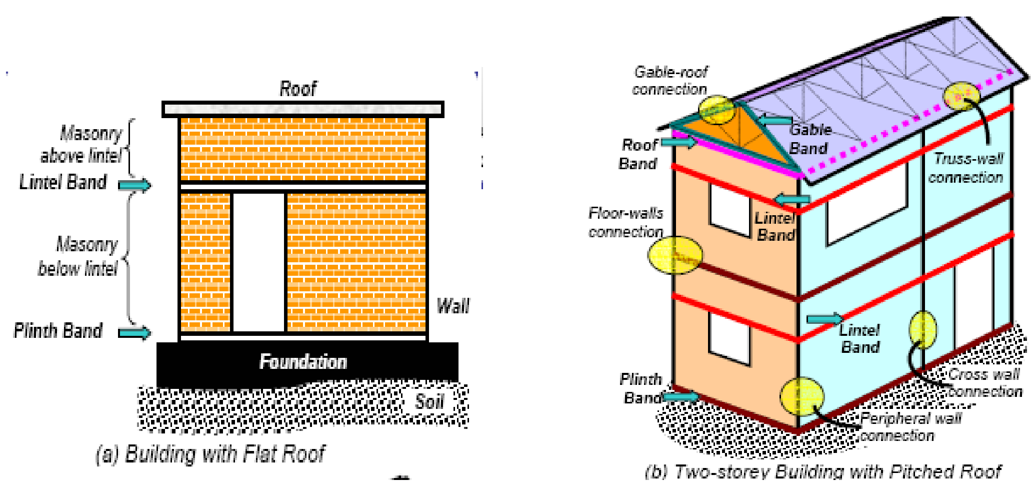


Fig 1.9 Horizontal bands for improving seismic resistance

(a) Building with no horizontal lintel band: collapse of roof and walls

(b) A building with horizontal lintel band in Killari village: no damage

Fig 1.10 The 1993 Latur Earthquake one masonry house in Killari village had horizontal lintel band and sustained the shaking without damage.

The dimensions of the band and the reinforcement inside depend upon the length of the walls between the perpendicular cross walls. The table below (Table-1.1) shows the dimensions to be adopted for the seismic bands and the internal reinforcement details to be provided. The reinforcement and bending details of seismic bands are given in the (Figure.1.11)**.** Reinforcing bars will be of Fe 415 type **[**TOR or, High Yield Strength Deformed, i.e. HYSD bars**]**

Table 1.1 dimension of seismic bands and internal reinforcement details

Internal length of wall	**Residential buildings**			**Important Public Buildings** (Schools, Hospitals, Meeting Halls, Anganwadis, etc.)		
	Size of the band	No. of Bars	Dia *(mm)*	Size of the band	No. of Bars	Dia *(mm)*
5 m or, less	10 cm x wall width	2	8	10 cm x wall width	2	10
6 m	10 cm x wall width	2	10	10 cm x wall width	2	12
7 m	15 cm x wall width	4	8	15 cm x wall width	4	10
8 m	15 cm x wall width	4	10	15 cm x wall width	4	12

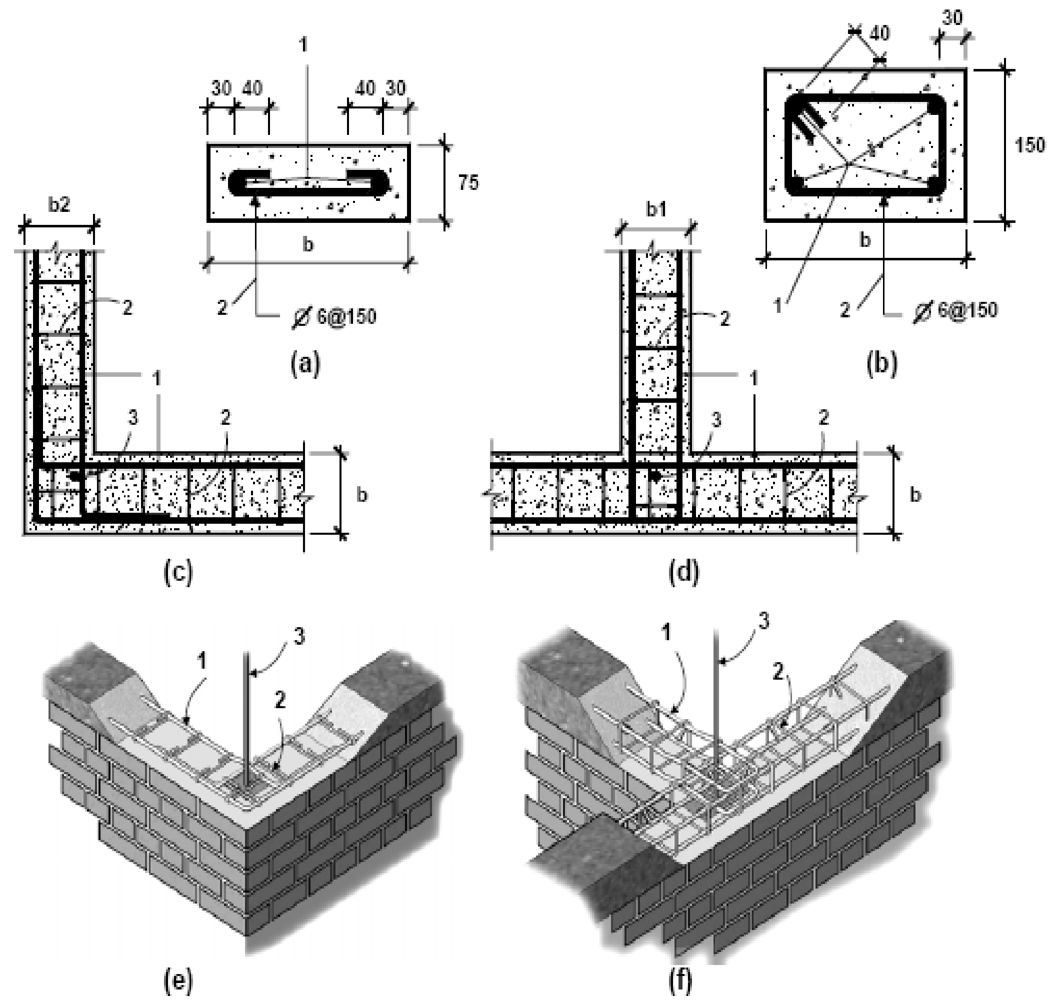
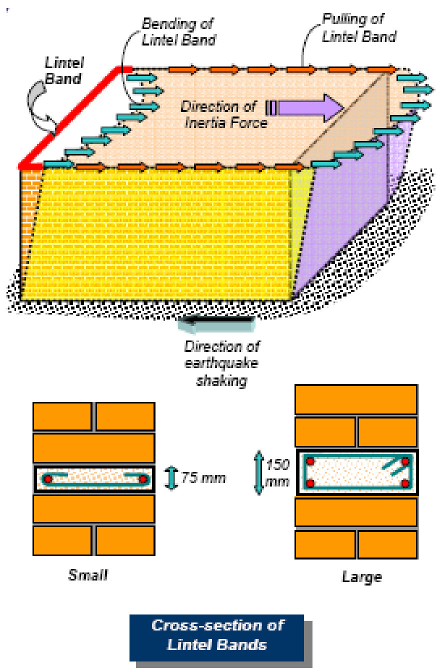


Fig 1.11 Reinforcement and bending details of seismic bands

Why are horizontal bands necessary in masonry buildings?

DESIGN OF LINTEL BANDS

During earthquake shaking, the lintel band undergoes bending and pulling actions (Figure1.12). To resist these actions, the construction of lintel band requires special attention. Bands can be made of wood (including bamboo splits) or of reinforced concrete (RC) (Figure 1.13); the RC bands are the best. The straight lengths of the band must be properly connected at the wall corners. This will allow the band to support walls loaded in their weak direction by walls loaded in their strong direction. Small lengths of wood spacers (in wooden bands) or steel links (in RC bands) are used to make the straight lengths of wood runners or steel bars act together. In wooden bands, proper nailing of straight lengths with spacers is important. Likewise, in RC bands, adequate anchoring of steel links with steel bars is necessary.



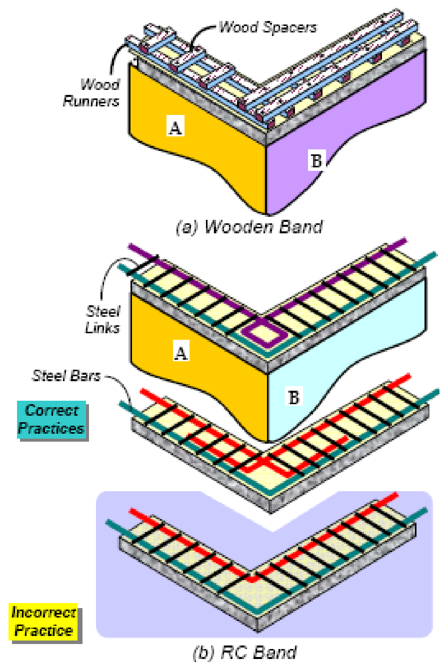


Fig 1.12 Cross-section of lintel bands

Fig 1.13 Horizontal bands

Why is vertical Reinforcement required in masonry Buildings?

Embedding vertical reinforcement bars in the edges of the wall piers and anchoring them in the foundation at the bottom and in the roof band at the top (Figure 1.12), forces the slender masonry piers to undergo *bending* instead of *rocking*. In wider wall piers, the vertical bars enhance their capability to resist horizontal earthquake forces and delay the X-cracking. Adequate cross-sectional area of these vertical bars prevents the bar from yielding in tension. Further, the vertical bars also help protect the wall from sliding as well as from collapsing in the weak direction.

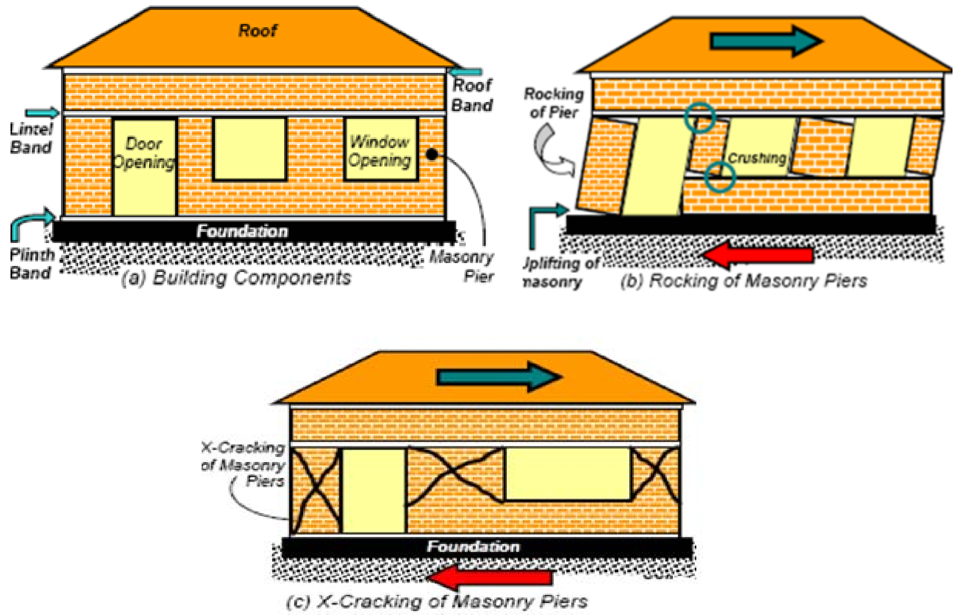
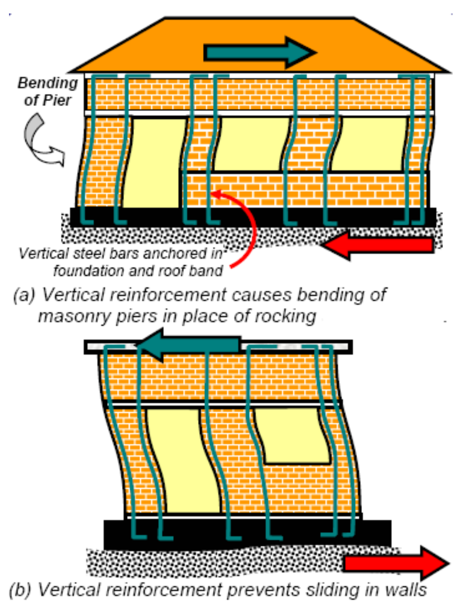


Fig 1.14 Earthquake response of a masonry building (No vertical reinforcement provided)



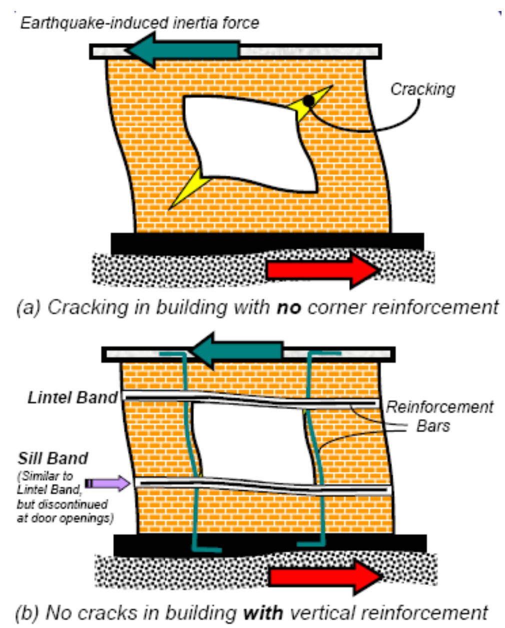


Fig 1.15 Vertical Reinforcement in masonry walls Wall behavior modified

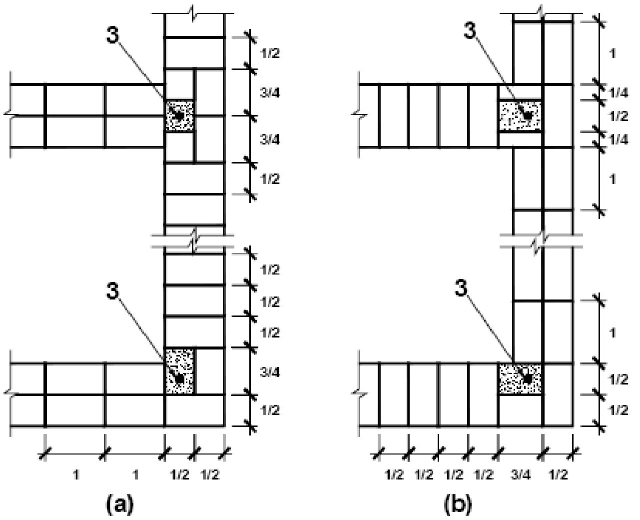
Fig 1.16 Cracks at corner of openings in a masonry building-reinforcement around them helps.

For earthquake safety reinforcing bars have to be embedded in brick masonry at the corners of all the rooms and the side of the door openings. Window openings larger than 60 cm in width will also need such reinforcing bars (Figure 1.16). The diameter of the bar depends upon then number of storey in the building. The recommendations are given in Table-2. Providing the vertical bars in the brickwork and concrete blocks requires special techniques which could be easily learnt by the supervising engineers and masons will need to be trained. These vertical bars have to be started from the foundation concrete, will pass through all seismic bands where they will be tied to the band reinforcements using binding wire and embedded to the ceiling band/roof slab as the case may be using a 300 mm 90° bend. Sometimes the vertical bars will not be made in one full length. In that case the extension of the vertical reinforcement bars are required, an overlap of minimum of 50 times the bar diameter should be provided. The two overlapped reinforcement bars should be tied together by using the binding wires.

Vertical Reinforcement in Masonry Structures

Table 1.2 Recommended size of vertical Steel in Seismic Bands

No. of storeys	Floor	**Residential buildings** *	**Important Public Buildings** * (Schools, Hospitals, Meeting Halls, Anganwadis, etc.)
		Dia of Single HYSD (TOR) Bar at corners of room (mm)	Dia of Single HYSD(TOR) Bar at corners of room (mm)
One	-	10	12
Two	Top	10	12
	Bottom	12	16
Three	Top	10	12
	Middle	12	16
	Bottom	12	16



a & b : Alternate courses in one brick wall
1 : One brick length
½ : Half brick length
¼ : Quarter of a brick length
¾ : Three quarters of a brick length
3 : Vertical reinforcement bars with Concrete/ mortar filling in pocket of M20 grade (1:1½:3 nominal mix)

Fig 1.17 Typical Details of Providing Vertical Steel Bars in Brick Masonry

Vertical Reinforcement at jambs at openings

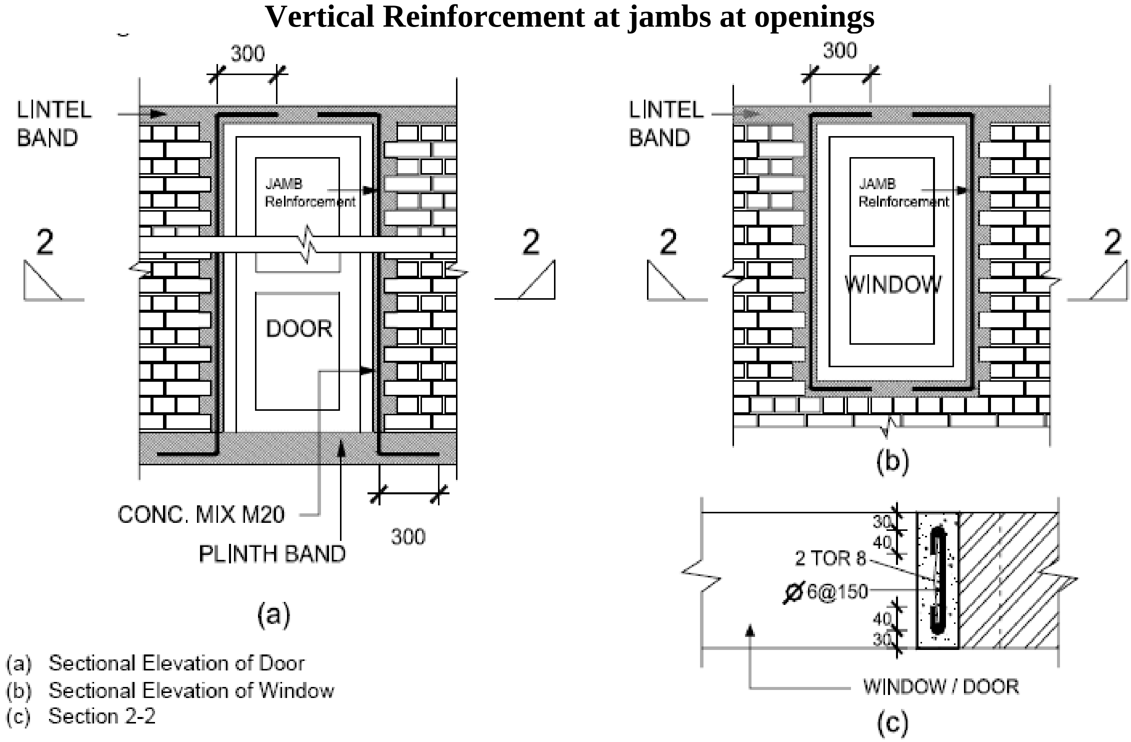


Fig 1.18.Typical details for providing vertical steel bars at openings

Foundation

Certain types of foundation s are more susceptible to damage than others. Isolated footing of columns can easily be subjected to differential settlement, particularly when they rest on soft soils. Mixed foundations in the same building are also not suitable. In most of cases what works best are trench foundations. Foundation width 'B' should be decided by the load coming on the foundation and the bearing capacity. Masonry width may be reduced by ½ times T in every step of 150 mm height.

