SEISMIC RETROFITTING GUIDELINES

OF

BUILDINGS IN NEPAL

2013

MASONRY

UNDP

CORD

MRB & Associates

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SEISMIC RETROFITTING GUIDELINES of Buildings in Nepal

MASONRY STRUCTURES

Empowered lives.
Resilient nations.

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1. INTRODUCTION

1.1 BACKGROUND

Nepal has long history of devastating earthquakes as the country is situated in the boundary between two active tectonic plates - the Indian Plate in the south and Tibetan plate in the North. As Nepal lies in the seismic prone area and earthquake occurs frequently, the buildings in Nepal need to be designed and constructed as earthquake safer buildings. However, the structures of Nepal are mostly non-engineered and semi – engineered construction, which basically lack seismic resistance detailing. In the past earthquakes Nepal, including Kathmandu valley, has witnessed severe damage to buildings and significant loss of human lives. The damages caused by the earthquakes in the past demonstrate the vulnerability of buildings in Nepal.

The building code has not been implemented in most part of the country and majority of the buildings do not meet seismic safety standards. In the few municipalities where the building code has been implemented from last decade, the implementation is still at preliminary stage and compliance is low. The country, therefore, is accumulating vulnerable buildings and the risk is being increased each year. Immediate attention to safety of these buildings is of utmost importance as a major earthquake is inevitable in the country.

It is neither practical nor feasible to demolish all the existing buildings and construct new buildings meeting seismic safety standard. A practical approach to increasing seismic safety standard of these buildings would be to strengthen them and upgrade their level of safety. The non -engineered, semi –engineered structures or 'engineered' structures which were built before the implementation of the code or which do not meet existing seismic safety standard can be rebuilt or reconstructed or strengthened or retrofitted to improve their performance during earthquake

1.2 PUPRPOSE

The primary purpose of this document is to provide an analysis and design methodology for use in the seismic evaluation and retrofit of the existing masonry buildings in Nepal. It is expected that this document will be used by retrofit design professionals for performing seismic evaluations and retrofit designs.

1.3 OBJECTIVE AND SCOPE

The objective of this document is to reduce vulnerability of buildings thereby decreasing likelihood of risk to loss of life and injury to the habitants of the buildings. This is accomplished by limiting the likelihood of damage and controlling the extent of damage in the building

1.4 CONCEPT OF REPAIR, RESTORATION AND RETROFITTING[1]

1.4.1 REPAIR

Repair to a damaged building is done in order to enable it to resume all its previous functions and to bring back its architectural shape. Repair does not pretend to improve the structural strength of the building and can be very deceptive for meeting the strength requirements of the next earthquake. The actions will include the following:

i. Patching up of defects such as cracks and fall of plaster.
ii. Repairing doors, windows, replacement of glass panes.
iii. Checking and repairing electric wiring
iv. Checking and repairing gas pipes, water pipes and plumbing services.
v. Re-building non-structural walls, smoke chimneys, boundary walls, etc.
vi. Re-plastering of walls as required.
vii. Rearranging disturbed roofing tiles.
viii. Relaying cracked flooring at ground level.
ix. Redecoration, whitewashing, painting, etc.

Repair restores only the architectural damages but do not restore the original structural strength of cracked walls or columns. So a repaired building may be very illusive as it will

[1] Adapted from IAEE Manual

hide all the weaknesses and the building will suffer even more severe damage if shaken again by an equal shock since the original energy absorbing capacity will not be available.

1.4.2 RESTORATION

It is the restoration of the strength the building had before the damage occurred. Restoration is done whenever there is evidence that the structural damage can be attributed to exceptional phenomena that are not likely to happen again and that the original strength provides an adequate level of safety.

The main purpose is to carry out structural repairs to load bearing elements. It may also involve cutting portions of the elements and rebuilding them or simply adding more structural material so that the original strength is more or less restored. The process may involve inserting temporary supports, underpinning, etc. Some of the approaches are stated below:

(i) Removal of portions of cracked masonry walls and piers and rebuilding them in richer mortar. Use of non-shrinking mortar will be preferable.

(ii) Addition of reinforcing mesh on both -faces of the cracked wall, holding it to the wall through spikes or bolts and then covering it suitably. Several alternatives have been used.

(iii) Injecting epoxy like material, which is strong in tension, into the cracks in walls, columns, beams, etc.

Where structural repairs are considered necessary, these should be carried out prior to or simultaneously with the architectural repairs so that total planning of work could be done in a coordinated manner and wastage is avoided.

1.4.3 SEISMIC STRENGTHENING (RETROFITTING)

Retrofitting is an improvement over the original strength when the evaluation of the building indicates that the strength available before the damage was insufficient and restoration alone will not be adequate in future quakes. The original structural inadequacies, material degradation due to time, and alterations carried out during use over the years such as making new openings, addition of new parts inducing dissymmetry in plan and elevation are responsible for affecting the seismic behavior of old existing buildings. But due to historical, artistic, social and economic reasons, generally substituting these weak structures with new earthquake resistant buildings is neglected.

2. DAMAGE PATTERNS

This chapter describes generally observed damaged patterns in masonry structures either due to earthquakes or due to other reasons including lack of maintenance.

It is important to understand the relative severities of the various types of damage as they relate to life safety and the protection of historic building fabric. By doing so, priorities for stabilization, repairs, and/or seismic retrofits can be established for each type of damage. If a particular damaged area or component of a building is likely to degrade rapidly if not repaired, then that damaged element assumes a higher priority than others that are not likely to deteriorate. If damage to a major structural element, such as a roof or an entire wall, increases the susceptibility to collapse, then a high priority is assigned because of the threat to life safety. If damage that could result in the loss of a major feature, such as a wall, compromises the historic integrity of the entire structure, then it is more critical than damage that would result in partial failure, but no loss.

2.1 CATEGORIZATION OF DAMAGE / VULNERABILITY

Table 2-1 : Damage Categories[2]

S.No.	Categories	Wall	Floor / Roof
1	No Damage	No Damage	No Damage
2	Slight Non-Structural Damage	Thin cracks in plaster, falling of plaster bits in limited parts	Thin cracks in small areas, tiles only slightly disturbed
3	Slight Structural Damage	Small cracks in walls, falling of plaster in large areas: damage to non-structural parts like chhajjas, parapets	Small cracks in slabs / A.C sheets; tiles disturbed in about 10% area: minor damage in under-structure of sloping roof

2 Based on I.A.E.E. Guidelines, further developed through observations in earthquakes in India, by Dr. A.S. Arya, Seismic Advisor, G S D M A.