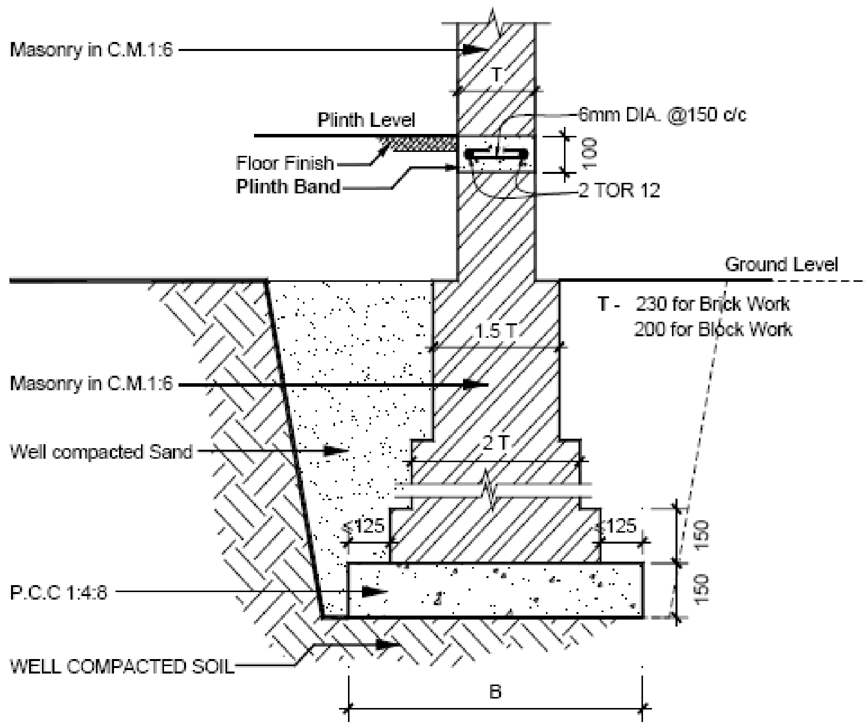


Fig 1.19 Foundation Details

CHAPTER 2

BASIC STEPS OF ANALYSIS

AND

DESIGN OF MASONRY BUILDING

PROCEDURE FOR ANALYSIS OF THREE STOREYED MASONRY BUILDING

2.1 STEP 1: DETERMINATION OF DESIGN EARTHQUAKE FORCES

“Equivalent Static seismic forces Procedure “being the simplest method of analysis was adopted to determine the seismic forces. Since the forces depend upon code based fundamental period of structures with some empirical modifier it required less computational effort.

a) The design base shear was computed as a whole, than distributed along the height of the buildings based on simple formulas appropriate for buildings with regular distribution of mass and stiffness.

b) The design Lateral force obtained at each floor level was distributed to individual Lateral Load resisting elements depending upon floor diaphragm action.

c) In case of rigid diaphragm (reinforced concrete monolithic slab beam floors or those consisting of prefabricated/precast elements with topping reinforced screed was taken as rigid diaphragm) action, the total shear in any horizontal plane was distributed to the various elements of Lateral force resisting, system on the basis of relative rigidity.

Following are the major steps of Equivalent static Analysis:

2.1.1 SIESMIC WEIGHT CALCULATIONS:

The seismic weight of each floor was taken as its full Dead Load plus appropriate amount of Imposed Load. While computing the seismic weight of each floor, the weight of columns and walls in any storey was equally distributed to the floors above & below the storey. The weight of Live Load for seismic calculation was taken as zero.

Dead Load and Live load at roof level

The Dead Load and the Live Load at roof level W_r consisted of the sum of (i) Weight of roof, (ii) Weight of walls and (iii) Weight of live load (LL).

i) Weight of roof was calculated as the product of length, breadth and weight of the roof slab.

ii) Weight of walls is calculated assuming half weight of walls at second storey is lumped at roof.

iii) Weight of live load (LL) for seismic calculation is taken as zero

DD and LL Load at each storey floor level:

The Dead Load and the Live Load at second storey roof level (W_{fi}) where i is the ith storey consisted of the sum of (i) Weight of floor, (ii) Weight of walls and (iii) Weight of Live Load (LL).

i) Weight of floor was calculated as the product of length, breadth and weight of the floor slab.

ii) Weight of walls was calculated assuming half weight of walls at i^{th} storey and half weight of walls at previous storey above which is lumped at roof.

iii) Live load is taken according to [IS 875 Part I][1]

DD and LL Load at first storey floor level:

The Dead Load and the Live Load at first storey roof level (W_{f2}) consisted of the sum of (i) Weight of floor, (ii) Weight of walls and (iii) Weight of Live Load (LL).

i) Weight of floor was calculated as the product of length, breadth and weight of the floor slab.

ii) Weight of walls was calculated assuming half weight of walls at each storey and half weight of walls at previous storey is lumped at roof.

Total seismic weight of building = $W_r + \sum W_{fi} + W_{f2}$

2.1.2 TIME PERIOD CALCULATIONS

The approximate fundamental natural period of a masonry building can be calculated from the clause 7.6.2 of [IS 1893(Part 1):2002][2] as,

$T_a = 0.09\ h/\sqrt{d}$

Where,

h= height of building in m, {i.e, (first storey) + (second storey)+(third storey) }

d= base dimension of building at the plinth level, in m, along the considered direction of lateral force (i.e, assuming earthquake in E-W direction)

$A_h = (ZI\ S_a)/\ (2Rg)$

The total design lateral base shear (V_B) along the direction of motion is given by

$V_B = A_h W$

WHERE DOES THE EARTHQUAKE FORCE ACT?

Earthquakes force is an inertia force. It acts on each mass particle of the structure and acts throughout the structure and is proportional to the mass and acceleration.

Again, in case of buildings, the floors are generally rigid in their plane and it can be assumed that all the points on the floor of a symmetric building move together with same displacement and acceleration. On the other hand, the acceleration increases along the height of the building and different floors have different acceleration.

Therefore for the sake of convenience, we assumed

The mass is lumped in certain points. (At the centre of its floors).

i) The earthquake forces are acting at these masses.

ii) The mass of the half of the storey above and half of the storey below is lumped at floor level.

iii) The force Q_i acting at a floor level is proportional to the lumped mass and the acceleration.

iv) The earthquake force is increasing along the height of the building, as the acceleration at floor level is increasing.

The total earthquake force on the building is expressed in terms of base shear,

V_B = which is equal to the sum of all floor loads $Q_{i.}$

$$V_B = {}^{n}\Sigma_{i=1} Q_i.$$

Where n is the number of storey.

2.1.3 VERTICAL DISTRIBUTION OF BASE SHEAR TO DIFFERENT FLOOR LEVELS

The design Lateral base shear (V_B) computed shall be distributed along the height of building as per the following expression:

$Q_i = V_B\, W_i h_i^2 / ({}^{n}\Sigma_{i=1} W_i h_i^2)$

Q_i= design lateral force at floor i,

W_i=seismic weight of floor i ,

h_i= height of floor i measured from base

n= number of storeys in the building is the number of levels at which mass are located.

Thus using the above formula the following was calculated

(i) Lateral Force at roof level

(ii) Lateral force at each storey roof level

(iii) Lateral force at 1st storey roof level.

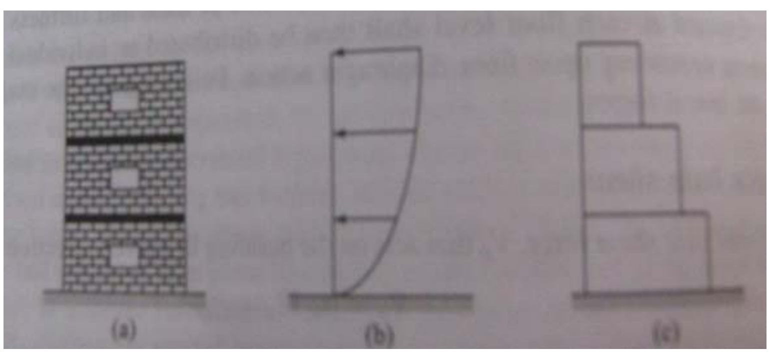


Fig. 2.1 (a) Seismic shear on building (b) Seismic loads(c) Storey Shear

Distribution of Lateral forces

In order to transfer the seismic forces to the ground, there should be a continuous load path in the building.

The general load path is as follows:

i) Earthquake forces, which originate in all the elements of the building, are delivered through the transverse wall of the building and it is bent between the floors.

ii) The lateral loads are transmitted from these transverse walls to the side shear wall by horizontal floor and roof diaphragms.

iii) The diaphragms distribute these forces to vertical resisting components if any which transfer the forces into the foundation, the diaphragms must have adequate stiffness and strength to transmit these forces.

iv) The distribution of lateral forces in the masonry buildings will depend upon the flexibility of horizontal diaphragm.

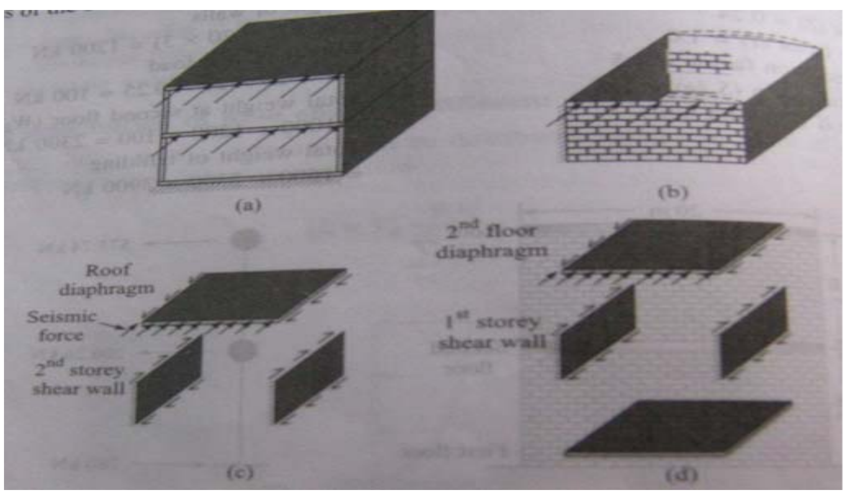


Fig 2.2. Lateral force distribution in a box type building (a) Box type masonry building subjected to lateral load (b) Bend of first storey/second storey transverse walls (c) distribution of lateral forces in second storey (d) Distribution of lateral forces in first storey.

The rigidity of the diaphragms is classified into two groups on relative flexibility:
Rigid and flexible diaphragm.

RIGID DIAPHRAGM

i) A diaphragm may be considered rigid when its midpoint displacement under lateral load is less than twice the average displacements at its ends.

ii) Rigid diaphragm distributes the horizontal forces to the vertical resisting elements in direct proportion to the relative rigidities.

ii) It is based on the assumption that the diaphragm does not deform itself and will cause each vertical element to deflect the same amount.

iv) Rigid diaphragms capable of transferring torsional and shear deflection forces are also based on the assumption that the diaphragm and shear walls undergo rigid body rotation and this produces additional shear forces in the shear wall.

v) Rigid diaphragms consist of reinforced concrete diaphragms, precast concrete diaphragms and composite steel check.

FLEXIBLE DIAPHRAGM

i) A Diaphragm is considered flexible, when the midpoint displacement, under lateral load, exceeds twice the average displacements of the supports.

ii) It is assumed that the relative stiffness of these non-yielding end supports is very great compared to that of the diaphragm.

iii) Diaphragms are often designed as simple beams between end supports and distribution of the lateral forces to the vertical resisting elements on a tributary width, rather than relative stiffness.

iv) Flexible diaphragm is not considered to be able capable of distributing torsional rotational forces. Flexible diaphragms consist of diagonally sheathed wood diaphragms, etc.

2.2 STEP 2: DETERMINATION OF WALL RIGIDITY

2.2.1 GENERAL GUIDELINES

i) The lateral load capacity of shear wall is mainly dependent on the in plane resistance rather than out of plane stiffness.

ii) The distribution of Lateral Load to the shear walls is based on the relative wall rigidities if a rigid diaphragm supports the walls and the segment of the wall deflects equally.

iii) The rigidity of shear wall is dependent on its dimensions, modulus of Elasticity (Em) , modulus of Rigidity (Gm) and the support conditions.

iv) The relative rigidity of shear wall elements is inversely proportional to their deflections when loaded with a unit horizontal force.

2.2.2 FACTORS AFFECTING RIGIDITY

1) Control joints are complete structural separations that break shear wall into elements like lintel bend. The elements must be considered as isolated members during shear wall rigidity analysis. The number and location of control joints within the total length of a wall may significantly affect elements rigidities especially flexure deformation.

2) Openings for doors, windows etc reduce the rigidity of shear wall elements. If openings are very small, their effect on the overall state of stress in a shear wall will become minor. Large openings will have pronounced effect when the openings in a shear wall become so large that the resulting wall approaches an assembly similar to a rigid frame or a series of elements linked by connecting beams, the walls will be analyzed accordingly.

   Pier Analysis

   In masonry structures, it is generally assumed that in one and two storey buildings the walls may be considered cantilevered and the segment of the walls between adjacent openings are called piers and might be considered fixed at top and bottom, depending upon the relative rigidness of the walls versus those of the floor diaphragms.

   The main assumptions in the analysis are (Schneider and Dickey, 1994):

   a) Rotational deformations of the portions above and below the openings are much smaller than those of the piers between the openings and are neglected.

b) Points of contra flexure are assumed at the midpoints of the piers and shears are assumed to be carried among the piers such that their top deflects by equal amount.

c) Lateral forces will be transformed to the various parallel resisting elements in direct proportion to their stiffness.

i) Stiffness refers to the lateral force magnitude required to produce a unit deflection

ii) Relative, rather than absolute stiffness can be computed since each wall is only being compared to the combined stiffness of the entire wall system.

Cantilever pier or wall

If the pier or wall is fixed only at the bottom and top is free to translate and rotate, it is considered a cantilevered wall.

When a force (P) is applied at the top of a pier , it will produce a deflection, , Δ_c that is the sum of the deflections due to bending moment (Δ'_m) Plus that due to the shear (Δ_v)

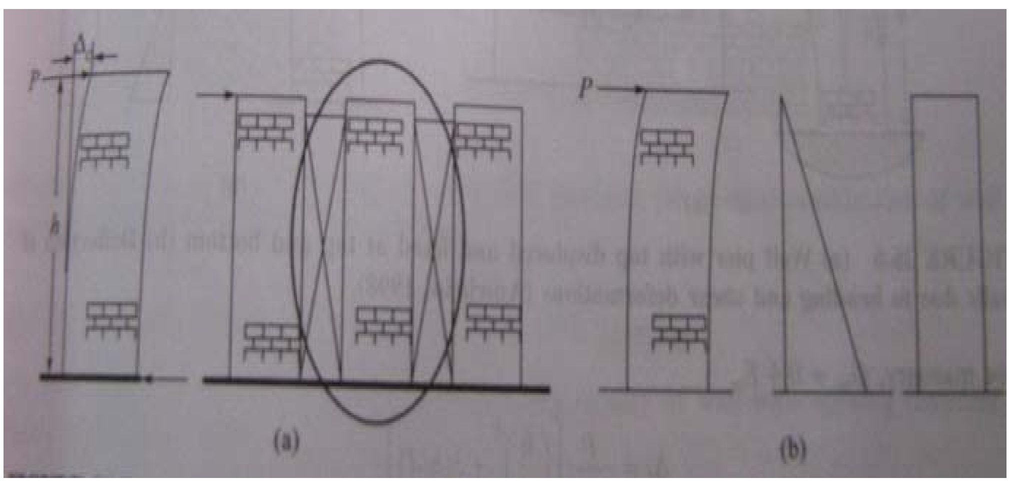


Fig2.3 Cantilever Pier or wall behavior to deflection

$\Delta_c = \Delta_m + \Delta_v$

Δ_m = deflection due to flexural bending

Δ_v = Deflection due to shear

P = lateral force on pier

h = height of pier

A = cross-section of pier

E_m = modulus of elasticity in compression

Δ_c =P/ E_mt [4(h/d)^3+3(h/d)

Rigidity of cantilever Pier $Rc = 1/ \Delta_c = E_m t/[(4(h/d)^3+3(h/d)]$

Fixed Pier or Wall

Fixed pier or wall fixed at top and bottom, the deflection from a force, P

$\Delta_f = \Delta_m + \Delta_v$

$= Ph^3/12E_m I + 1.2\ Ph/AG_m$

Δ_f =P/ E_mt [(h/d)^3+3(h/d)]

Rigidity of fixed pier

$R_f = 1/ \Delta_f = E_m t/ [(h/d)^3+3(h/d)]$

i. Very squat shear wall *(h/d<0.25),* rigidities based on shear deformation are reasonably accurate

ii. For deformation height of shear wall *(0.25<h/d<4.0),* including both the components of deflection

iii. For high h/d ratio, the effect of shear deformation is very small and rigidity based on flexural stiffness is reasonably accurate (Drydale, Hamid and Baker, 1994).

Horizontal and Vertical Combinations

If the shear wall segments are combined horizontally, the combined rigidity

$R = R_{c1} + R_{c2} + R_{c3}$, if the segments are combines vertically $1/R_{c1} + 1/R_{c2} + 1/R_{c3}$

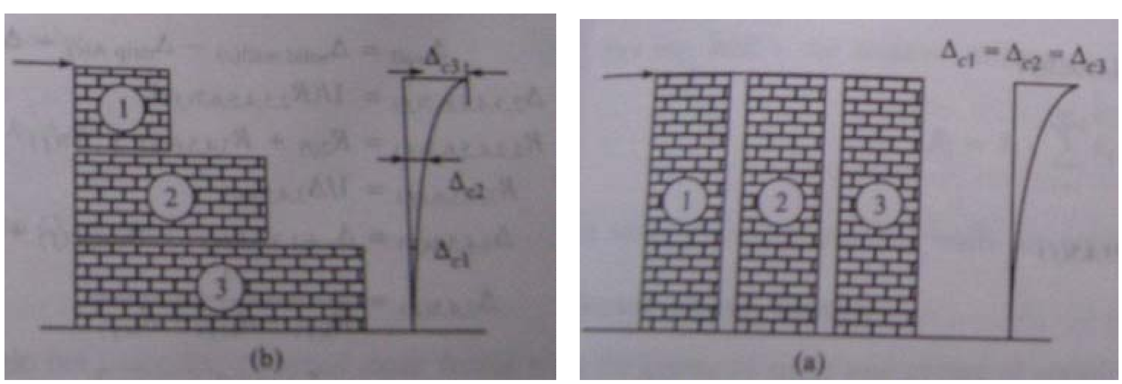


Fig 2.4 a) Horizontal combination and b) Vertical combination of Rigidity

Method for calculating the rigidity of the wall with opening

The following steps are required for calculating the rigidity of wall with opening.