4	Moderate Structural Damage	Large and deep cracks in walls; widespread cracking of walls, columns and pier; or collapse of one wall. The load carrying capacity of structure is partially reduced	Large cracks in slabs; some A.C sheets, broken; upto 25% tiles disturbed / fallen moderate damage to understructure of sloping roofs
5	Sever Structural Damage	Gaps occur in walls; two or more inner or outer walls collapse; Approximately 50% of the main structural elements fail. The building takes a dangerous state	Floor badly cracked, part may fall; under-structure of sloping roof heavily damaged, part may fall; tiles badly affected & fallen
6	Collapse	A large part or whole of the building collapses	A large part or whole floor and roof collapses or hang precariously

2.2 DAMAGE TYPOLOGIES[3]

The following subsections include descriptions, figures, and photographs of the damage types observed in masonry buildings. The typical damage types are illustrated in figure below.

2.2.1 SHEAR/ DIAGONAL CRACKS

Diagonal cracks are typical results of in-plane shear forces. The cracks are caused by horizontal forces in the plane of the wall that produce tensile stresses at an angle of approximately 45 degrees to the horizontal. Such X-shaped cracks occur when the sequence of ground motions generates shear forces that act first in one direction and then in the opposite direction. These cracks often occur in walls or piers between window openings.

The severity of in-plane cracks is judged by the extent of the permanent displacement (offset) that occurs between the adjacent wall sections or blocks after ground shaking ends. More severe damage to the structure may occur when an in-plane horizontal offset occurs in combination with a vertical displacement, that is, when the crack pattern follows a more direct diagonal line and does not “stair-step” along mortar joints. Diagonal shear cracks can cause extensive damage during prolonged ground motions because gravity is constantly working in combination with earthquake forces to exacerbate the damage.

In-plane shear cracking, damage at wall and tie-rod anchorages, and horizontal cracks are relatively low-risk damage types. However, while in-plane shear is not considered hazardous

[3] Adapted from Planning and Engineering Guidelines for the Seismic Retrofitting of Historic Adobe Structures, E. Leroy Tolles, Edna E. Kimbro, William S. Ginell

from the perspective of life safety, it is often costly in terms of loss to historic fabric. In-plane shear cracks often cause severe damage to plasters and stuccos that may be of historic importance, such as those decorated with murals.

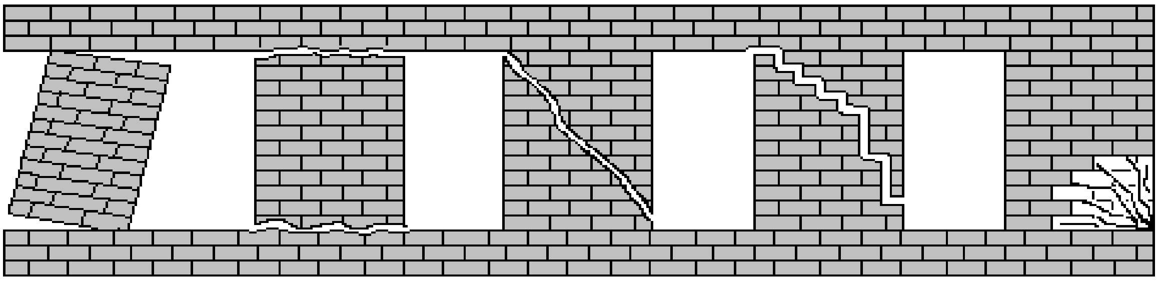
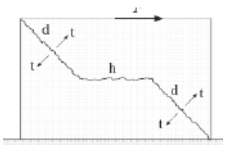
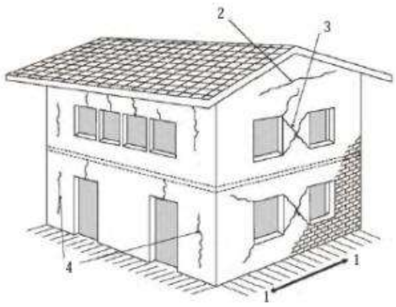


Figure 2-1Different crack patterns of un-reinforced masonry pier

(Source Yi, 2004)





1: Earthquake motion
2: Horizontal crack in gables
3: Diagonal cracks due to shear
4: Cracks due to bending of wall

Cracks and Damages	Remark
 1990 Majalengka Earthquake, Indonesia  1994 Liwa earthquake, Indonesia Damage at corners of openings, Indonesia	
Heavily damaged single storey rubble masonry wall with concrete roof in Manukawa & Sukhpur, India Walls survived due to diaphragm action from roof. Cantilever beams embedded in walls also helped this.   Note: window openings are also not close to corners.	*Ref: Repair and strengthening guide for earthquake damaged low rise domestic buildings in Gujarat, India*

1. Large window openings close to corners and short column failures 2. Diagonal cracking at corner column caused by twisting of frame and short column failure.  	The picture also shows cracks at corner along with diagonal cracks. *Ref: Repair and strengthening guide for earthquake damaged low rise domestic buildings in Gujarat, India*
 Infill panels to an reinforced concrete frame building acting as non-structural shear walls, provided stability to the overall frame – Bharasar, Gujrat	*Ref: Repair and strengthening guide for earthquake damaged low rise domestic buildings in Gujarat, India*
 Uri town – Civil Defence Bldg.– Diagonal cracks in window openings and tilting of walls in Burnt Brick Cement Masonry wall, Kashmir, India.	*Ref: Repair, Restoration and retrofitting of Masonry Buildings in EQ affected Areas of jammu & Kashmir*
 Shear crack in a building during Haiti earthquake *Photo Courtesy :* *Er. Hari Darshan Shrestha*	
  Pakistan earthquake *Photo Courtesy :* *Er. Hari Darshan Shrestha*	

 Diagonal crack seen in a local building in Kathmandu, Nepal *Photo Courtesy : Er.Subin Desar*	

2.2.2 VERTICAL CRACKS

Vertical cracks often develop at corners during the interaction of perpendicular walls and are caused by flexure and tension due to out-of-plane movements. This type of damage can be particularly severe when vertical cracks occur on both faces, allowing collapse of the wall section at the corner.