- Assume full wall as solid calculate the deflection of the solid wall as a cantilever, $\mathit{\Delta}_{soli\mathbf{d(c)}}$
- Take a StripA of height equal to the height of largest opening. Calculate deflection of this strip. $(\mathit{\Delta}_{strip\ of\ highest\ opening(c)}$.
- Divide the wall in StripA into No. of segments .Assuming each solid strip as solid calculate the deflections of all the piers ($\Delta_{piers(f)}$)
- Calculate total defection of the wall with opening

(Δ_{total})= $\mathit{\Delta}_{soli\mathbf{d(c)}}$- $\mathit{\Delta}_{strip}$+ $\Delta_{piers(f)}$

The reciprocal of this value becomes relative rigidity of the wall. [R = 1/(Δ_{total})]

3) A shear wall element which is structurally internal at its end with a shear wall that is normal to the element forming an ‘L’ or ‘T’ in plane shape is called a corner element. The rigidity of a corner element is greater than that of a straight element. The amount of increase in rigidity is taken into account empirically when rigidity analysis is done.

2.2.3 PROCEDURE FOLLOWED IN RIGIDITY CALCULATION

a) The rigidity of the piers and solid wall are calculated by taking solid wall as cantilever pier, the piers and strip of wall are assumed as fixed pier.

Rigidity of cantilever pier $R_C = E_t / [4(h/d)^3+3(h/d)]$

Rigidity of cantilever pier $R_f = Et / [(h/d)^3+3(h/d)]$

b) They are then combined according to their horizontal and vertical position.

Total Rigidity of the wall is calculated as

$\Delta_{wall} = \Delta_{solidwall} - \Delta_{strip)} + \Delta_{piers\ combined}$

c) The north wall and south wall rigidity are calculated and then the relative stiffness of each wall is calculated.

North shear wall $= R_{wall(north)} / (R_{wall(north)} + R_{wall(south)})$

South shear wall $= R_{wall(south)} / (R_{wall(north)} + R_{wall(south)}$

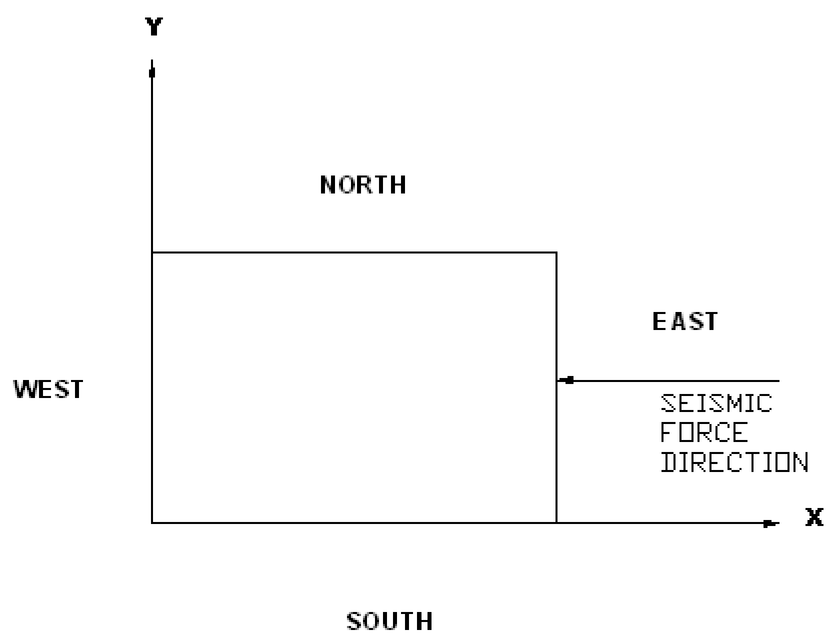


Fig.2.5 Building position and seismic force direction

2.3 STEP 3: DETERMINATION OF TORSIONAL FORCES

To calculate the shear forces due to torsion, first the location of the centre of mass and the centre of rigidity was calculated.

2.3.1 LOCATION OF THE CENTRE OF MASS

i) First the respective weights (W) of walls (N-wall, S-wall, E-wall and W-wall) and Roof Slab were found out in KN.

ii) The X-coordinate and Y-coordinates (in m) were found out.

Then for each item (Roof Slab, N-Wall, S-wall, E-wall, and W-wall) the statical moments about a point (WX and WY in KN-m) were calculated using the respective weights of walls as forces in the moment summation.

iii) The cumulative of ΣW, ΣWX and ΣWY were found out.

The centre of Mass (X_{CM} and Y_{CM}) were found out using the formulae

$$X_{CM} = \Sigma WX / \Sigma W$$

$$Y_{CM} = \Sigma WY / \Sigma W$$

Because of symmetrical layout of the building, the centre of mass occurs near the centre of building.

2.3.2 LOCATION OF THE CENTRE OF RIGIDITY

i) First the relative stiffness (R_x and R_y) of the walls (N-wall, S-wall, E-wall and W-wall) were found out.

ii) The X-coordinate and Y-coordinates (in m) were found out.

iii) Then for each item (N-Wall, S-wall, E-wall, and W-wall) the statical moments about a point (YR_x and XR_y) were calculated using the relative stiffness of the walls as forces in the moment summation. . The stiffness of the slab was not considered in the determination of centre of rigidity.

iv) The cumulative of ΣR_x, ΣR_y , $\Sigma X R_y$ and ΣYR_x were found out.

The centre of Rigidity (X_{CR} and Y_{CR}) were found out using the formulae

$$X_{CR} = \Sigma XR_y / \Sigma R_y$$

$$Y_{CR} = \Sigma YR_x / \Sigma R_x$$

Torsional Eccentricity

Torsional Eccentricity in y- direction:

Eccentricity between centre of mass and centre of rigidity

$e_y = Y_{CM} - Y_{CR}$

Adding minimum 5% accidental eccentricity

$=(0.05\times Y)$

Total eccentricity = $e_y + (0.05\times Y)$

Torsional eccentricity in x- direction

Eccentricity between centre of mass and centre of rigidity

$e_x = X_{CM} - X_{CR}$

Adding minimum 5% accidental eccentricity

$(0.05\times X)$

Total eccentricity = $e_x + (0.05\times X)$

2.3.3 TORSIONAL MOMENT

The torsional moment due to E-W seismic force rotate the building in y direction.

Hence $M_{TX} = V_x C_y$

Similarly if considered seismic force in N-S direction

$M_{Ty} = V_y C_x$

$V_y = V_x$ because S_a/g is constant value of 2.5 for Time period $0.11 \leq T \leq .55$

2.3.4 DISTRIBUTION OF FORCES IN NORTH AND SOUTH SHEAR WALLS

STEPS TO BE FOLLOWED

i) Since we are considering the seismic force only in E- W direction, the walls in N-S direction will resist the forces and the walls in E-W direction were ignored.

ii) Computation of Relative Stiffness (R_x),

iii) Calculation of distance of considered wall from centre of rigidity ($dy = 2Y - e_y$) in m, $R_x\, dy$ and $R_x\, dy^2$ for both N-wall and S-wall.

iv) Negative torsional shear was neglected.

v) Calculation of distribution of direct shear (KN)

Direct shear force from North-wall $= V_x R_N$

Direct shear force from South-wall $= V_x R_S$

Calculation of Torsional shear force (KN)

Torsional force in North-wall = $(\Sigma R_x\ dy)/(\Sigma R_x\ dy^2) \times (V_x\ e_y)$

Torsional force in South-wall = $(\Sigma R_x\ dy)/(\Sigma R_x\ dy^2) \times (V_x\ e_y)$

2.3.5 DISTRIBUTION OF FORCES IN EAST AND WEST SHEAR WALLS

STEPS TO BE FOLLOWED

i) Computation of Relative Stiffness (R_y),

ii) Calculation of distance of considered wall from centre of rigidity ($dx = 2X - e_x$) in m, $R_y\ dx$ and $R_y\ dx^2$ for both E-wall and W-wall.

iii) Calculation of distribution of direct shear force (KN)

Direct shear force from East-wall $= V_y R_E$

Direct shear force from West-wall $= V_y\ R_W$

Calculation of Torsional shear force:

Torsional force in East-wall = $(\Sigma R_y\ dx)/(\Sigma R_y\ dx^2) \times (V_y\ e_x)$

Torsional force in West-wall = $(\Sigma R_y\ dx)/(\Sigma R_y\ dx^2) \times (V_y\ e_x)$

2.3.6 DISTRIBUTION OF THE TOTAL SHEAR TO INDIVIDUAL PIERS WITHIN THE WALL

The shear carried by the North and South shear walls is now distributed to individual piers on the basis of their respective stiffness. Shear in pier group is further subdivided in vertical piers 1, 2, 3 on proportion to their stiffness.

2.4 STEP 4: DETERMINATION INCREASE IN AXIAL LOAD DUE TO OVERTURNING

In shear wall analysis the principal forces are

i) In plane shear (direct + torsional)

ii) In plane moment (in plane shear × ½ height of pier)

iii) Dead & Live Load carried by the pier.

iv) Lateral forces from Wind or Earthquakes which create severe overturning moments on buildings. If the overturning moment is great enough it may overcome the dead weight of the structure & may cause tension at the end of the piers of shear walls. It may also induce high compression forces in the pier of walls that may increase the axial load in addition to the dead load and live load.

The increase in axial load in piers due to overturning moments was evaluated as below:

Overturning moment at ith floor level

$(M_{ovt})_2 = V_r\ (h_i+h_{i+1}) + V_{i+1}\ h_i$

(where' i' is the floor level and V_i is the seismic force at ith floor level

Total overturning moment on pier in the first storey M_{ovt}

$=(M_{ovt})_2$+(total V) × (distance to the second floor level from critical level of the pier in the ith storey

Let at the sill height of the pier = h_{ir}

Thus the axial load on a pier due to overturning change to P_{ovt} is

$P_{ovt} = (M_{ovt})\ (l_iA_i)/I_n$

l_i = Distance from the centre of gravity of net wall section in the ith storey to the centroid of the pier

$= {}^{n}\Sigma_{i=1}\ (l_iA_i)/\Sigma A_i$

A_i= cross-sectional area of pier in question

I_n= Moment of Inertia of net wall section in first storey

$= {}^{n}\Sigma_{i=1}\ l_iA_i^2$

M_{ovt} =Total shear (V_x) × (vertical distance between second floor to critical plane of weakness, assuming at the level of sill) + (Applied overturning Moment at second floor level)

It is assumed that the stiffness of second storey walls is the same as first storey, the total direct shear in E-W direction of seismic load i.e, in X direction is divided in North and South shear wall is proportional to their stiffness

Direct shear in North Wall (V_{NX})

Direct shear in South Wall (V_{SX})

Distribution of Lateral force along the height of North and South wall is

2.4.1 DISTRIBUTION OF LATERAL FORCES

Lateral force at a height h_i $= V_{NX} \times W_i h_i^2 / ({}^{n}\Sigma_{i=1} W_i h_i^2)$

Using the above formula the following were calculated for both North and South shear wall

(i)Lateral force at each storey roof level

M_{ovt}

$= V_r (h_i + h_{i+1}) + V_{i+1} h_i +$ (total V) × (distance of i^{th} floor level from critical level of the pier in the (i-1) storey)

Increase in axial Load due to overturning moment

$P_{ovt} = M_{ovt} L_i A_i / I_n$

Where $I_i A_i$ = centroid of net section of wall.

I_n = Moment of inertia of net section of wall.

2.4.2 CALCULATION OF CENTROID OF NET SECTION OF WALL

Distance from left edge to centroid of net section of wall

$= \Sigma A_i I / \Sigma I$

2.4.3 CALCULATION OF MOMENT OF INERTIA OF NET SECTION OF WALL

$I_n = I + A_i li^2$

Where $I = td^3/12$ (in m^4)

2.4.4 INCREASE IN AXIAL LOAD IN PIERS OF NORTH WALL

Increase in axial Load due to overturning moment

$= P_{ovt} = M_{ovt} L_i A_i / I_n$

Where $l_i A_i$ = centroid of net section of wall

I_n = Moment of inertia of net section of wall

2.4.5 CALCULATION OF CENTROID AND MOMENT OF INTERTIA OF NET SECTION OF WALL

Centroid

Centroid of each pier is calculated and then for the net section of the wall

Distance from left edge to centroid = $\Sigma A_i l / \Sigma l$

Where l= distance from lift edge of wall to centroid of pier and A_i = Area

Moment of Inertia

Moment of inertia of each pier is calculated and then for the net section of the wall by formula below.

Where $I_n = I + A_i li^2$

$I = td^3/12$

I is the moment of inertia of each pier, where d is the width and t is the thickness of the pier

2.4.6 INCREASE IN AXIAL LOAD IN INDIVIDUAL PIERS OF SOUTH SHEAR WALL

Overturning moment in wall M_{ovt}

= Total shear at third floor X critical height (h_{cr}) + $M_{ovt(2)}$

Increase in axial Load due to overturning moment

= $P_{ovt} = M_{ovt} L_i A_i / I_n$

Where $l_i A_i$ = centroid of net section of wall

I_n = Moment of inertia of net section of wall

2.5 STEP 5: DETERMINATION OF PIER LOADS, MOMENTS AND SHEAR

2.5.1 PROCEDURE

i) Dead load intensity is calculated for each wall (North and south wall) per metre length of wall

Dead load intensity= weight of first storey from IInd floor to sill level+ weight of each storey+ weight of floor at each storey level (assuming North and South wall will take equal amount of load+ weight of roof)

ii) Live load intensity is also calculated as [weight of floor at each storey level (assuming north and south wall will take equal amount of load+ live load of roof]

iii) P_d= Effective Loading Width of Pier × Dead Load Intensity in KN/m

P_L= Effective Loading Width of Pier × live Load Intensity in KN/m

Effective Loading width of pier = Width of pier + ½ of each adjacent opening of pier

iv) Shear V_E is calculated from the distribution of total shear to individual piers

v) Moment = V_E × h/2 where h is the height of pier

2.6 STEP 6: DESIGN OF SHEAR WALLS FOR AXIAL LOAD AND MOMENTS

i) Moment (M) calculated in step 5 is taken into calculation.

ii) Effective depth of pier is calculated taking some cover $d_{effective}=d_{total}$-cover

iii) Area of jamb steel which are to provided at the pier boundary is calculated as follows

$$As=M/(f_s \times 0.9 \times d_{effective})$$

$$f_s=0..55\ Fe=0.55 \times 415=230\ N/mm^2$$

iv) For the calculated area of jamb steel the no.of steel bars are assumed and arrangement is done according to IS 4326:1993

v) Adequacy of individual piers under compression and moment is checked by interaction formula. $(fa/F_a) + (f_b/F_b) \leq 1.33$

Where $fa= P_{total}\ (P_d+ P_L+ P_{ovt}) / (\text{width of pier (d)} \times t)$

$f_b= M/(td^2/6)$

F_a= Permissible compressive stress=2.5 N/mm^2

F_b=Permissible bending stress=2.5+0.25×2.5=3.125(as per IS :1905)

2.7 STEP 7: DESIGN OF SHEAR WALL FOR SHEAR

In load bearing masonry buildings, the walls, which carry gravity loads also acts as shear walls to resist lateral load. The structural walls parallel to lateral load and subjected to in-plane (shear) forces and bending are called shear walls. Shear in building is resisted by providing the bands or bond beams. The bands represent a horizontal framing system, which transfer the horizontal shear induced by the earthquakes from the floors to shear (structural) walls. It also connects all the structural walls to improve the integral action. In combination with vertical reinforcement, it improves the strength, ductility and energy dissipation capacity of masonry walls.

2.7.1 DESIGN OF BOND BEAM

Total seismic Force in the direction of seismic force (E-W) direction=V

Moment produced (M) = V×L/8 where L is the length of Building Plan

T=M/d d is the breadth of plan

$As=T/f_s$ f_s=230N/mm^2

The Design of Plinth band, Lintel band has been done according to the

Table 6 and table 7 of [IS 4326:1993][3] and

(Refer to Appendix 1.6,1.7,1.7,1.9,1.10,1.11,1.12)

CHAPTER 3

CALCULATION INVOLVED IN ANALYSIS AND DESIGN OF A THREE STOREYED RESIDENTIAL BUILDING

3.1 INTRODUCTION

In this project we have illustrate the seismic analysis and design procedure for a low rise residential masonry building.

A three-storey masonry building situated in zone V has been analysed and designed. The seismic analysis has been carried out by considering earthquake only in one direction..