Cracks and Damages	Remark
   **PADANG , INDONESIA** *Photo Courtesy : Er. Hari Darshan Shrestha*	
 Vertical crack in a building during Haiti earthquake *Photo Courtesy : Er. Hari Darshan Shrestha*	
 Vertical crack in a building during an earthquake, Pakistan *Photo Courtesy : Er. Hari Darshan Shrestha*	

Vertical cracks in old buildings in Kathmandu, Nepal



Photo Courtesy :
Er.Subin Desar

Vertical cracks in old buildings in Kathmandu, Nepal

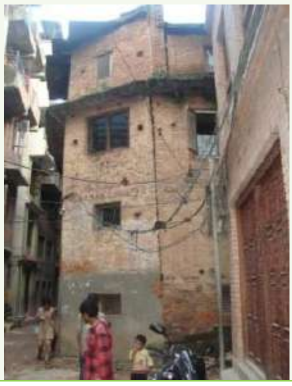


Photo Courtesy :
Ar. Anjali Manandhar

Side wall of a building damaged during Gorkha Earthquake, 2001

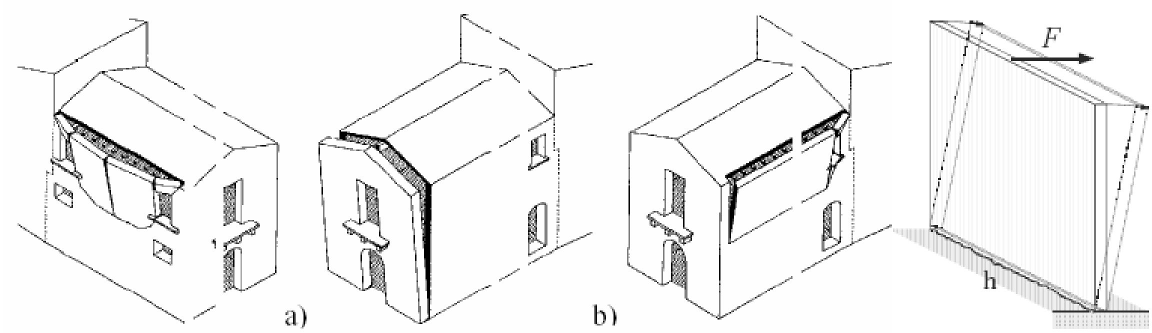
CRACKS AT CORNERS

In-plane shear forces cause diagonal cracks that start at the top of a wall and extend downward to the corner. This type of crack results in a wall section that can move laterally and downward during extended ground motions. Damage of this type is difficult to repair and may require reconstruction.

Damages and Cracks	Remark
   Modern cut-stone masonry building in Mirzapur, Gujrat, India	*Ref: 2. Repair and strengthening guide for earthquake damaged lowrise domestic buildings in Gujarat, India*
 Uri town – Civil Defence Bldg.– Corner cracks and vertical cracks in Burnt Brick Cement Masonry wall, India	*Ref: Repair, Restoration and retrofitting of Masonry Buildings in EQ affected Areas of jammu & Kashmir*
Cracks seen at corners in old buildings in Lalitpur, Nepal   	
  	*Photo Courtesy : Ar. Anjali Manandhar*

2.2.3 OUT OF PLANE / BULGING

Out-of-plane flexural cracking is one of the first crack types to appear in masonry building during a seismic event. Freestanding walls, such as garden walls, are most vulnerable to overturning because there is usually no horizontal support along their length, such as that provided by cross walls or roof or floor systems.



Ref: In-plane stiffness of wooden floor

Damages and Cracks	Remark
  1994 Liwa Earthquake , Indonesia 2006 Central Java Earthquake, Indonesia	
   *(Er. Hari Darshan Shrestha*)	Damage to stone masonry during Northern Pakistan Earthquake
  Partially collapsed buildings showing out of place defect during Kashmir earthquake, India	*Ref: Repair, Restoration and retrofitting of Masonry Buildings in EQ affected Areas of Jammu & Kashmir*

 Partial collapse of gable wall for a single storey random masonry wall in Kera, Gujrat, India Partial collapse of gable wall for a single storey random masonry wall in Kera, Gujrat, India	*Ref: 2. Repair and strengthening guide for earthquake damaged low rise domestic buildings in Gujarat, India*
  	*Ref: In-plane stiffness of wooden floor*
  Out of plane damage during Haiti earthquake *(Hari Darshan Shrestha)*	
  Out of plane damage during Haiti earthquake *(Hari Darshan Shrestha)*	
  *(Subin Desar)*	

 	Damages during Gorkha Earthquake, 2001	

2.2.4 BED JOINT SLIDING

Bed joint sliding is caused by lateral load on the building structure. It results due to the inadequate bed joint bonding strength.

Damages and Cracks	Remark
An example of bed joint can be seen in following pictures of buildings devastated after Haiti earthquake.   *Photo Courtesy :* *Er. Hari Darshan Shrestha*	

2.2.5 TOE CRUSHING

Toe crushing is cased when the lateral load on the building tends to overturn the building structure. It cause load concentration at the toe which crushes the local material at the toe.

Damages and Cracks		REMARK
	Toe crushing defect can be seen in the picture beside, taken after Haiti earthquake. *Photo: Hari Darshan Shrestha*	

	After an earthquake in Pakistan, a building can be seen with cracks and major defects at the toe. *Photo: Hari Darshan Shrestha*	

3. VULNERABILITY ASSESSMENT OF EXISTING BUILDINGS

Vulnerability assessment of buildings can be performed according to methodology prescribed in “SEISMIC VULNERABILITY EVALUATION GUIDELINE FOR PRIVATE AND PUBLIC BUILDINGS” developed by Ministry of Urban Development and Building Construction under Earthquake Risk Reduction and Recovery Preparedness Program for Nepal.

For detailed Evaluation please refer Chapter 4 following this chapter.

4. RETROFITTING CRITERIA

4.1 BUILDING SYSTEM AND GENERAL REQUIREMENTS

Masonry structures gain stability from the support offered by cross walls, floors, roof and other elements such as piers and buttresses Load bearing walls are structurally more efficient when the load is uniformly distributed and the structure is so planned that eccentricity of loading on the members is as small as possible. Avoidance of eccentric loading by providing adequate bearing of floor/roof on the walls providing adequate stiffness in slabs and avoiding fixity at the supports, etc, is especially important in load bearing walls in multistory structures[4]. These matters should receive careful consideration during the planning stage of retrofitting of masonry structures.

4.1.1 BUILDING LIMITATION

Application of this guideline is limited to load bearing masonry buildings which meet the following criteria.

Building type	Maximum Storey height according to seismic zone			Remarks
	A	**B**	**C**	
Masonry with rigid diaphragm	3	3	3	
Masonry with flexible floors	2	3	3	

In case of buildings not meeting the above criteria, the provisions in this guide can be applied but building specific detailed analysis must be carried out.

[4] IS 1905

4.1.2 SEISMIC ZONES

For the purpose of this guideline, the seismic zones are designated as recommended in NBC 109 – 1994 as follows.

Three seismic zones as recommended in NBC 109

Zone	Zone Coefficient	Risk
A	$Z \geq 1.0$	Widespread Collapse and Heavy Damage
B	$0.8 \geq Z > 1.0$	Moderate Damage
C	$Z < 0.8$	Minor Damage

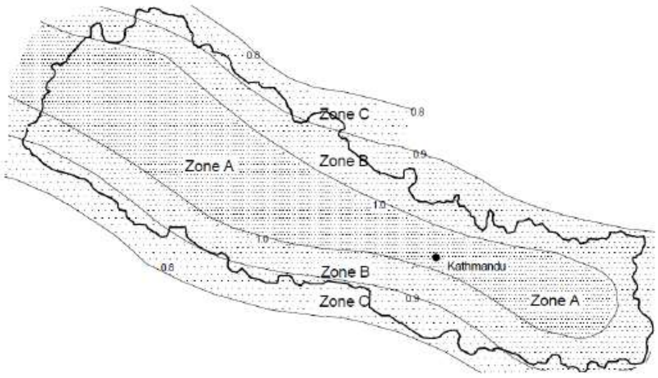


Figure 4-1Seismic zones for masonry structures (NBC 109)

4.1.3 FLOOR SYSTEM

Rigid floor: A floor system that provides lateral as well as rotational restraint (that is, full restraint) to the wall at the floor level. For example, when the floor/roof spans on the walls so that reaction to load of floor/roof is provided by the walls, or when an RCC floor/roof has bearing on the wall (min 9 cm), irrespective of the direction of the span and foundation footings of a wall.

Flexible floor: All floor systems other than defined as rigid floor such as timber floor and roof truss are flexible floors.

4.1.4 LOAD PATH

A continuous load path is most for masonry structures, like all other types of structures, subjected to earthquake loading. Discontinuity in load path in masonry structures arises due to:

a. Lack of redundancy
b. Vertical Irregularities

c. Plan irregularities

a. Lack of redundancy

Lack of redundancy is a condition in which failure of one element in the lateral load resisting of the structures results in complete failure of the structure. Rehabilitation or retrofitting measure requires addition of elements to provide redundancy in the structure. Addition of redundancy is always better than only strengthening of the non-redundant element.

b. Vertical irregularities

Vertical irregularities are discontinuity of lateral force resisting system, weak stories, soft stories, mass and vertical discontinuities. Vertical irregularity should be eliminated as far as practical by providing new vertical lateral-force resisting elements. Vertical irregularities within the limit shown in Figure 2 can be treated by strengthening mechanism at the point of discontinuity or by treating at the element level.

i. Weak storey

It is one in which the storey lateral strength is less than 80 percent of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction.

ii. Soft storey

It is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the stories above.

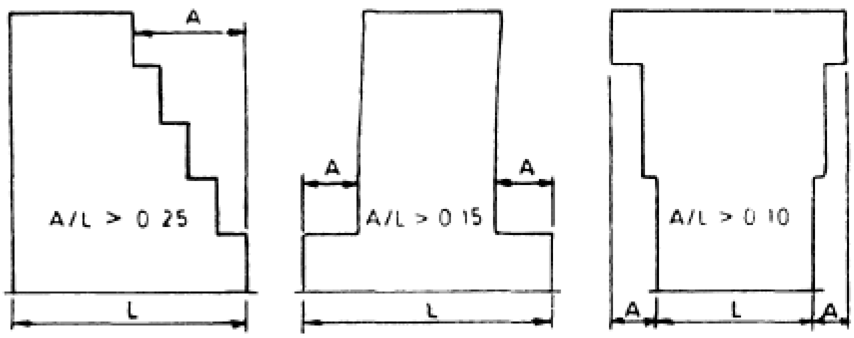


Figure 4-2 Vertical irregularities limit up to which treatment at discontinuity point or at element level is possible.

Plan irregularities*5*

Plan irregularities that create torsion should be eliminated with the addition of lateral-force-resisting bracing elements that will support all major diaphragm segments in a balanced manner. For the irregularities limit specified in Fig. 3, it is possible to allow the irregularity to remain by strengthening those structural components that are overstressed such as the re-entrant corners.

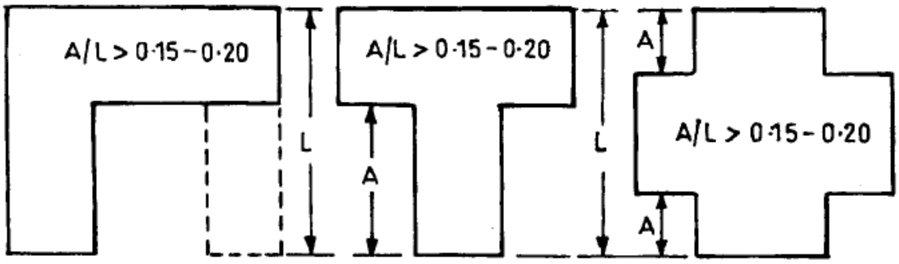


Figure 4-3 Plan irregularity limits that require strengthening of overstressed elements and re-entrant corners

For buildings which exceed irregularity limits specified in Fig. 3, it is recommended that those buildings are either strengthened by adding additional lateral load resisting elements or by separating the different parts of the building (as shown in Fig. 4) as separate system.