3.1(a) BUILDING DATA

The plan and elevation of building are shown in figure below

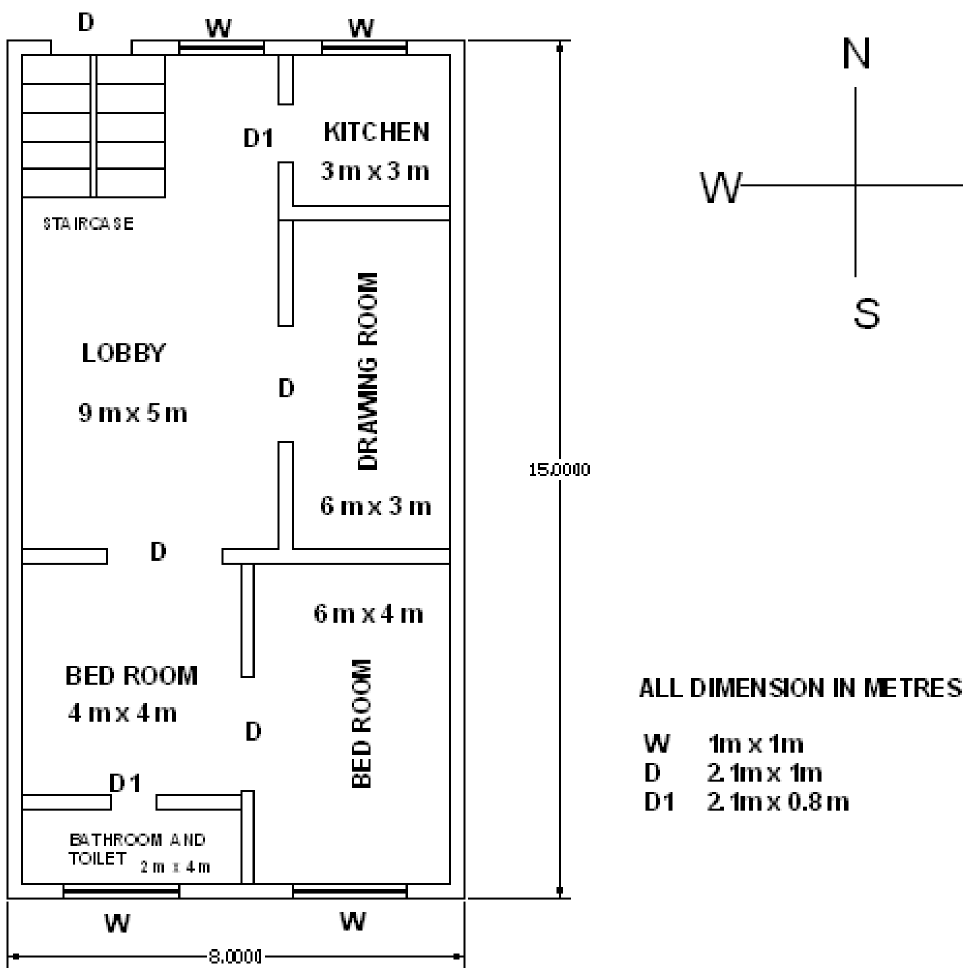


Fig 3.1 Plan of building

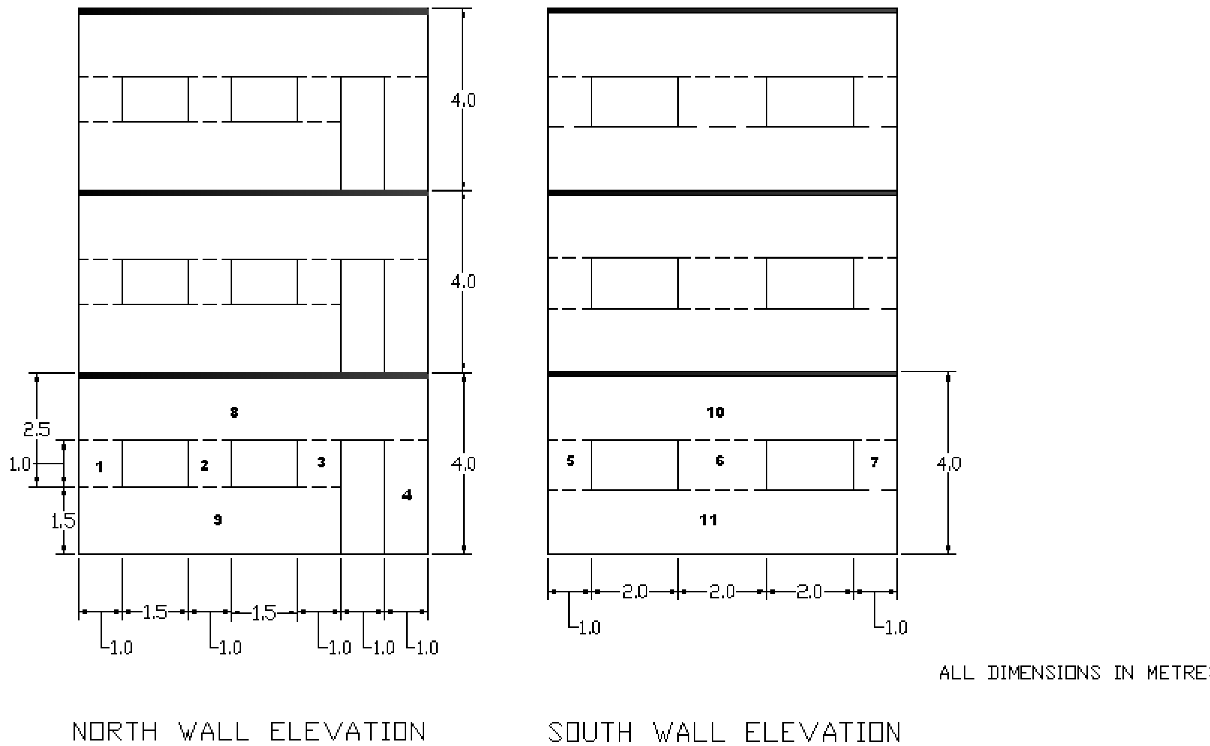


Fig3.2 North wall and south wall Elevation

3.1(b) MATERIAL STRENGTH

1. Permissible compressive strength (f_m) =2.5 N/mm^2
   (Assuming unit strength =35MPa and mortar H1 type)

   In code IS 4326:1993 specifies that well burnt bricks and solid concrete bricks possessing a compressive / crushing strength not less than 35 MPa shall be used. The compressive strength of unit may be defined as the maximum stress to which unit can be subjected by a gradually increasing load applied in perpendicular direction either to bedding plane or normal position. The mortar used I masonry constructions in seismic area depends upon the design seismic coefficient. For (0.08 <An (seismic coefficient) <0.12) and (0.12< An, 0.05) type building H1 type mortar (cement: sand 1:4) is used.

2. Permissible stress in steel in tension =0.55 f_y
   (Use high deformed bar Fe415 i.e, f_y =230 N/mm^2)

3.1(c) LOADS

LIVE LOAD DATA

Live Load on roof =1.0 KN/mm^2 (for seismic calculation =0)
Live Load on floor =1.0 KN/ mm^2

DEAD LOAD DATA

Thickness of floor and roof slab=120 mm

Weight of slab =3 KN/ mm^2

(Assuming weight density of concrete =25 KN/ mm^3)

Thickness of wall =250 mm

Weight of wall =5 KN/ m^2

(Assuming weight density of masonry = 20 KN/ m^3)

3.1(d) SEISMIC DATA: (as per IS 1893 (part 1):2002

(i) Seismic zone = zone V

(ii) Zone factor (Z) =0.36, Zone factor given is for maximum considered Earthquake (MCE) and service Life of structure in a zone. The factor 2 is used so as reduce the Maximum considered Earthquake (MCE) zone factor to the factor for design basis Earthquake (Table 2)

(iii) Importance factor (I) =1

(iv) Response Reduction Factor =3

The value of R for building is given in Table 7: IS 1893 (Part 1); 2002.

(v) Soil medium type, for which average response acceleration coefficient are as

$$S_a/g = \begin{cases} 1+15T, & 0.00 \le T \le 0.10 \\ 2.50, & 0.10 \le T \le 0.55 \\ 1.36/T, & 0.55 \le T \le 4.00 \end{cases}$$

(v) Direction of seismic force = E- W Direction.

3.2 STEP 1: DETERMINATION OF DESIGN LATERAL EARTHQUAKE FORCES

For determination of Lateral Load earthquake “Equivalent Static Lateral forces Procedure “is adopted.

(i) SIESMIC WEIGHT CALCULATIONS:

The seismic weight of each floor is its full dead Load plus appropriate amount of imposed Load. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be equally distributed to the floors above & below the storey. The weight of Live Load for seismic calculation is taken as zero.

Seismic weight calculations

	Description:	Load Calculations	Total
	DL and LL load at roof level		
(i)	Weight of roof	3×8×15	360 KN
(ii)	Weight of walls (Assuming half weight of walls at second storey is lumped at roof)	½ {2(8+15) ×4×5}	460 KN
(iii)	Weight of live load (LL) (for seismic Calculation, LL on Roof is zero)	0×8×15	0 KN
	(W_r) weight at roof level (i) + (ii) + (iii)	360+460+0	820 KN

DD and LL Load at second storey floor level

	Description:	**Load Calculations**	**Total**
(i)	Weight of floor	3×8×15	360 KN
(ii)	Weight of walls (Assuming half weight of walls at second storey and half weight of walls at first storey is lumped at roof)	2×1/{2(8+15)×4×15}	920 KN
(iii)	Weight of Live Load (LL)	1×8×15	120 KN
	(W_{f1}) Weight at second storey level(i) + (ii) + (iii)	360+920+120	1400KN

DD and LL Load at first storey floor level

	Description:	**Load Calculations**	**Total**
(i)	Weight of floor	3×8×15	360 KN
(ii)	Weight of walls (Assuming half weight of walls at second storey and half weight of walls at first storey is lumped at roof)	2×1/{2(8+15)×4×15}	920 KN
(iii)	Weight of Live Load (LL)	1×8×15	120 KN
	(W_{f2}) Weight at second storey level(i) + (ii) + (iii)	360+920+120	1400KN
	Total seismic weight of building ($W_r + W_{f1} + W_{f2}$)	820 +1400+1400	3620 KN

(ii) Time period calculations

The approximate fundamental natural period of a masonry building can be calculated from the clause 7.6.2 of IS 1893(Part 1):2002 as,

$T_a = 0.09\ h/\sqrt{d}$

Where,

h= height of building in m, {i.e, 4.0 (first storey) +4.0 (second storey)+4.0 (third storey) = 12.0 m}

d= base dimension of building at the plinth level, in m, along the considered direction of lateral force (i.e, 8 m assuming earthquake in E-W direction)

$T_a = 0.09\ x12/\sqrt{8} = 0.038$ sec

S_a/g =2.5, for T=0.038

$A_h = (ZI\ S_a)/\ (2Rg) = (0.36/2)\ (1/3)\ (2.5) =0.15$

The total design lateral base shear (V_B) along the direction of motion is given by

$V_B = A_hW=0.15$ x 3620=543 KN

iii) Vertical distribution of Base Shear to different floor levels:

The design Lateral base shear (V_B) computed shall be distributed along the height of building as per the following expression:

$Q_i= V_B\ W_ih_i^2/\ (^n\Sigma_{i=1}\ W_ih_i^2)$

Q_i= design lateral force at floor I,

W_i=seismic weight of floor I,

h_i= height of floor i measured from base

n= number of storeys in the building is the number of levels at which mass are located.

Lateral Force at roof level

$= V_B\ W_ih_i^2/\ (^n\Sigma_{i=1}\ W_ih_i^2)$

$= (543x820x12^{2)}/\ (820x12^2+1400x8^2 +1400x4^2)$

= 278.87 KN

Lateral force at 2nd storey roof roof level

$= V_B\ W_ih_i^2/\ (^n\Sigma_{i=1}\ W_ih_i^2)$

$= (543 \times 1400 x 8^{2)} / (820 \times 12^2 + 1400 \times 8^2 + 1400 \times 4^2)$

$= 211.46$ KN

Lateral force at 1st storey roof level

$= V_B\ W_i h_i^2 / ({}^n\Sigma_{i=1}\ W_i h_i^2)$

$= (543 \times 1400 \times 4^{2)} / (820 \times 12^2 + 1400 \times 8^2 + 1400 \times 4^2)$

$= 52.865$ KN

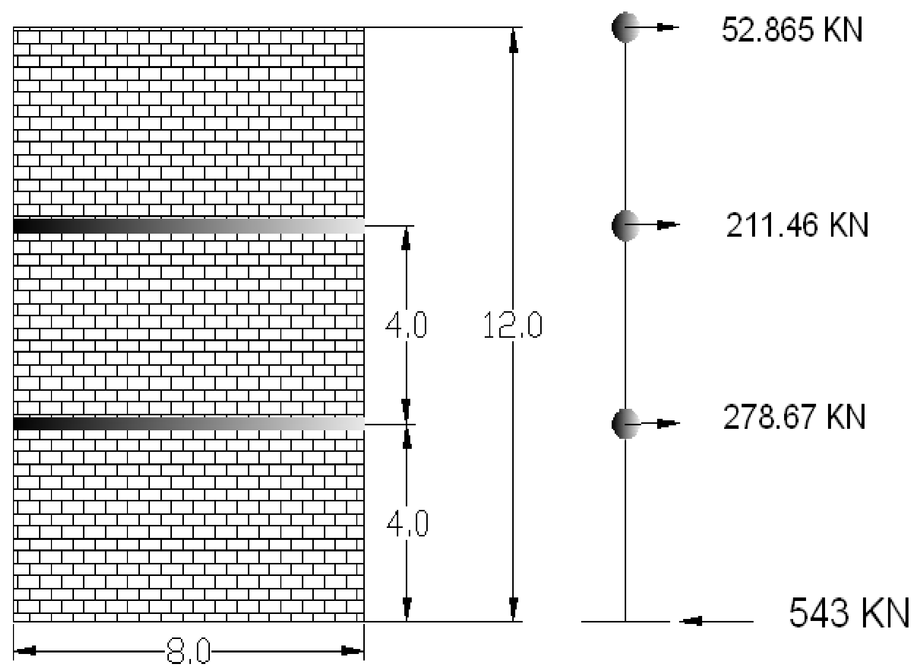


Fig 3.3 Elevation of building and seismic load or storey shear

3.3 STEP 2: DETERMINATION OF WALL RIGIDITY

Rigidity of North shear wall

$\Delta_{wall} = \Delta_{solidwall(c)} - \Delta_{stripA(c)} + \Delta_{1,2,3,9,4(f)}$

$\Delta_{1,2,3,9,4(f)} = 1/ R_{1,2,3,9,4(f)}$

$R_{1,2,3,9,4(f)} = R_{1,2,3,9(f)} + R_{4(f)}$

$R_{1,2,3,9(f)} = 1/ \Delta_{1,2,3,9(f)}$

$\Delta_{1,2,3,9(f)} = \Delta_{solid1,2,3,9(c)} - \Delta_{stripB(f)} + \Delta_{1,2,3(f)}$

$\Delta_{1,2,3(f)} = 1/(R_{1(f)} + R_{2(f)} + R_{3(f)})$

Rigidity of cantilever pier is given by

$R_c = E_t / [4(h/d)^3+3(h/d)]$

Rigidity of fixed pier is given by

$R_f = E_t / [(h/d)^3+3(h/d)]$

$R_{solid(c)} = E_t / [4(4/8)^3+3(4/8)]$

$= 0.5\ E_{t,}$

$\Delta_{solid(c)} = 2.0/E_t$

$R_{strip\ A(c)} = E_t / [4(2.5/8)^3+3(2.5/8)]$

$=0.944\ E_t$

$\Delta_{strip\ A(c)} = 1.06/E_t$

$R_{solid\ 1,2,3,9(f)} = E_t / [4(2.5/6)^3+3(2.5/6)]$

$=0.756\ E_t$

$\Delta_{solid1,2,3,9\ (f)} =1.322/E_t$

$R_{strip\ B(f)} = E_t / [(1/6)^3+3(1/6)]$

$=1.98\ E_t$

$\Delta_{strip\ B(f)} = 0.546/E_t$

$R_{1(f)} = R_{2(f)} = R_{3(f)} = E_t / [(1/1)^3+3(1/1)]$

$=0.25E_t$

$\Delta_{1,2,3(f)} = 1.33/E_t$

$\Delta_{1,2,3,9(f)} = 1.322/E_t - 0.5046/E_t + 1.33/E_t =2.15 /E_t$

$R_{1,2,3,9(f)} = E_t / 2.15 = 0.465\ E_t$

$R_{4(f)} = E_t / [(2.5/1)^3+3(2.5/1)]$

$=0.043\ E_t$

$\Delta_{1,2,39,4(f)} = 1.968/E_t$

$\Delta_{wall} = 2.0/E_t - 1.06/E_t + 1.96/E_t = 2.908/E_t$

$R_{wall} = 0.343E_t$

Rigidity of South shear wall

$\Delta_{wall} = \Delta_{solidwall(c)} - \Delta_{stripA2(c)} + \Delta_{5,6,7(f)}$

$\Delta_{5,6,7(f)} = 1/R_{5,6,7(f)}$

$R_{5,6,7(f)} = R_{5(f)} + R_{6(f)} + R_{7(f)}$

$R_{5(f)} = R_{7(f)} = E_t / [(1/1)^3+3(1/1)]$

$= 0.25\ E_{t,}$

$R_{6(f)} = E_t / [(1/2)^3+3(1/2)]$

$= 0.615\ E_{t,}$

$R_{5,6,7(f)} = 2x0.25\ E_t + 0.615\ E_t$

$= 1.115\ E_{t,}$

$\Delta_{5,6,7(f)} = 1/R_{5,6,7(f)}$

$= 0.896/E_{t,}$

$R_{solid(c)} = E_t / [4(4/8)^3+3(4/8)]$

$= 0.5\ E_{t,}\ \Delta_{solid(c)} = 2.0/E_t$

$R_{strip\ A(c)} = E_t / [4(1/8)^3+3(1/8)]$

$= 2.612\ E_t$

$\Delta_{solidA2\ (c)} = 0.382/E_t$

$\Delta_{wall} = \Delta_{solid\ wall\ (c)} - \Delta_{solidA2\ (c)} + \Delta_{5,6,7\ (f)}$

$= 2/E_t - 0.382/E_t + 0.896/E_t$

$= 2.513/E_t$

$R_{wall} = 0.398E_t$

Relative stiffness of walls

North shear wall =0.343/ (0.343+0.398) =0.462

South shear wall =0.398/ (0.343+0.398) =0.538

3.4 STEP 3: DETERMINATION OF TORSIONAL FORCES

Location of the centre of mass

Centre of mass, X_{CM} and $Y_{CM,}$ is calculated by taking the statical moments about a point, say, south-west corner, using the respective weights of walls as forces in the moment summation. Because of symmetrical layout of the building, the centre of mass will occur near the centre of building i.e, X_{CM} = 4 cm, and Y_{CM} = 7.5 cm. However for methodology purpose the calculations for the centre of mass is shown in Table

Table 3.1 Calculation of centre of mass

Item	Weight I (KN)	X (m)	Y (m)	WX(KN-m)	WY (KN-m)
Roof Slab	8x15x3=360	4.0	7.5	1440	2700
N-Wall	8x4x5=160	4.0	15	640	2400
S- Wall	8x4x5=160	4.0	0.0	640	0
E-Wall	15x4x5=300	8.0	7.5	2400	2250
W-Wall	15x4x5=300	0.0	7.5	0	2250
	ΣW=1280			ΣWX=5120	ΣWY=9600

$X_{CM} = \Sigma WX / \Sigma W$ = 4.0m from west wall

$Y_{CM} = \Sigma WY / \Sigma W$ = 7.5 m from east wall

Location of the centre of Rigidity

The centre of rigidity, X_{CR} and Y_{CR} is calculated by taking statical moments about a point say South-West corner, using the relative stiffnesses of the walls as forces in the moment summation. The stiffness of the slab is not considered in the determination of centre of rigidity. The calculation for the centre of rigidity is as shown in Table 3.2

Table 3.2 Calculation of centre of rigidity

ITEM	R_x	R_y	X (m)	Y (m)	Y R_x	X R_y
N-Wall	0.462	-	-	15	6.93	-
S-Wall	0.538	-	-	0.0	0	-
E-Wall	-	0.50	8.0	-	-	4.0
W-Wall	-	0.50	0.0	-	-	0.0
	ΣR_x =1.0	ΣR_y =1.0			Σ Y R_x =6.93	Σ X R_y =4.0

X_{CR} = $\Sigma XR_y / \Sigma R_y$ = 4.0 m from West wall

Y_{CR} =$\Sigma YR_x / \Sigma R_x$= 6.93.m from South wall

Torsional Eccentricity

Torsional Eccentricity in y- direction:

Eccentricity between centre of mass and centre of rigidity

e_y = 7.50 - 6.93 =0.57m

Adding minimum 5% accidental eccentricity

0.05x15=0.75m

Total eccentricity = 0.57 + 0.75 = 1.32 m

Torsional eccentricity in X- direction

Eccentricity between centre of mass and centre of rigidity

e_x= 4.0 – 4.0 =0.00 m

Adding minimum 5% accidental eccentricity

0.05x8=0.40 m

Total eccentricity = 0.00 + 0.40 = 0.40 m

TORSIONAL MOMENT

The torsional moment due to E-W seismic force rotate the building in y direction.