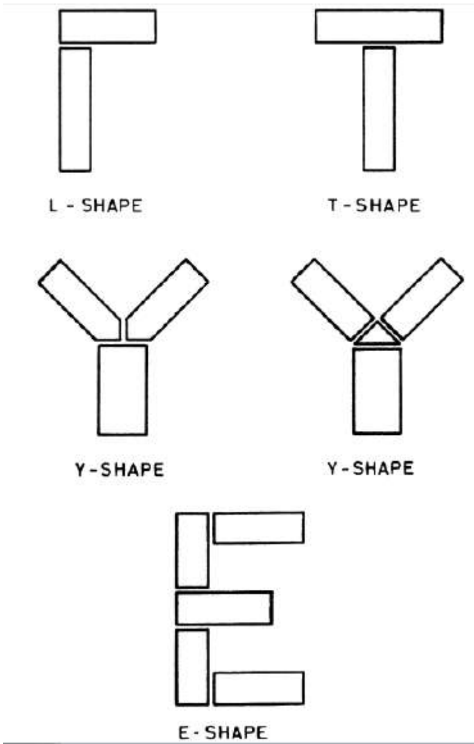


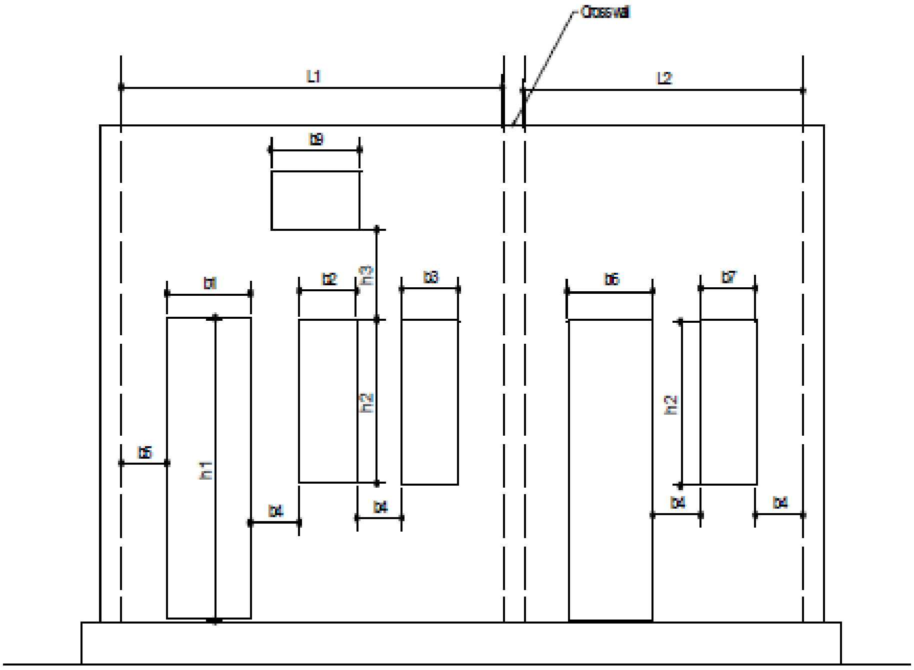
Figure 4-4Recommended shapes for buildings with irregular plans

[5] See the commentary

4.1.3 OPENINGS

4.1.3.1 Openings in walls

Openings in walls should be limited as per the provision of NBC 109 as shown in figure 5 below. In case the openings in the existing structure do not meet the criteria of NBC 109, it is recommended that the openings are either closed or reduced in size in order to meet the recommendations.



Note:

i) $b_1+b_2+b_3$ $<0.5\ L_1$ for one storey, $<0.42\ L_1$ for two-storeyed, $<0.33\ L_1$ for three-storeyed

ii) b_6+b_7 $<0.5\ L_2$ for one storey, $<0.42\ L_1$ for two-storeyed, $<0.33\ L_1$ for three-storeyed

iii) b_4 >0.5 h, but not less than 600 mm

iv) b_5 >0.25 h, but not less than 600 mm

v) b_3 $>$ 600 mm and $>$ 0.5 (bigger of b_2 and b_9)

Figure 4-5Recommended sizes of the openings

4.1.3.2 Opening in diaphragms

Openings in diaphragms (rigid floors) increase shear stresses and induce secondary moments in the diaphragm segments adjacent to the openings[6].

Diaphragm openings immediately adjacent to exterior masonry walls should not be greater than 2.5 m.

The special strengthening measures for diaphragm openings should be as recommended in following section.

4.1.4 HEIGHT TO THICKNESS RATIO

The masonry wall height to thickness ratio should be less than as given in the table below:

Wall type	**Zones**		
	A	**B**	**C**
Top storey of multi-storey building	9	14	14
First storey of multi-storey building	15	16	18
All other conditions	13	16	16

When the above requirements are not satisfied for the existing building, provisions given in section should be adopted.

4.1.5 HEIGHT OF THE WALL

The height should be taken as unsupported height (can be taken as center to center height for slabs) of the wall between floor slabs. The band beams (sill/lintel) are assumed to provide necessary lateral support for the masonry wall in out-of-plane direction if the beams are anchored into the return walls.

In case of walls without overburden loads and with flexible floor/roof not spanning in the out-of-plane direction of the wall, the height should be taken as 1.5 times the unsupported height of the walls.

4.1.6 CROSS WALLS

Cross-walls provision (see Annex 3)

[6] See the commentary

5. ANALYTICAL PROCESS

5.1 ANALYTICAL METHODS

Analysis of the building in the existing condition and including measures of retrofitting should be conducted to determine forces and deformations due to the applied and expected loads. The analysis procedure can be divided into following categories:

1. Simplified linear analysis: A simplified or idealized model of the building and its elements should be prepared along with idealized existing loads and expected loads during the service period of the structure. The simplified model can be of the whole building or of individual elements. In case of individual elements, the idealization should reflect behavior of the global system after assembly. The calculations can be made manually or through help of easily available computer tools without requirement of sophisticated professional structural analysis software.
2. Linear analysis: Linear static analysis of the idealized building in 3-D or 2-D with application of all the loads using software (and with many limitations manually as well in case of very small building). The linear analysis can often be extended to include dynamic effect due to time-history loading. It should be noted that complex modeling of masonry structure to capture real behavior is often a challenging task and the output should always be verified with simplified models.
3. Non-linear analysis: As masonry exhibits non-linear behavior even in small dynamic loading due to appearance of cracks, a detailed non-linear analysis is always preferred. However, the modeling is a complex task requiring sophisticated tools and advance technical know-how which is often not a case. Additionally, there are only few commercially available tools which can capture realistic non-linear behavior of masonry structure.

5.2 CHOICE OF METHOD

Simplified linear analysis: Method 1 can be applied, if all of the following conditions are met:

i. The building doesn't have any irregularity
ii. The building is within the specified limit of height-to-thickness ratio
iii. Opening in the building walls meet the conditions set in NBC 109

5.2.1 Linear analysis:

This method is applicable, if following conditions are met:

- The building's irregularities are within the limit specified in Fig. 2 and 3
- The building is within the specified limit of height-to-thickness ratio.
- Even if the analysis is carried out using computer software, it's recommended that a check should also be performed using idealized model.

5.2.2 Non-linear analysis:

For all other buildings, it is recommended that a detailed non-linear analysis should be performed. It is advisable to check performance of structures specified above using, at least, non-linear static analysis if possible. A schematic outline for static non-linear analysis is given in Annex.

The following units describe analytical approach using idealized model with simplified linear analysis. The calculation approach can also be extended for use in linear analysis.

5.3 HORIZONTAL FORCES

The base shear shall be calculated and distributed at floor level according to NBC 105.
For the purpose of obtaining stiffness of individual elements, the process described in Annex 1 can be used.
The horizontal seismic forces as obtained according to NBC 105 shall be distributed to each wall parallel to the direction of the force according to the following criteria:

5.3.1 FOR RIGID DIAPHRAGM

The distribution to vertical elements will be in proportion to the relative stiffness with respect to each other as shown in Figure below.

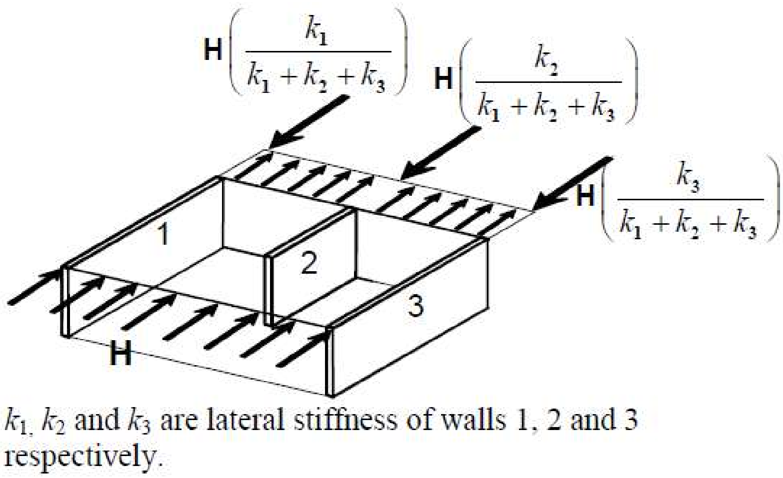


Figure 5-1Proportional distribution of horizontal storey level force to individual walls for rigid diaphragm at floor level

For rigid diaphragm

The distribution to vertical elements will be in proportion to the relative stiffness with respect to the contributing area (tributary area) basis as shown in Figure below.

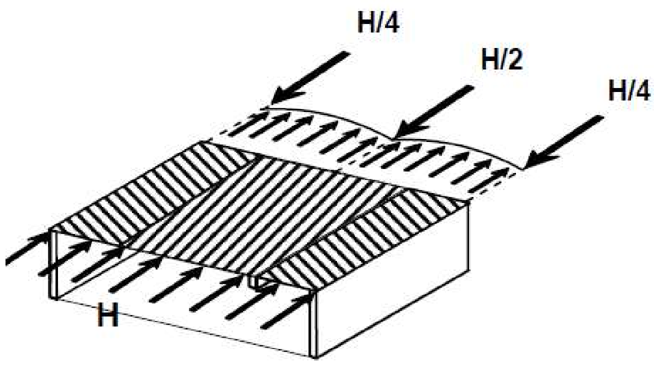


Figure 5-2Proportional distribution of horizontal storey level force to individual walls for flexible diaphragm at floor level

5.4 CHECKS

5.4.1 CHECK FOR SHEAR (IN-PLANE LOADING)

Shear wall strength

i) The shear wall strength shall be calculated as follows:

$V_a = v_a\, Dt$

where:

D = In plane length of masonry wall (mm)

t = thickness of wall (mm)

v_a = permissible masonry shear strength (MPa) given as shown below

$v_a = 0.1v_{te} + 0.15\ (P_{CE}/A_n)$

where:

v_{te} = Average bed-joint shear strength (MPa) determined from in-place shear test and not to exceed 0.6 MPa

P_{CE} = Expected gravity compressive force applied to wall or pier component stress

A_n = Area of net mortared/grouted section (mm^2)

or,

$v_a = 0.1 + (1/6)*(P_{CE}/A_n)$ to a maximum of 0.5 N/mm^2 when in place shear test data is not available

ii) The rocking shear strength shall be calculated as follows:

For walls without openings:

$V_r = (0.50\ P_D + 0.25\ P_w)\ D/H$

For wall with openings:

$V_r = 0.5\ P_D\ (D/H)$

iii) Acceptance criteria for shear walls (in-plane loading)

The acceptability of un-reinforced masonry shear walls shall be determined as follows:

i) When $V_r < V_a$ --> [Rocking controlled mode: When the pier rocking shear capacity is less than the pier shear capacity]

$V_{wx} < \Sigma V_r$

ii) When $V_a < V_r$, [Shear controlled mode: Where the pier shear capacity is less than the pier rocking capacity]

V_{wx} shall be distributed to the individual wall piers, V_p, in proportion to D/H and the following equations shall be met.