Hence $M_{TX} = V_x C_y$

$= 543 \times 1.32$

$= 716.76$ KNm

Similarly if considered seismic force in N-S direction

Hence $M_{Ty} = V_y C_x$

$= 543 \times 0.4$

$= 217.2$ KN-m

$V_y = V_x$ because S_a/g is constant value of 2.5 for Time period $0.11 \leq T \leq .55$

Distribution of forces in North and South shear walls

Since we are considering the seismic force only in E- W direction, the walls in N-S direction will resist the forces and the walls in E-W direction may be ignored. Table 3.3 shows the calculation of distribution of direct shear and torsional shear.

Table 3.3 Distribution of forces in North and South shear walls

Item	R_x	dy (m)	R_x dy	R_x dy^2	Direct shear force (KN)	Torsional shear force (KN)	Total shear force (KN)
N-wall	0.462	8.07	3.728	31.67	250.866	+47.75	298.616
S-wall	0.538	6.93	3.728	24.3	292.134	-47.75	292.134

Negative torsional shear shall be neglected.

Distance of considered wall from centre of rigidity $= 15-6.93$

$= 8.07$ m

Direct shear force from North-wall $= V_x R_N$

$= 543 \times 0.462$

$= 250.866$ KN

Direct shear force from South-wall = $V_x R_s$

$=543 \times 0.538$

$=292.134$ KN

Torsional force in North-wall = $(\Sigma R_x\, dy)/(\Sigma R_x\, dy^2) \times (V_x e_y)$

$= 3.728/55.96 \times 716.76$

$=47.75$ KN

Table 3.4 Distribution of forces in East and West shear walls

Item	R_y	dx * (m)	R_y dx	$R_y\, dx^2$	Direct shear force (KN)	Torsional shear force (KN)	Total shear force (KN)
E-wall	0.5	11	5.5	60.5	271.5	-19.745	271.5
W-wall	0.5	4	2	8 Total=68.5	271.5	-7.180	278.68

Distance of considered wall from centre of rigidity = (15 - 4) = 11 m

Direct shear force in East wall = $V_y R_E$

$=543 \times 0.5$

$= 271.5$ m

Direct shear force in West wall = $V_y R_w$

$=543 \times 0.5$

$= 271.5$ m

Torsional force in West-wall = $(\Sigma R_y\, dx)/(\Sigma R_y\, dx^2) \times (V_y e_x)$

$= 5.5/60.5 \times 217.2$

$=19.745$ KN

Torsional force in West-wall = $(\Sigma R_y\, dx)/(\Sigma R_y\, dx^2) \times (V_y e_x)$

$= 2/60.5 \times 217.2$

$=7.180$ KN

DISTRIBUTION OF THE TOTAL SHEAR TO INDIVIDUAL PIERS WITHIN THE WALL

The shear carried by the North and South shear walls is now distributed to individual piers on the basis of their respective stiffness.

NORTH SHEAR WALL:

Pier Group	stiffness	Relative stiffness	Shear force
1,2,3,9	0.465	0.915	273.233
Pier 4	0.043	0.085	2323

Shear 273.233 KN in pier group 1,23,9 is further subdivided in vertical piers 1, 2, 3 on proportion to their stiffness. The stiffness of pier 1, 2, 3 is 0.25 each, so the shear force carried by each pier is

Pier	Stiffness	Relative stiffness	Shear force (KN)
1	0.25	0.33	273.233 × 0.33= 90.17
2	0.25	0.33	90.17
3	0.25	0.33	90.17
4	0.043	0.085	298.676 × 0.085 = 23.22
5	0.25	0.225	65.73
6	0.615	0.55	160.67
7	0.25	0.225	65.73

3.5 STEP 4: DETERMINATION INCREASE IN AXIAL LOAD DUE TO OVERTURNING

The increase in axial load in piers due to overturning moments may be evaluated as below:

Overturning moment at second floor level

$(M_{ovt})_2 = V_r\ (h_2+h_3) + V_3\ h_2$

Total overturning moment on pier in the first storey M_{ovt}

$=(M_{ovt})_2$+(total V) x (distance to the second floor level from critical level of the pier in the first storey

Let at the sill height of the pier = h_{ir}

Thus the axial load on a pier due to overturning change to P_{ovt} is

$P_{ovt} = (M_{ovt})\ (l_iA_i)/I_n$

l_i = Distance from the centre of gravity of net wall section in the first storey to the centroid of the pier

$= {}^n\Sigma_{i=1}\ (l_iA_i)/\ \Sigma A_i$

A_i= cross-sectional area of pier in question

I_n= Moment of Inertia of net wall section in first storey

$= {}^n\Sigma_{i=1}\ l_iA_i^2$

M_{ovt} =Total shear (V_x) x (vertical distance between second floor to critical plane of weakness, assuming at the level of sill)+ (Applied overturning Moment at second floor level)

Assume the stiffness of second storey walls is the same as first storey , the total direct shear in E-W direction of seismic load i.e, in X direction is divided in North and South shear wall is proportional to their stiffness.

Direct shear in North Wall (V_{NX}) = 250.866

Direct shear in North Wall (V_{SX}) = 292.134

Distribution of Lateral force along the height of North and South wall is

NORTH SHEAR WALL

Lateral force at a height h_i= $V_{NX} \times W_i h_i^2 / ({}^n\Sigma_{i=1} W_i h_i^2)$

Lateral force at roof level

$= (250.866 \times 828 \times 12^{2)}/ ((820 \times 12^2) + (1400 \times 8^2) + (1400 \times 4^2))$

$= (250.866 \times 118080)/ 230080$

=128.74 KN

Lateral force of Second floor level

$= (250.866 \times 1400 \times 8^2)/ ((820 \times 12^2) + (1400 \times 8^2) + (1400 \times 4^2))$

=97.69 KN

Lateral force at First floor level

$= (250.866 \times 1400 \times 4^2)/ ((820 \times 12^2) + (1400 \times 8^2) + (1400 \times 4^2))$

= 24.4236 KN

SOUTH SHEAR WALL

Lateral force at Roof Level

$= 292.134 \times 820 \times 12^2/ 2300080$

=149.92 KN

Lateral force at Second floor Level

$=292.134 \times 1400 \times 8^2 /230080$

=1113.16 KN

Lateral force at first floor level

$=292.134 \times 1400 \times 4^2/230080$

=28.44 KN

M_{ovt} = Vr (h2 +h3) + V3h2 + (total V) × (distance of second floor level from critical level of the pier in the first storey)

= 128.74 (4+4) +97.69 ×4 +250.866×2.5

=2047.845

Critical height h_{cr}= 1.5+1=2.5 m

Increase in axial Load due to overturning moment

$P_{ovt} = M_{ovt}\ L_iA_i\ /I_n$

Where l_iA_i = centroid of net section of wall is calculated as shown in table 5.6

I_n = Moment of inertia of net section of wall is calculated as shown in table 5.7

Table 3.5 Calculation of Centroid of Net Section of north wall

Pier	Area (A_i) m^2	l (distance from lift edge of wall to centroid of pier) m	$A_il(m^3)$
1	1×0.25=0.25	0.5	0.125
2	1×0.25=0.25	3	0.750
3	1×0.25= 0.25	5.5	1.375
4	1×0.25=0.25	7.5	1.875
		Σl =1.0	$\Sigma\ A_il$ = 4.125

Distance from left edge to centroid of net section of wall

= 4.125/1 = 4.125 m

Table 3.6 Calculation of moment of Inertia of net section of north wall

Pier	A_i m^2	Ii (m)	$A_il(m^3)$	A_il^2 (m^4)	$I= td^3/12$	$I_n= I + A_ili^2$
1	0.25	3.625	0.906	3.285	0.25×1/12=0.02	3.305
2	0.25	1.125	0.281	0.316	0.02	0.326
3	0.25	1.375	0.344	0.472	0.02	0.492
4	0.25	3.375	0.844	2.848	0.02	2.865
	1					6.79 = 7 m^4

Increase in axial load in individual piers of north wall is determined in table 3.7

Table 3.7 Increase in axial load in Individual piers of North wall

Pier	$A_i l (m^3)$	$P_{ovt} = M_{ovt} L_i A_i / I_n$
1	0.906	265.04
2	0.281	82.20
3	0.344	100.64
4	0.844	246.91

Increase in axial Load in piers of South Wall

Overturning moment in South wall M_{ovt}

= Total shear at third floor ×critical height (h_{cr}) + $M_{ovt}(2)$

=292.134 ×2.5 + 149.92 (4+4) + 113.76×4

=2384.735 KNm

Increase in axial Load due to overturning moment

= $P_{ovt} = M_{ovt} L_i A_i / I_n$

Where $l_i A_i$ = centroid of net section of wall is calculated as shown in table 3.8

I_n = Moment of inertia of net section of wall is calculated as shown in table 3.9

Table 3.8 Calculation of Centroid of net section of south wall

Pier	Area (A_i) m^2	l (distance from lift edge of wall to centroid of pier) m	$A_i l (m^3)$
5	1×0.25=0.25	0.5	0.125
6	2×0.25=0.5	4.0	2.00
7	1×0.25= 0.25	7.5	1.875
		Σl =1.0	$\Sigma A_i l$ = 4

Distance from left edge to centroid = 4/1=4 m

Table 3.9 Calculation of moment of Inertia of net section of south wall

Pier	A_i m^2	Ii (m)	$A_i l (m^3)$	$A_i l^2$ (m^4)	$I = td^3/12$	$I_n = I + A_i li^2$
5	0.25	3.5	0.875	3.06	0.25×1/12=0.02	3.08
6	0.25	0	0	0	0.04	0.04
7	0.25	3.5	0.875	3.06	0.02	3.98
	∑=1					∑=6.20 m^4

Increase in axial load in Individual piers of South shear wall

M_{ovt} =2384.735 KNm

I_n= 6.20 m^4

Table 3.10 Increase in axial load in Individual piers of South shear wall

Pier	$A_i l (m^3)$	$P_{ovt} = M_{ovt} L_i A_i / I_n$ (KN)
5	0.875	336.55
6	0	0
7	0.875	336.55

3.6 STEP 5: DETERMINATION OF PIER LOADS, MOMENTS AND SHEAR:

Dead Load Intensity is calculated as (per metre length of wall)

North wall: first storey

1	Weight of first storey (from level of 2nd floor level to sill level)	2.5×0.25×20 =12.5 KN/m
2	Weight of second storey	4×0.25×20 =20
3	Weight of third storey	4×0.25×20 =20
4	Weight of floor at 2nd storey level (Assuming North & South shear wall	1/2(0.12× 15 ×25) = 22.5 KN/m
5	Weight of roof	1/2(0.12×15×25) =22.5 KN/m
	Total Load	9.75 KN/m

South wall: first storey

1	Weight of first storey (from level of 2nd floor level to sill level)	2.5×0.25×20 =12.5 KN/m
2	Weight of second storey	4×0.25×20 =20
3	Weight of third storey	4×0.25×20 =20
4	Weight of floor at 2nd storey level (Assuming North & South shear wall	½ (0.12× 15 ×25) = 22.5 KN/m
5	Weight of roof	½ (0.12×15×25) =22.5 KN/m
	Total Load	9.75 KN/m

Live Load Intensity (per metre length of wall) calculated as:

North wall: first storey

1	Live Load on floor (1 KN/m^3) (Assuming North and South shear wall will take equal amount of load)	½(1×15) =7.5 KN/m
2	Live Load on roof (1 KN/m^3) (Assuming North and South shear wall will take equal amount of load)	½(1×15) =7.5 KN/m
	Total Load	15 KN/m

South wall: first storey

1	Live Load on floor (1 KN/m^3) (Assuming North and South shear wall will take equal amount of load)	½(1×15) =7.5 KN/m
2	Live Load on roof (1 KN/m^3) (Assuming North and South shear wall will take equal amount of load)	½(1×15) =7.5 KN/m
	Total Load	15 KN/m

The total Axial Load (due to Dead Load, Live Load and overturning), shear and moment in the individual parts of both the shear walls are calculated in Table 3.10 & Table 3.11as below

Table 3.11 Axial Load, Moment, shear in piers of North shear wall

North wall: first storey

Pier	Effective Width of Pier	P_d^1	P_L^2	P_{ovt} (KN)	Shear V_E for moment	Moment $= V_E \times h/2$
1	1.75	170.625	26.25	265.04	90.17	45.085
2	2.5	243.75	37.5	82.20	90.17	45.085
3	2.25	219.375	33.75	100.64	90.17	45.085
4	1.5	146.25	22.5	246.91	23.22	11.61

P_d= Effective Loading Width of Pier × Dead Load Intensity in KN/m

Effective Loading width of pier = Width of pier + ½ of each adjacent opening of pier

Table 3.12 Axial Load, Moment, shear in piers of South shear wall

South wall: first storey

Pier	Effective Width Of PIER	P_d^1	P_L^2	P_{ovt} (KN)	SHEAR FORCE V_E FOR MOMENT	MOMENT $= V_E \times h/2$
5	2	195	30	336.55	65.73	32.865
6	4	390	60	0	160.67	80.335
7	2	195	30	336.55	65.73	32.865

3.7 STEP 6: DESIGN OF SHEAR WALLS FOR AXIAL LOAD AND MOMENTS

North shear wall

Table 3.13 Determination of jamb steel at the pier boundary

Pier	Moment (KNm)	Effective Depth (mm)	Area of jamb steel A_s *(mm^2)	No of Bars	P (KN)
1	45.085	900	242.002	4@10Φ	461.915
2	45.085	900	242.002	4@10Φ	363.450
3	45.085	900	242.002	4@10Φ	353.765
4	11.61	900	62.32	4@10Φ	415.660

Table 3.14 Check for adequacy of piers

Pier	P(KN)	d (m)	t (m)	f_a/F_a	f_b/F_b	$(f_a/F_a) + (f_b/F_b)$	
1	461.915	1	0.25	0.73	0.343	1.073	OK
2	363.45	1	0.25	0.58	0.343	0.19894	OK
3	353.765	1	0.25	0.56	0.343	0.903	OK
4	415.660	1	0.25	0.66	0.086	0.748	OK

Jamb steel at the pier boundary is given by:

A_s= M/(f_s X 0.9X$d_{effective}$)

f_s= 0.55 Fe

= 230 N/mm^2

$d_{effective} = d_{total}$ – cover

Adequacy of individual piers under compression & moment is checked by interaction formula

$(f_a/F_a)+ (f_b /F_b) << 1.33$

$f_a = P_{total} (P_d + P_L + P_{ovt})$

F_a= Permissible compressive stress

=2.5 N/mm^2 (As per IS:1905)

f_b= M/(td^2/6)

F_b= Permissible bending stress

= (2.5+0.25+2.5) N/mm^2

=3.125 N/mm^2 (As per IS:1905)

South shear wall

Table 3.15 Determination of jamb steel at the pier boundary

Pier	Moment (KNm)	Effective Depth (mm)	Area of jamb steel A_s *(mm^2)	No of Bars	P (KN)
5	32.865	900	176.41	3@10Φ	561.55
6	80.335	1800	215.61	3@10Φ	450
7	32.865	900	176.41	3@10Φ	561.55

Table 3.16 Check for adequacy of piers

Pier	P(KN)	d (m)	t (m)	f_a /F_a	f_b /F_b	(f_a /F_a) + (f_b /F_b)	
5	561.55	1	0.25	0.898	0.2504	1.1484	OK
6	450	2	0.25	0.36	0.1536	0.5236	OK
7	561.55	1	0.25	0.898	0.2504	1.1484	OK

3.8 STEP 7: DESIGN OF SHEAR WALLS FOR SHEAR

DESIGN OF BOND BEAM

Total seismic shear in E-W direction = 543 KN

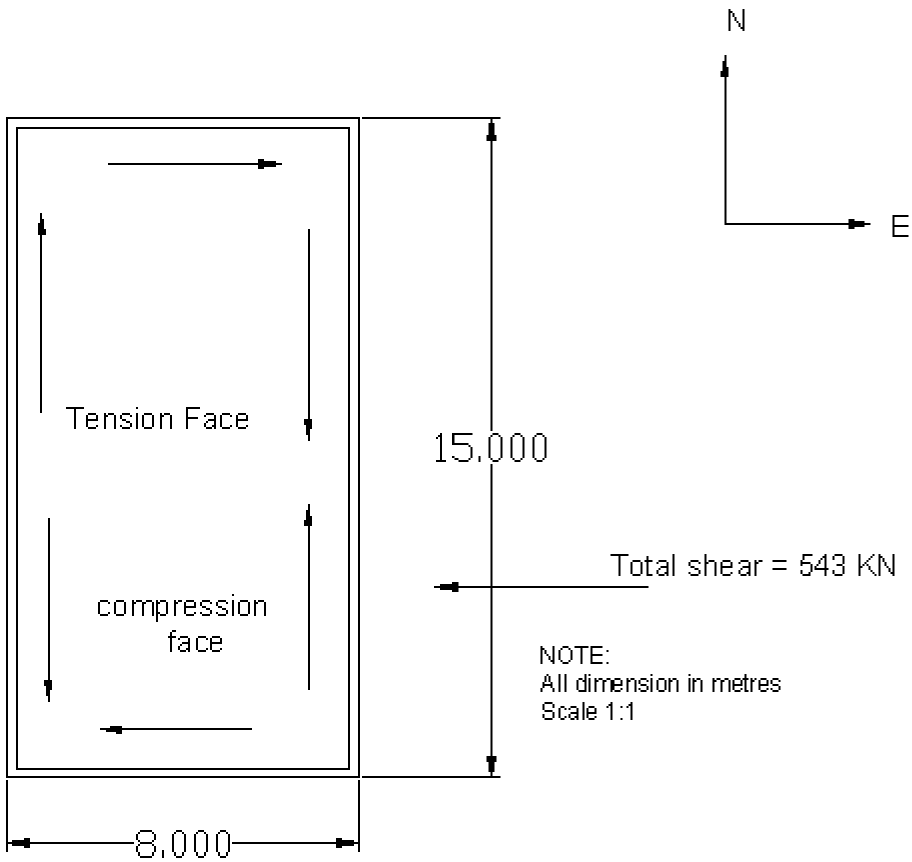


Fig 3.5 Design of bond beam

Moment produced (M)

= V x L/8 = 543 x 15/8

= 1018.125 KN-m

T = M/d = 1018.125/8 = 127.265 KN

$A_s = T/f_s$ = 127.265 x 1000/230

= 553.326 mm^2

Use 3 bars 16 Φ (= 602 mm^2)

3.9 STEP 8: STRUCTURAL DETAILS

([IS 4326:1993][3] & (Refer to Appendix 1.6, 1.7, 1.8, 1.9, 1.10, 1.11, 1.12)

1. The three storeyed residential building belongs to category C thus strengthening (a to g) to be provided in all storeyes includes the following:

   a) Masonry mortar

   b) Lintel Band

   c) Roof Band

   d) Vertical Steel especially at corners and junctions of walls

   e) Vertical steel at jambs of openings

   f) Plinth band

   g) Dowel Bars.