$V_p < V_a$

$V_p < V_r$

If $V_p < V_a$ and $V_p > V_r$ for any pier, the pier shall be omitted from the analysis and the procedure shall be repeated using the remaining piers.

5.4.2 CHECK FOR DIAPHRAGM DISPLACEMENT

The deflection in plane of the diaphragm shall not exceed the permissible deflection of attached elements such as walls.

Permissible deflection of diaphragm shall be that deflection which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads without endangering the occupants of the building.

The deflection can be checked using the approach described in Annex 2.

However, if span to width ratio of the diaphragm is less than 6, such deflection checks are not necessary.

5.4.3 ROCKING STRENGTH:

The maximum horizontal shear which can be resisted by a rocking pier failing under static-in-plane is given by equation 4-1 and 4-2[7].

$$V_r = \left(0.50P_D + 0.25P_W\right)\frac{D}{H'} \qquad \text{.................................(4-1)}$$

For walls with openings

$$V_r = 0.5P_D\frac{D}{H'} \qquad \text{……………………..(4-2)}$$

Where, D is the pier width

[7] Seismic Evaluation and Strengthening of Existing Buildings, Dr. Durgesh C. Rai, IIT Kanpur

P_D is Superimposed dead load at the top of the pier under consideration

P_w is weight of wall

H' is least clear height of opening on either side of pier

The detailed calculation is shown in Annex Section 7.1.

6. RETROFITTING OF DIFFERENT ELEMENTS

6.1 GENERAL

This guideline focuses on seismic strengthening (Retrofitting) of unreinforced Masonry Structures. Seismic retrofitting may require intervention at element level but the required performance shall be achieved at global level. The extent of the modifications must be determined by the general principles and design methods stated in earlier chapters, and should not be limited to increasing the strength of members that have been damaged, but should consider the overall behavior of the structure. Commonly, strengthening procedures should aim at one or more of the following objectives:8

i. Increasing the lateral strength in one or both directions, by reinforcement or by increasing wall areas or the number of walls and columns.

ii. Giving unity to the structure by providing a proper connection between its resisting elements, in such a way that inertia forces generated by the vibration of the building can be transmitted to the members that have the ability to resist them. Typical important aspects are the connections between roofs or floors and walls, between intersecting walls and between walls and foundations

iii. Eliminating features that are sources of weakness or that produce concentrations of stresses in some members. Asymmetrical plan distribution of resisting members, abrupt changes of stiffness from one floor to the other, concentration of large masses, large openings in walls without a proper peripheral reinforcement, gable walls are examples of defect of this kind.

iv. Avoiding the possibility of brittle modes of failure by proper reinforcement and connection of resisting members. Since its cost may go to as high as 50 to 60% of the cost of rebuilding, the justification of such strengthening must be fully considered.

[8] Adapted from IAEE Manual

v. Buildings which are symmetrical in plan and regular in elevation are safer than the asymmetrical ones. Thus, effort shall be made to make the buildings symmetrical and regular. The different forms of recommended geometrical configurations are illustrated in Figure 5-1.

vi. Openings in load bearing walls should be restricted as shown in figure 5-2.

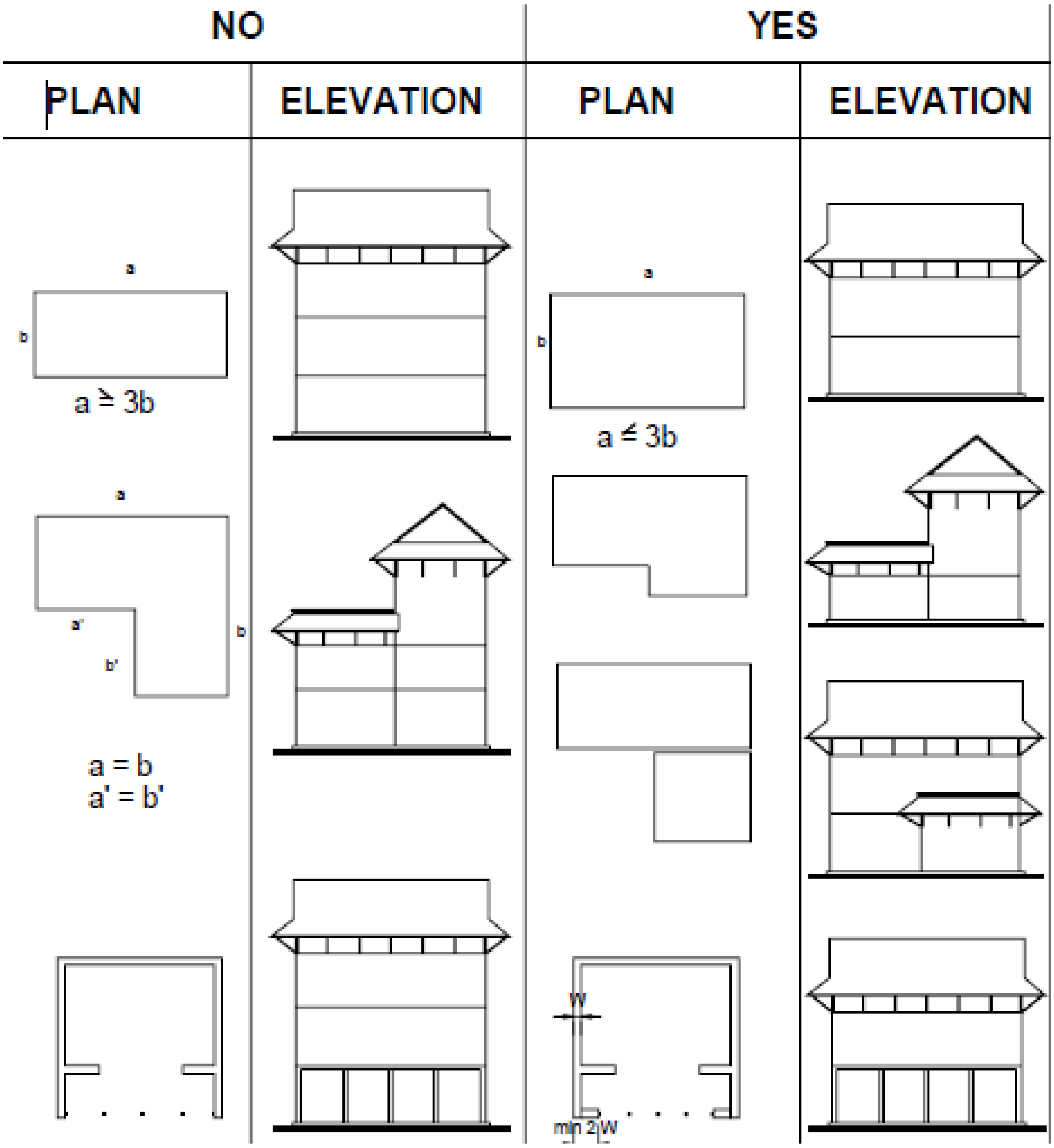
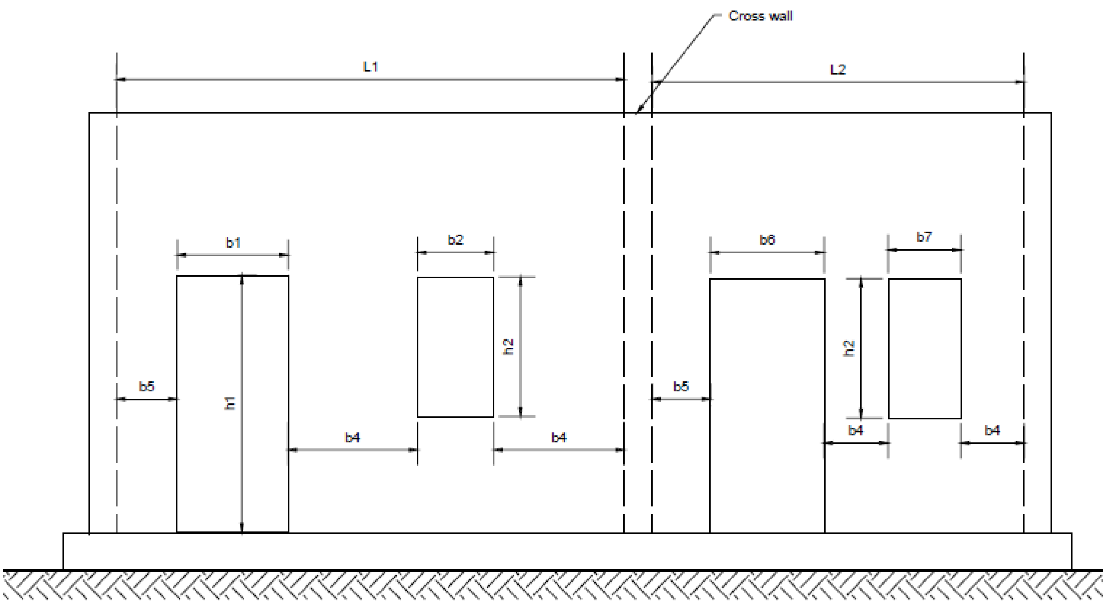