LINTEL BAND

It is a band provided at lintel level on all load bearing internal, external longitudinal and cross walls. The band is made of reinforced concrete not leaner than M15, or reinforced brickwork not leaner than 1:3.

ROOF BAND

It is band provided immediately below the roof or floors.

PLINTH BAND

It is a band provided at plinth level of walls on top of the foundation wall .This is to provided where strip footings of masonry are used and the soil is either uneven in its properties as frequently happens in hill tracts

The vertical thickness of RC Band is kept minimum of 150 mm. The longitudinal steel bars are 4 No. of bars of diameter 12mm.The longitudinal steel bar is held in position by steel links or stirrups 6mm diameter spaced at 150 mm apart.

VERTICAL REINFORCEMNENT

The Diameter of HYSD steel used as vertical reinforcement is masonry walls is 12mm Φ in top storey, 16 mm Φ in middle and bottom storey .The vertical reinforcement is covered with concrete M15 or mortar 1:3 grade in suitably created pockets around the bars. This will ensure their safety from corrosion and good bond with masonry. The vertical reinforcement is properly embedded in the plinth masonry of foundations and roof slab so as to develop tensile strength in bond. It shall be passing through the lintel band and floor slabs or floor level band in all storeys. The bars in each storey may be welded or lapped suitably.

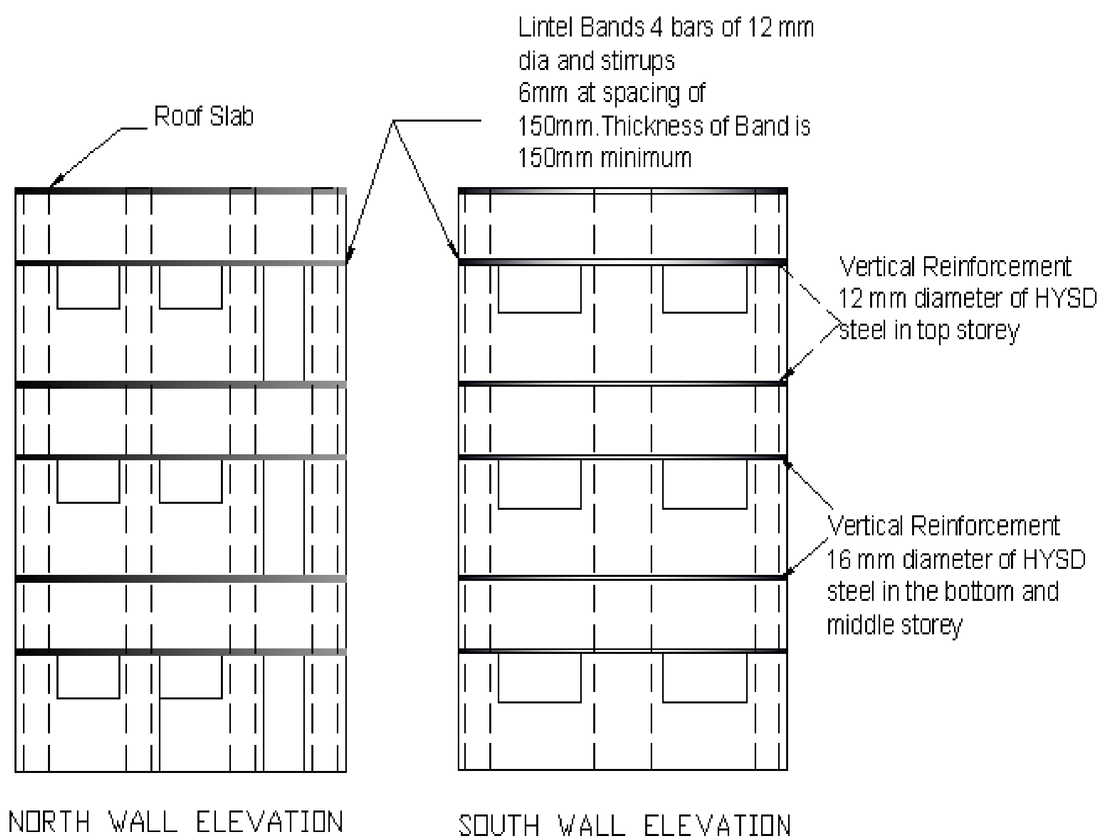


Fig 3.5 Reinforcement detailing of the building

3.10 STAIRCASE DETAILS

In this work we have not done design of staircase. The staircase design should be done separately. To avoid bracing /strut action, the staircase should be separated from the rest of the building

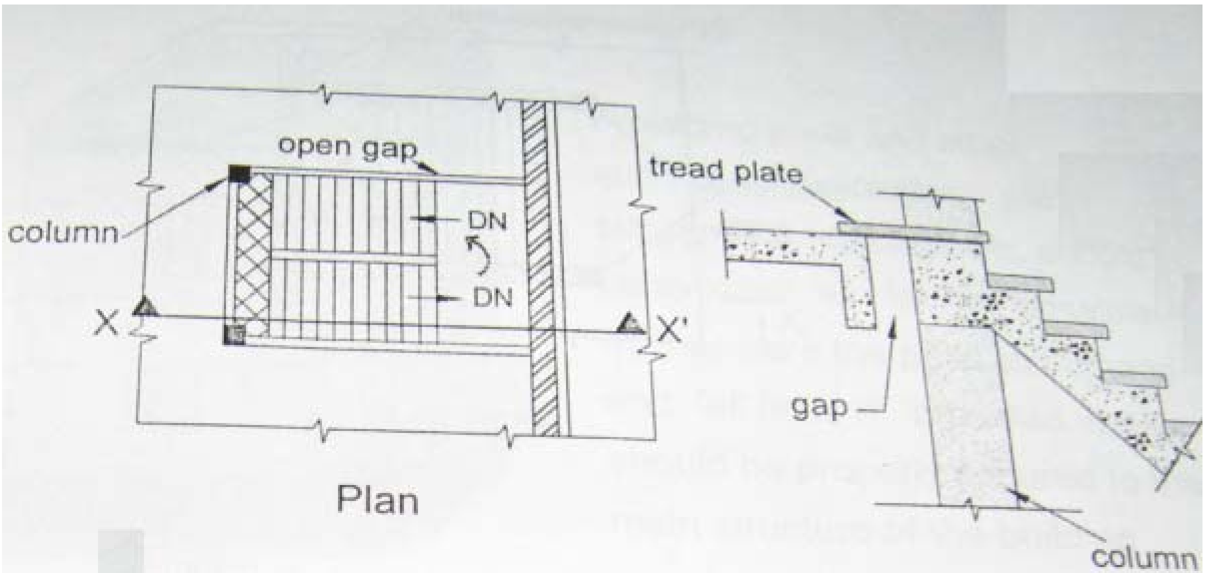


Fig 3.6 Separated staircase

3.11 PARTITION WALLS

The partition wall has not been considered in this design of this masonry building. The partition wall increases the stiffness of the structure, thus by not considering its design we have considered for the critical side and safe design

CONCLUSIONS

Masonry buildings are the most common type of construction used for all housing around the world. But the post earthquake survey has proved that the masonry buildings are most vulnerable to and have suffered maximum damages in the past earthquakes. A survey of the affected areas in past earthquakes demonstrated that the major losses of lives were due to collapse of low-strength masonry buildings. Due to the brittleness of the masonry material, lack of ductility, strength and locally used traditional material in a traditional manner without the earthquake-resistant features are the main causes of collapse of building during earthquake.

The present work is a step towards with regard to illustrate a procedure for seismic analysis and design of masonry building. The procedure has been presented by considering each clause as mentioned in IS 1905 and IS 4326:1993 with the help of an example of a three-storeyed residential masonry building.

CONCLUSION DRAWN:

1. The best shapes of earthquake resistant buildings are regular shapes and preferably with two axes of symmetry. This ensures the centre of gravity and rigidity will be the same or close to each other resulting in minimization of torsion moment in building.
2. Provision of bonds at different level increases the number of lateral force and thereby reduces the effect of seismic forces remarkably.
3. Provision of vertical reinforcement in flexural walls helps to resist the moments generated due to seismic force. This in turn helps in safe distribution of the lateral load to the shear walls.
4. Provision of vertical reinforcement in shear walls increases the load carrying capacity and the flexural strength of the wall.
5. Vertical steel at walls especially at corners, at openings of shear wall resists the compression and flexure forces helps in preventing sliding or collapse of building.

6. A number of construction aspects are required to ensure the box action. Firstly, connections between the walls should be good. This can be achieved by (a) ensuring good interlocking of the masonry courses at the junctions, and (b) employing horizontal hooks, at various levels, at intersection of the orthogonal walls.

7. The sizes of openings need to be kept small and preferably closer to the centre. The smaller the opening the larger is the resistance offered by the wall.

8. Lastly, the tendency of a wall to topple when pushed in the weak direction can be reduced by limiting its length-to-thickness and height-to-thickness ratio called the slenderness ratio.

REFERENCES

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BY PANKAJ AGARWAL & MANISH SHRIKHANDE
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2. GUIDELINES FOR EARTHQUAKE RESISTANT BUILDINGS, NPEEE SHORT TERM COURSES ON EARTHQUAKE RESISTANT DESIGN OF RC BUILDINGS JULY 24-29, 2006 IIT ROORKEE

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5. MIA, MASONRY CODE AND SPECIFICATION,A Co-PUBLICATION OF MASONRY INSTITUTE OF AMERICA AND CRC PRESS,NEWYORK 1998.

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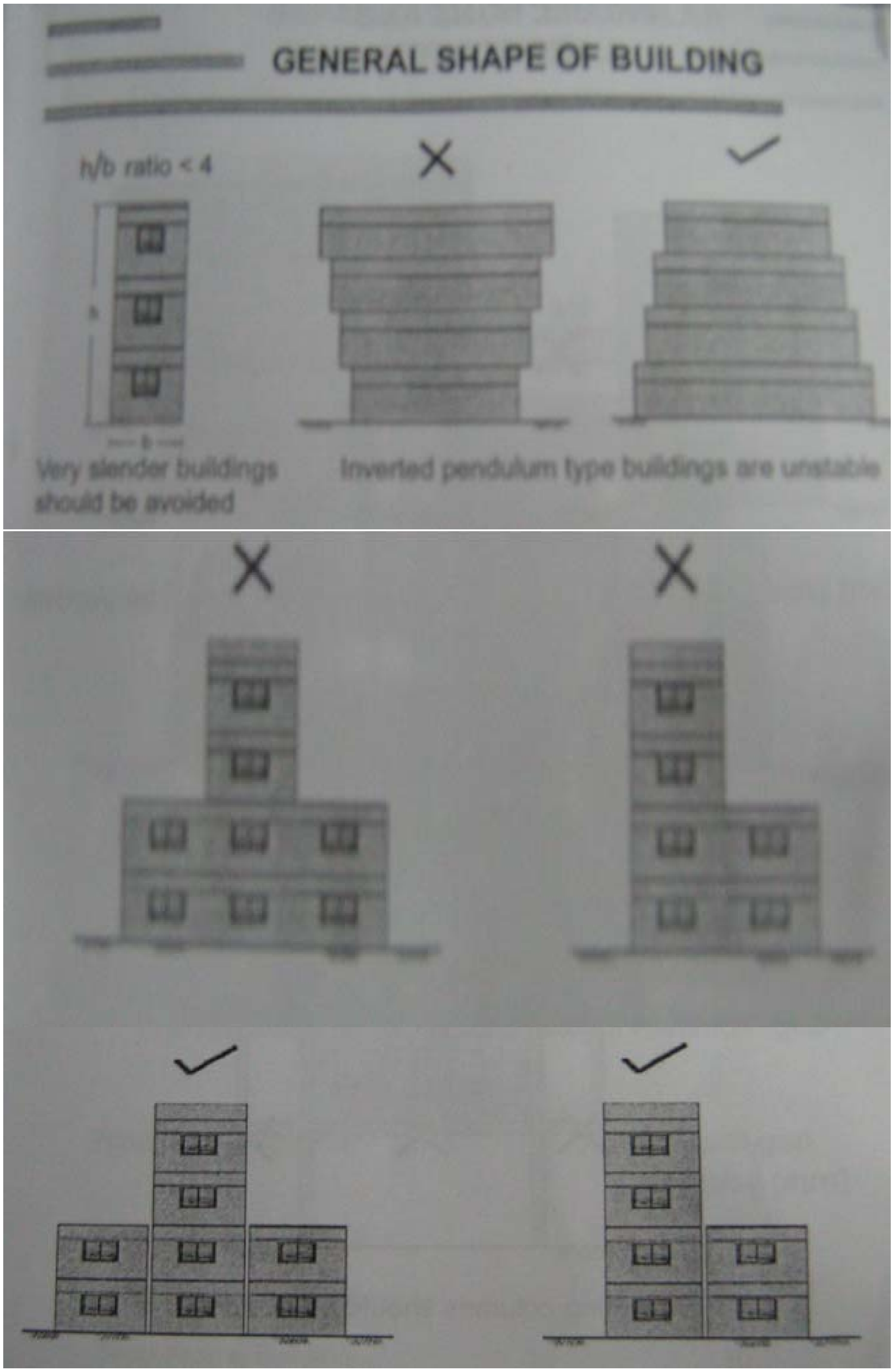
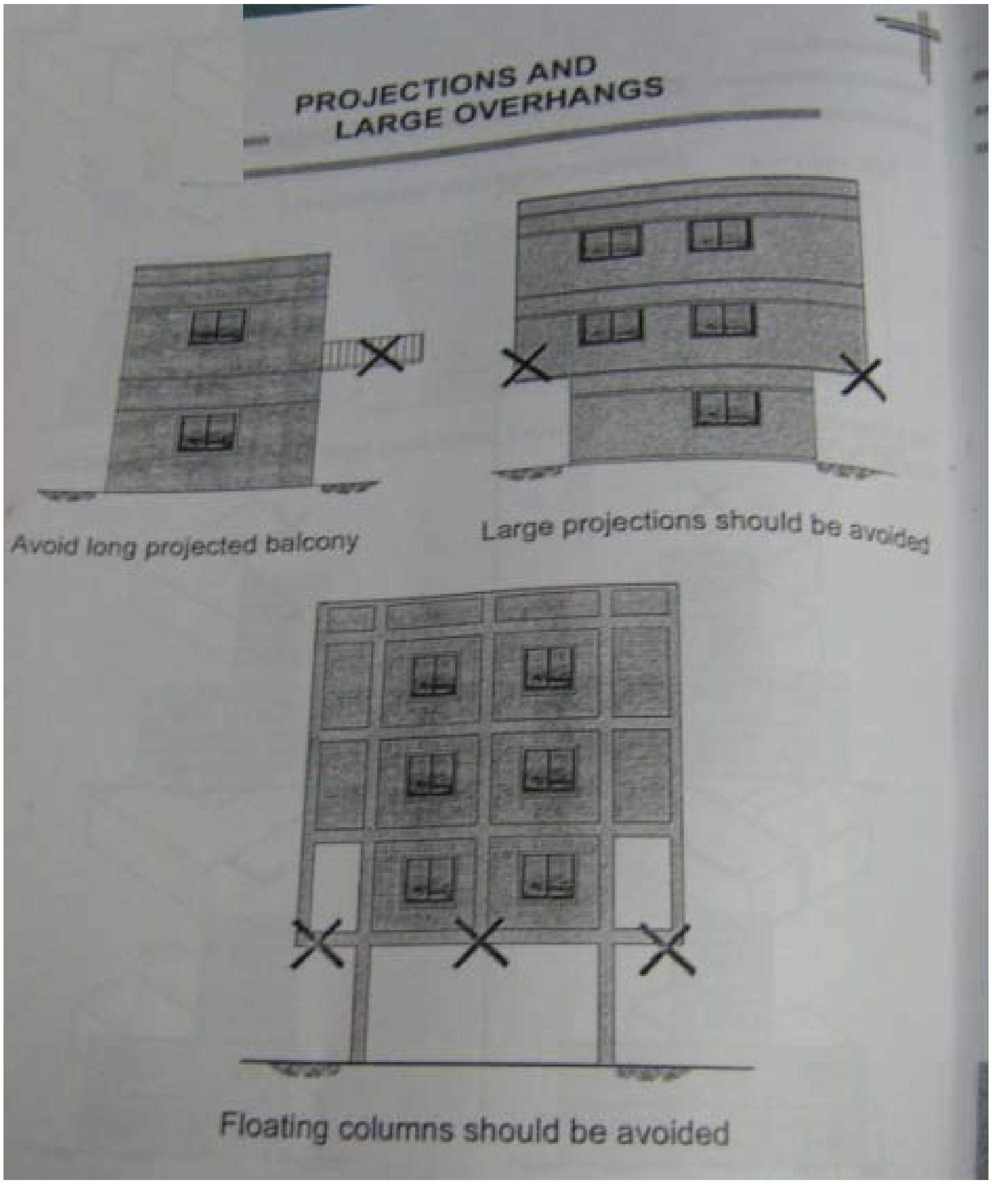
7. IS 1893 (PART I), 2002, INDIAN STANDARD CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES (5TH REVISION).

8. IS 1905: CODE OF STUCTURAL USE OF UNREINFORCED MASONRY (3RD REVISION)

9. APPENDIX **A**

APPENDIX A


GENERAL SHAPE OF BUILDING
h/b ratio < 4
Very slender buildings should be avoided
Inverted pendulum type buildings are unstable



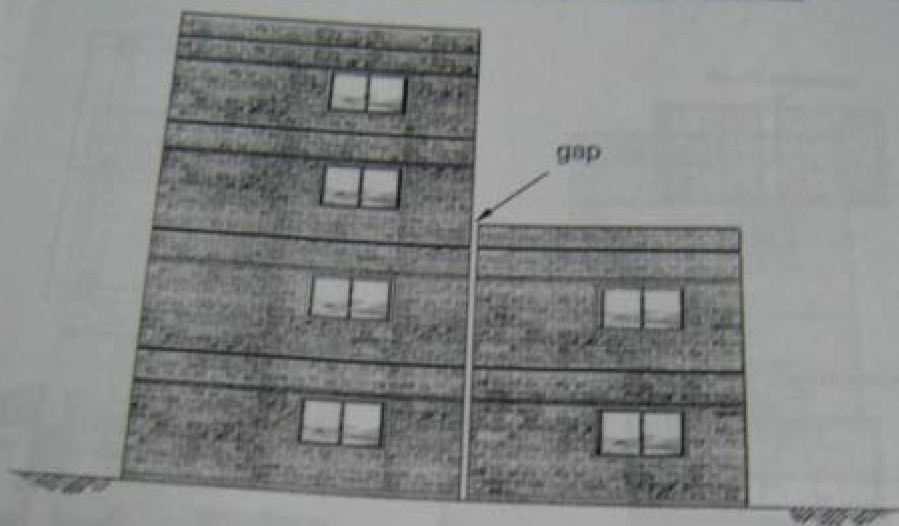
PROJECTIONS AND LARGE OVERHANGS

Avoid long projected balcony

Large projections should be avoided

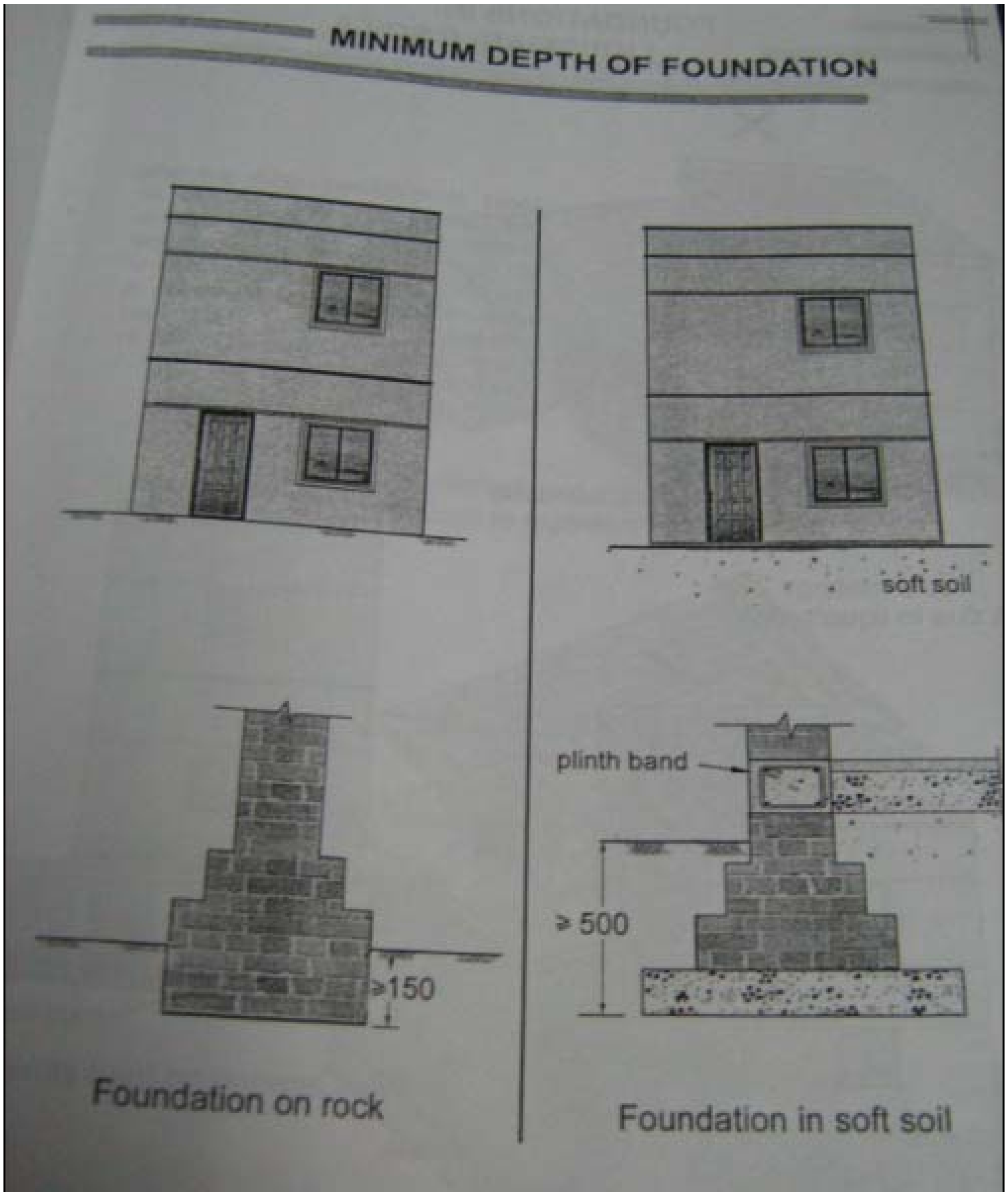
Floating columns should be avoided

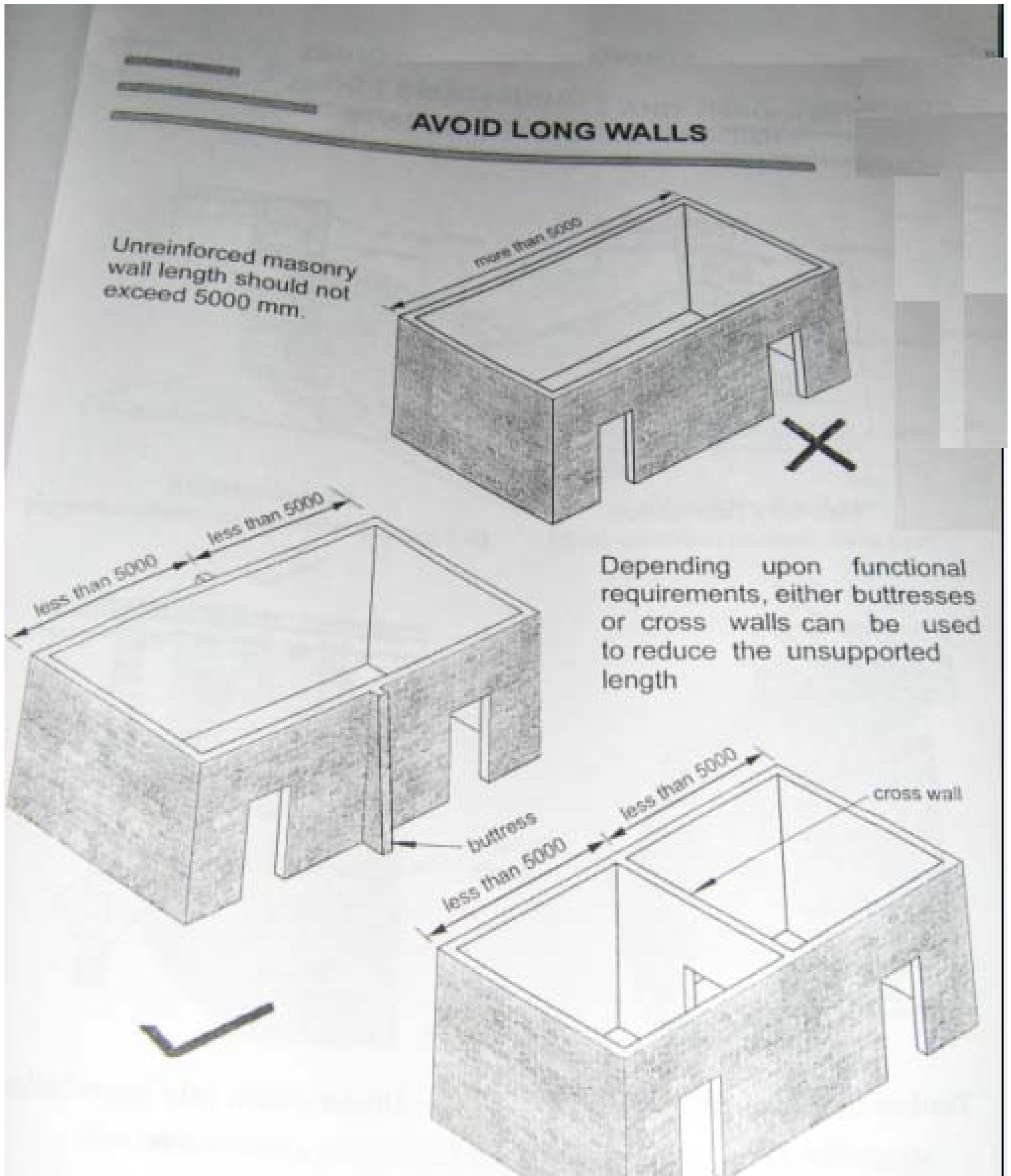
SEPARATION OF DISSIMILAR BUILDINGS

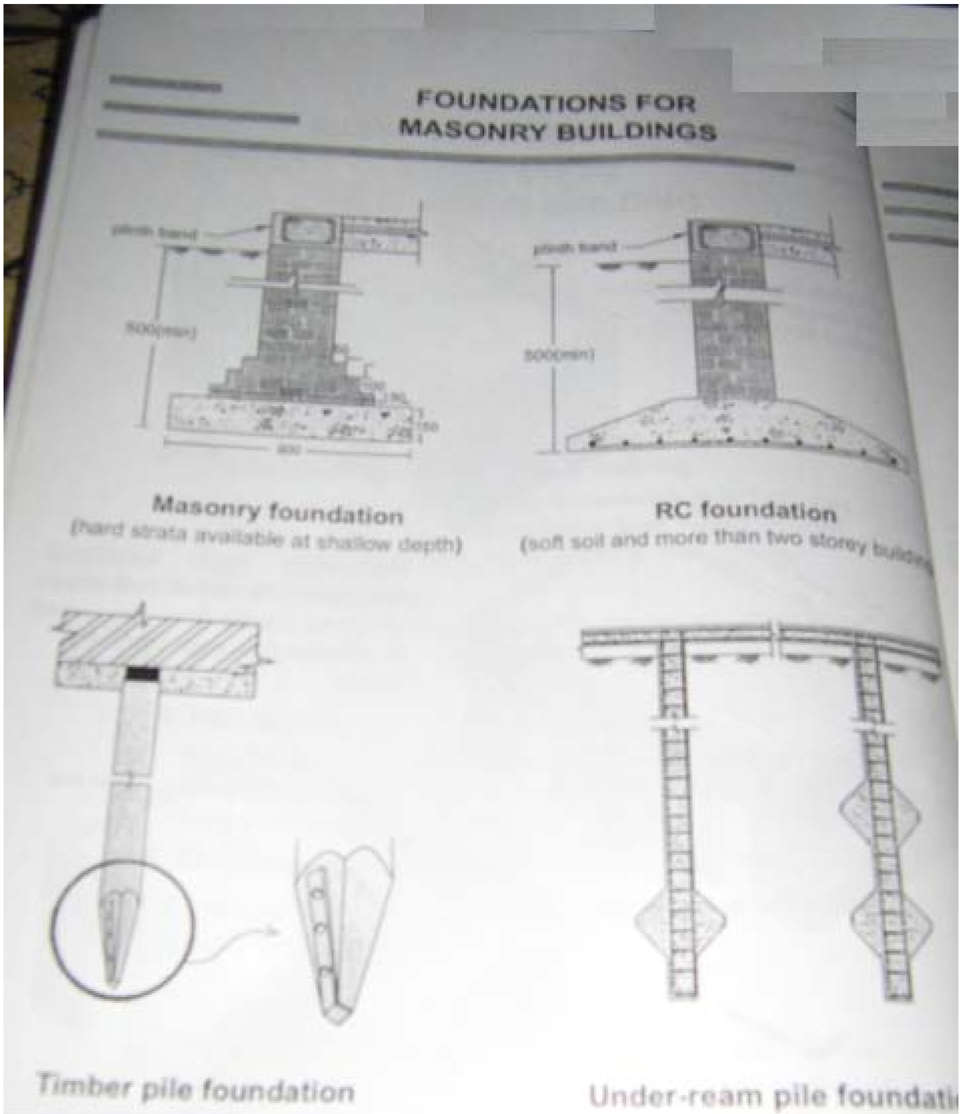


To avoid collision, adjacent dissimilar buildings should be separated by a minimum gap

Type of construction	Minimum gap per storey (mm)
Load Bearing Building	15
RCC Frame Building	20
Steel Frame Building	30


MINIMUM DEPTH OF FOUNDATION
soft soil
plinth band
≥ 500
≥150
Foundation on rock
Foundation in soft soil


AVOID LONG WALLS
Unreinforced masonry wall length should not exceed 5000 mm.
more than 5000
less than 5000
less than 5000
Depending upon functional requirements, either buttresses or cross walls can be used to reduce the unsupported length
buttress
less than 5000
less than 5000
cross wall

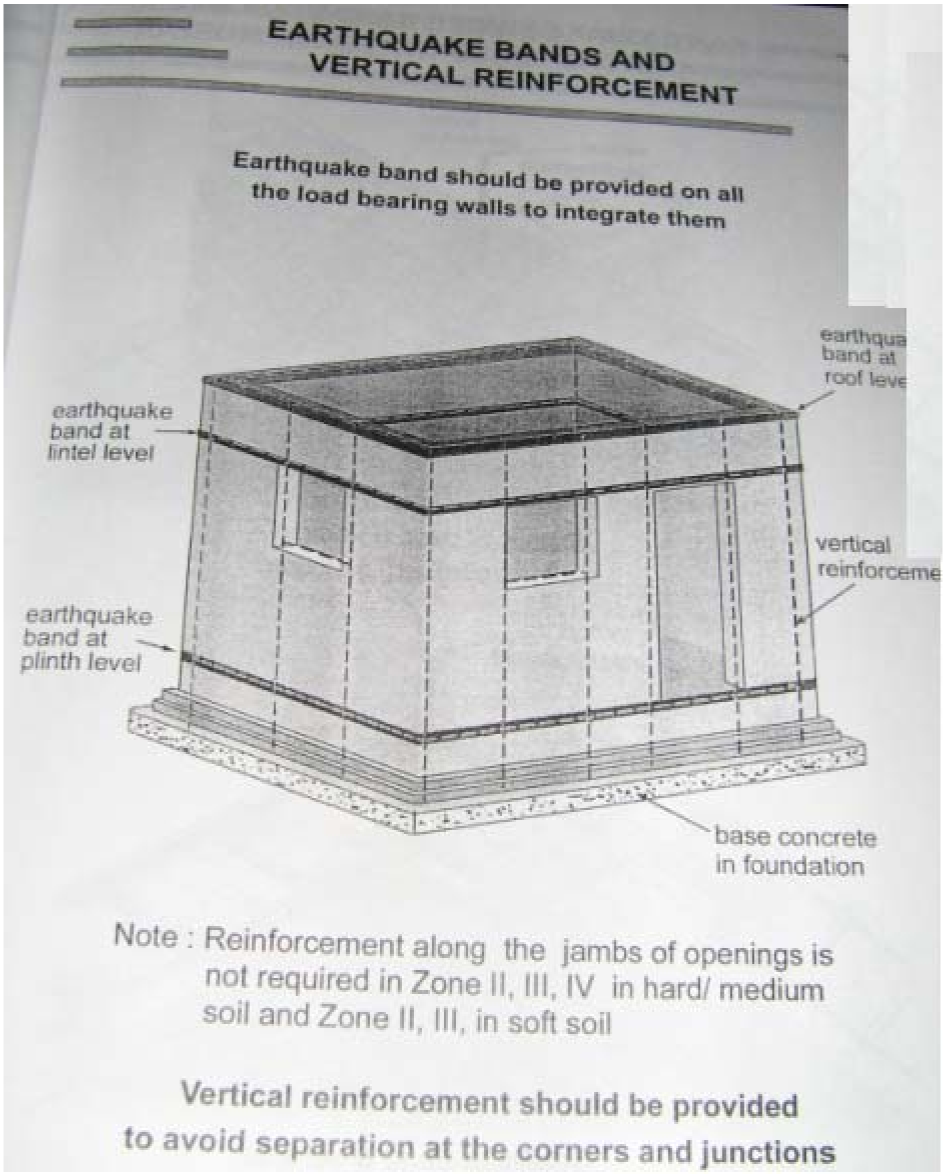

FOUNDATIONS FOR MASONRY BUILDINGS

Masonry foundation
(hard strata available at shallow depth)

RC foundation
(soft soil and more than two storey build…

Timber pile foundation

Under-ream pile foundati…

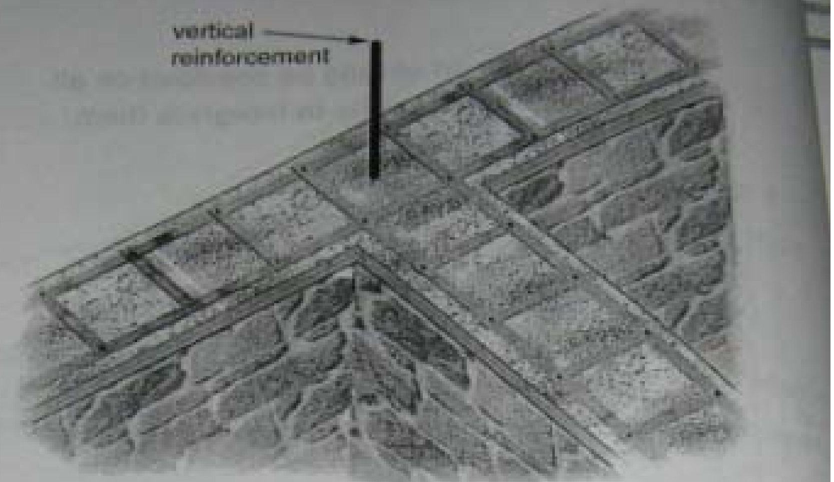


EARTHQUAKE BANDS AND VERTICAL REINFORCEMENT

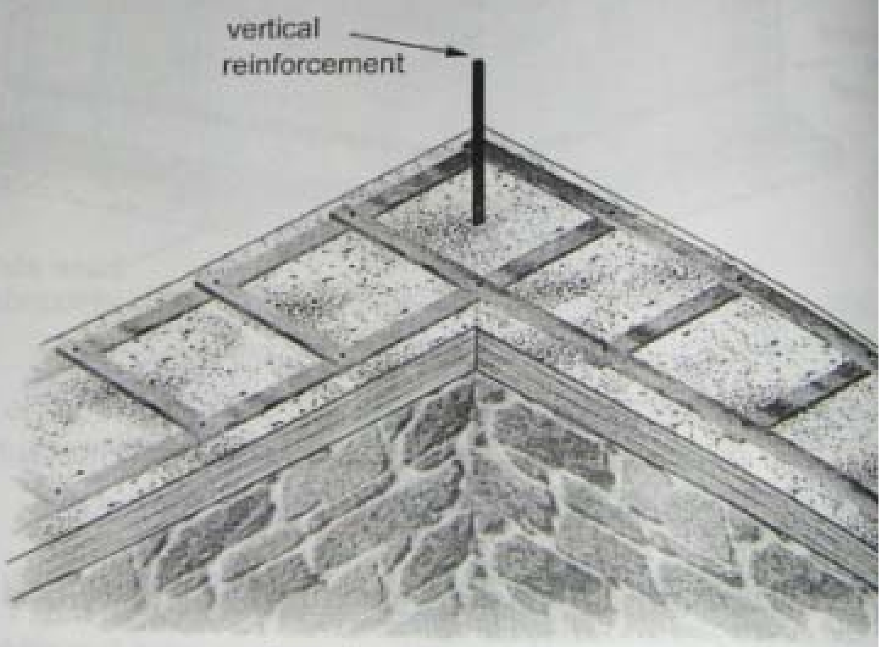
Earthquake band should be provided on all the load bearing walls to integrate them

Note : Reinforcement along the jambs of openings is not required in Zone II, III, IV in hard/ medium soil and Zone II, III, in soft soil

Vertical reinforcement should be provided to avoid separation at the corners and junctions



Details of band & vertical reinforcement at T-junction



Details of band & vertical reinforcement at L-junction

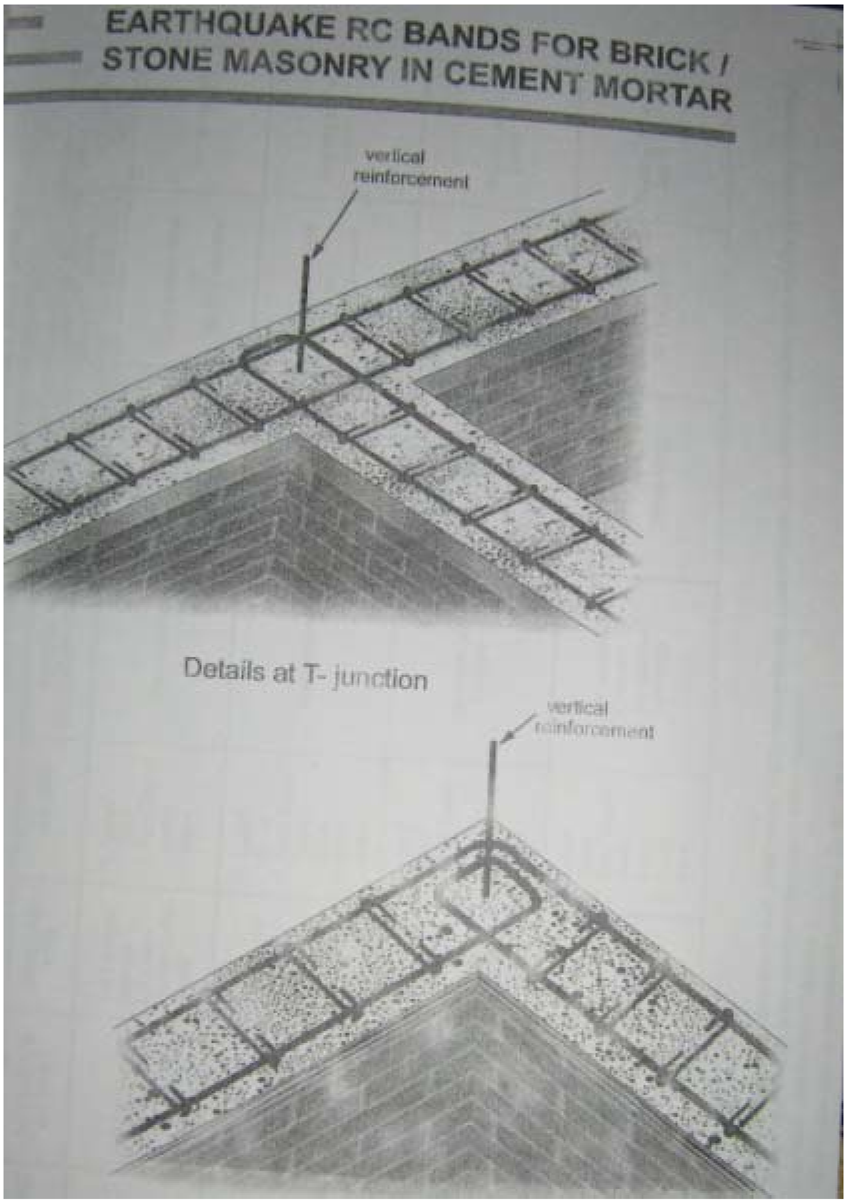

EARTHQUAKE RC BANDS FOR BRICK / STONE MASONRY IN CEMENT MORTAR
vertical reinforcement
Details at T- junction
vertical reinforcement

TABLE 1- EARTHQUAKE RESISTANT PROVISIONS IN MASONRY BUILDINGS
(BUILDINGS WITH STRIP FOUNDATION ON ROCK/ HARD SOIL/ MEDIUM SOIL)

ZONE	MAXIMUM HEIGHT	MORTAR MIX	BANDS	VERTICAL REINFORCEMENT	DIAMETER OF VERTICAL REINFORCEMENT	MORTAR MIX FOR BAND / VERTICAL REINFORCEMENT	BRACING OF PITCHED ROOF
II	Four Storey (Less than 15m)	1:6 Cement-Sand or 1:3 Lime-Cinder/Lime-Surkhi	1. Lintel Band 2. Roof Band/ Gable Band	Not Required	Not Required	1:3 Cement-Sand or M 15 Concrete	Not Required
III	Four Storey (Less than 15m)	1:6 Cement-Sand or 1:2:9 Cement-Lime-Sand	1. Lintel Band 2. Roof Band/ Gable Band 3. Plinth Band	Not Required	Not Required	1:3 Cement-Sand or M 15 Concrete	Bracing at Tie Level
IV	Four Storey (Less than 15m)	1:6 Cement-Sand or 1:2:9 Cement-Lime-Sand	1. Lintel Band 2. Roof Band/ Gable Band 3. Plinth Band	At Joints and Corners	Single Bar of 12 mm (TMT/TOR) or 16mm (MS)	1:3 Cement-Sand or M 15 Concrete	Bracing at Tie Level
V	Four Storey (Less than 15m)	1:4 Cement-Sand or 1:1:6 Cement-Lime-Sand	1. Lintel Band 2. Roof Band/ Gable Band 3. Plinth Band	1. At Joints and Corners 2. At Jambs of Openings	Single Bar of 20 mm (TMT/TOR) or 25mm (MS)	1:3 Cement-Sand or M 15 Concrete	Bracing at Tie Level

TABLE 2- EARTHQUAKE RESISTANT PROVISIONS IN MASONRY BUILDINGS

(BUILDINGS WITH STRIP FOUNDATION ON SOFT SOIL)

ZONE	MAXIMUM HEIGHT	MORTAR MIX	BANDS	VERTICAL REINFORCEMENT	DIAMETER OF VERTICAL REINFORCEMENT	MORTAR MIX FOR BAND / VERTICAL REINFORCEMENT	BRACING OF PITCHED ROOF
II	Four Storey (Less than 15m)	1:6 Cement-Sand or 1:3 Lime-Cinder/Lime-Surkhi	1. Lintel Band 2. Roof Band/ Gable Band	Not Required	Not Required	1:3 Cement-Sand or M 15 Concrete	Not Required
III	Four Storey (Less than 15m)	1:6 Cement-Sand or 1:2:9 Cement-Lime-Sand	1. Lintel Band 2. Roof Band/ Gable Band 3. Plinth Band	At Joints and Corners	Single Bar of 12mm (TMT/TOR) or 16mm (MS)	1:3 Cement-Sand or M 15 Concrete	Bracing at Tie Level
IV	Four Storey (Less than 15m)	1:6 Cement-Sand or 1:2:9 Cement-Lime-Sand	1. Lintel Band 2. Roof Band/ Gable Band 3. Plinth Band	1. At Joints and Corners 2. At Jambs of Openings	Single Bar of 12 mm (TMT/TOR) or 16mm (MS)	1:3 Cement-Sand or M 15 Concrete	Bracing at Tie Level
V	Four Storey (Less than 15m)	1:4 Cement-Sand or 1:1:6 Cement-Lime-Sand	1. Lintel Band 2. Roof Band/ Gable Band 3. Plinth Band	1. At Joints and Corners 2. At Jambs of Openings	Single Bar of 20 mm (TMT/TOR) or 25mm (MS)	1:3 Cement-Sand or M 15 Concrete	Bracing at Tie Level

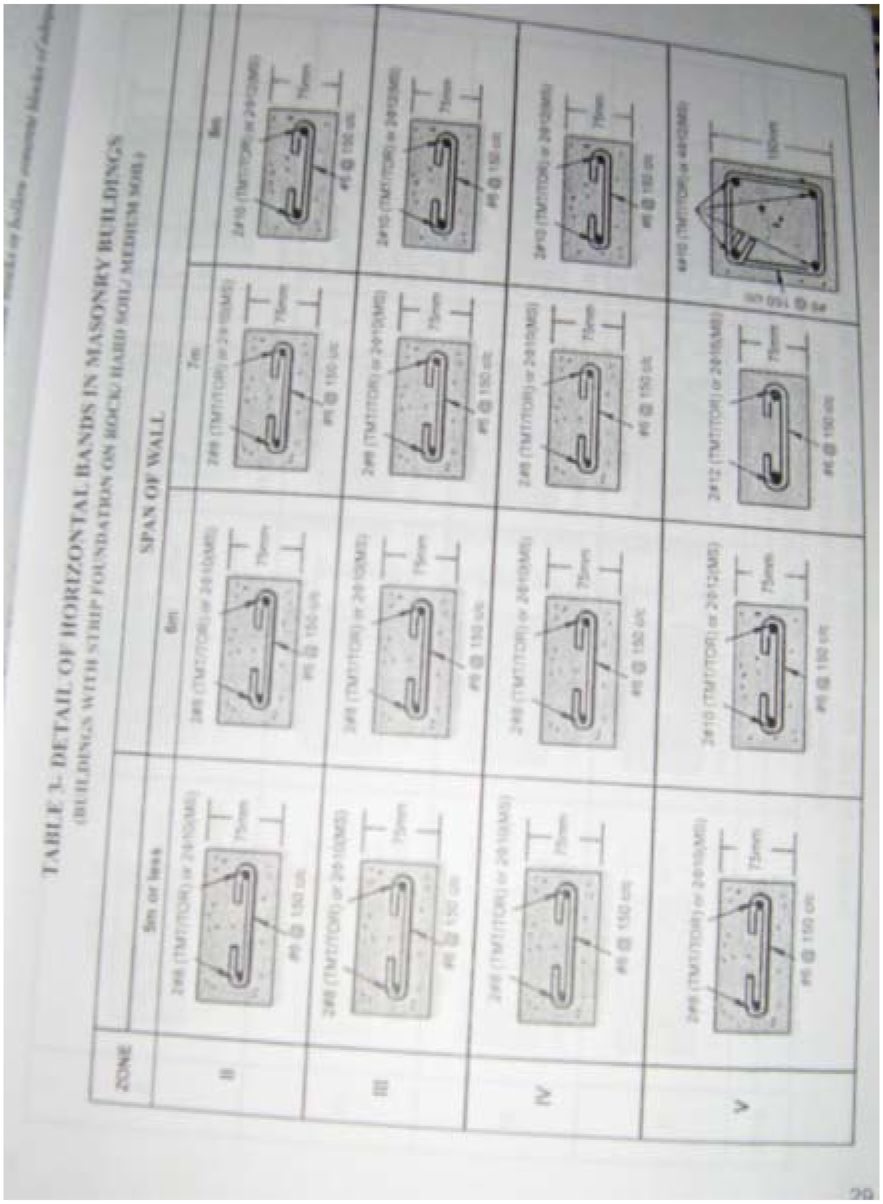


TABLE 3- DETAIL OF HORIZONTAL BANDS IN MASONRY BUILDINGS
(BUILDINGS WITH STRIP FOUNDATION ON ROCK/ HARD SOIL/ MEDIUM SOIL)

ZONE	SPAN OF WALL			
	5m or less	6m	7m	8m
II	2#8 (TMT/TOR) or 2#10(MS); 75mm; #6 @ 150 c/c	2#8 (TMT/TOR) or 2#10(MS); 75mm; #6 @ 150 c/c	2#8 (TMT/TOR) or 2#10(MS); 75mm; #6 @ 150 c/c	2#10 (TMT/TOR) or 2#12(MS); 75mm; #6 @ 150 c/c
III	2#8 (TMT/TOR) or 2#10(MS); 75mm; #6 @ 150 c/c	2#8 (TMT/TOR) or 2#10(MS); 75mm; #6 @ 150 c/c	2#8 (TMT/TOR) or 2#10(MS); 75mm; #6 @ 150 c/c	2#10 (TMT/TOR) or 2#12(MS); 75mm; #6 @ 150 c/c
IV	2#8 (TMT/TOR) or 2#10(MS); 75mm; #6 @ 150 c/c	2#8 (TMT/TOR) or 2#10(MS); 75mm; #6 @ 150 c/c	2#8 (TMT/TOR) or 2#10(MS); 75mm; #6 @ 150 c/c	2#10 (TMT/TOR) or 2#12(MS); 75mm; #6 @ 150 c/c
V	2#8 (TMT/TOR) or 2#10(MS); 75mm; #6 @ 150 c/c	2#10 (TMT/TOR) or 2#12(MS); 75mm; #6 @ 150 c/c	2#12 (TMT/TOR) or 2#16(MS); 75mm; #6 @ 150 c/c	4#10 (TMT/TOR) or 4#12(MS); 150mm; #6 @ 150 c/c

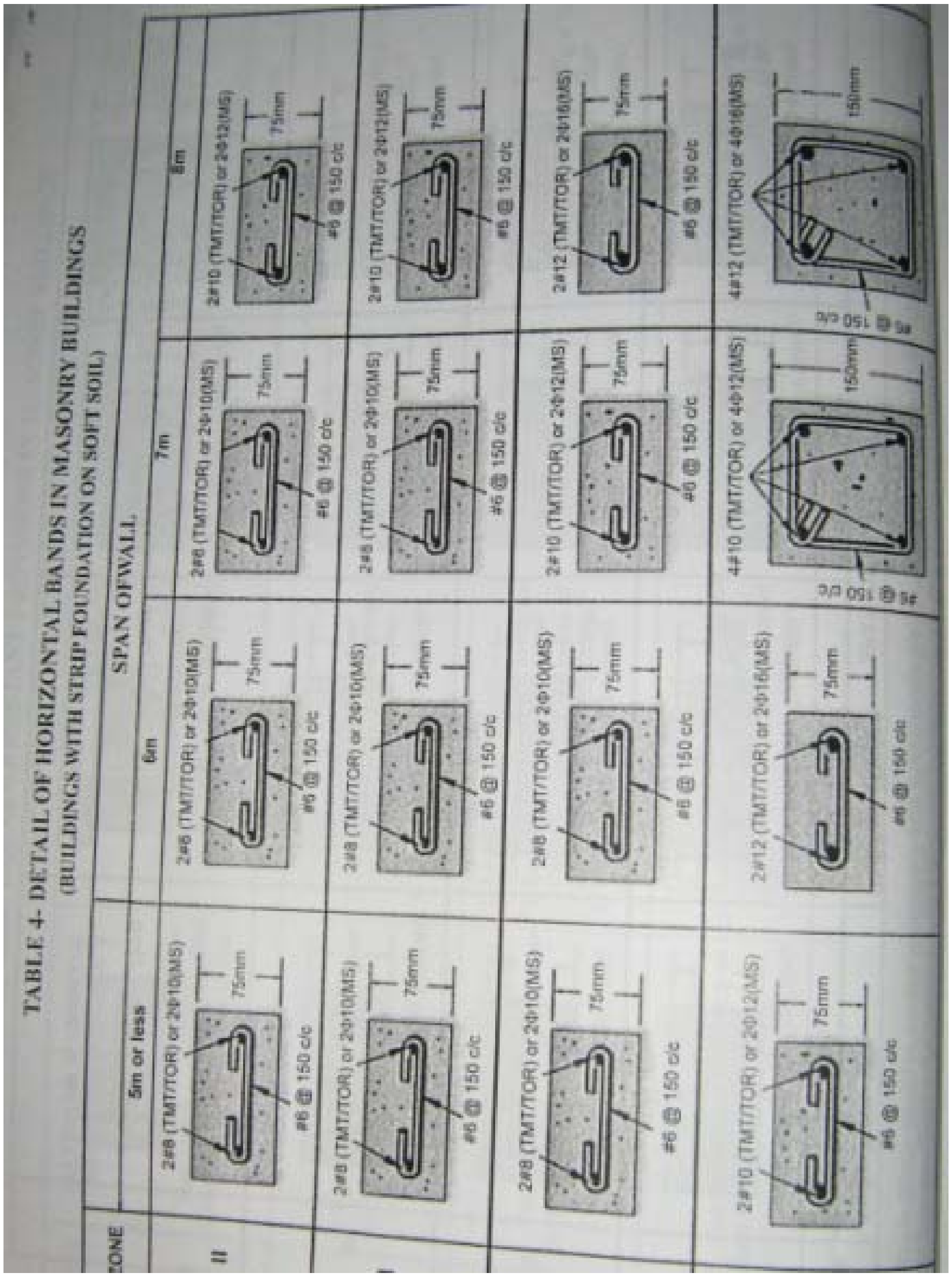


TABLE 4- DETAIL OF HORIZONTAL BANDS IN MASONRY BUILDINGS
(BUILDINGS WITH STRIP FOUNDATION ON SOFT SOIL)

ZONE	SPAN OFWALL			
	5m or less	6m	7m	8m
II	2#8 (TMT/TOR) or 2Φ10(MS), 75mm, #6 @ 150 c/c	2#8 (TMT/TOR) or 2Φ10(MS), 75mm, #6 @ 150 c/c	2#8 (TMT/TOR) or 2Φ10(MS), 75mm, #6 @ 150 c/c	2#10 (TMT/TOR) or 2Φ12(MS), 75mm, #6 @ 150 c/c
	2#8 (TMT/TOR) or 2Φ10(MS), 75mm, #6 @ 150 c/c	2#8 (TMT/TOR) or 2Φ10(MS), 75mm, #6 @ 150 c/c	2#8 (TMT/TOR) or 2Φ10(MS), 75mm, #6 @ 150 c/c	2#10 (TMT/TOR) or 2Φ12(MS), 75mm, #6 @ 150 c/c
	2#8 (TMT/TOR) or 2Φ10(MS), 75mm, #6 @ 150 c/c	2#8 (TMT/TOR) or 2Φ10(MS), 75mm, #6 @ 150 c/c	2#10 (TMT/TOR) or 2Φ12(MS), 75mm, #6 @ 150 c/c	2#12 (TMT/TOR) or 2Φ16(MS), 75mm, #6 @ 150 c/c
	2#10 (TMT/TOR) or 2Φ12(MS), 75mm, #6 @ 150 c/c	2#12 (TMT/TOR) or 2Φ16(MS), 75mm, #6 @ 150 c/c	4#10 (TMT/TOR) or 4Φ12(MS), 150mm, #6 @ 150 c/c	4#12 (TMT/TOR) or 4Φ16(MS), 150mm, #6 @ 150 c/c