Figure 6-1 Recommended forms of buildings (Adapted from NBC 203)



RECOMMENDATION REGARDING OPENINGS IN LOAD BEARING WALLS

NOTE:

b1 + b2 < 0.3 L1 for one storey, 0.25 L1 for one plus attic storeyed

b6 + b7 < 0.3 L2 for one storey, 0.25 L2 for one plus attic storeyed, three storeyed.

b4 ≥ 0.5 h2 but not less than 600 mm.

b5 ≥ 0.25 h1 but not less than 450 mm.

Figure 6-2 Recommendation regarding openings in load bearing walls

6.2 STRENGTHENING OF FLOOR/ROOF

6.2.1 GENERAL

Load bearing masonry structures should be strengthened in such as a way that the whole building performs as one unit in a box system. The in-plane rigidity provided by floor and roof (diaphragm) is a major factor in order to ensure box-system of the structure.

6.2.2 DIAPHRAGMS

The floor and roof system in a building act as diaphragms which are horizontal elements that transfer earthquake induced inertial forces to vertical elements of the lateral-force-resisting systems i.e. walls.

Diaphragms and their connections to vertical elements providing lateral support shall comply with the following requirements9.

[9] FEMA 356 page 2-21

i. RCC slabs

Masonry walls shall be connected using reinforcement or anchors to the roof and all floors with a connection capable of resisting a seismic lateral force induced by the wall of 1500 N/m. Walls shall be designed to resist bending between connections where the spacing exceeds 1.2 m.

Slabs shall consist of cast-in-place concrete systems that, in addition to supporting gravity loads, transmit inertial loads developed within the structure from one vertical lateral-force-resisting element to another, and provide out-of-plane bracing to other portions of the building.

If the masonry walls are constructed with vertical reinforcement, the vertical bars at corners and junctions of walls shall be taken into the floor slab, roof slab or roof band.

RCC slabs not connected with the masonry walls by continuation of vertical reinforcement shall be anchored with the wall with suitable connection as shown in the figure below.

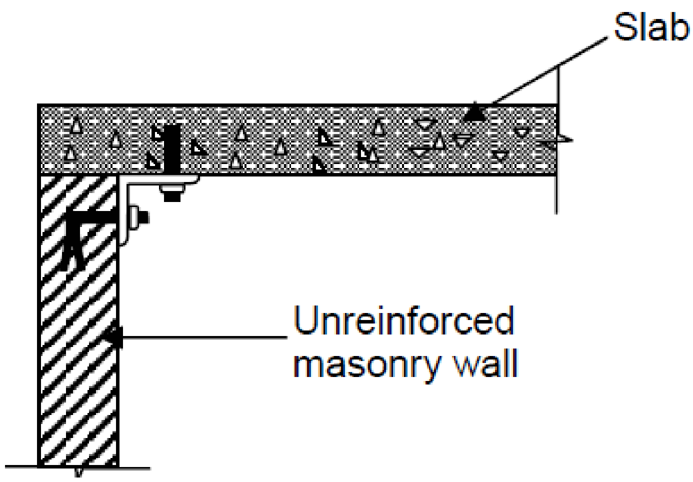


Figure 6-3Anchorage of RCC slab with masonry wall

ii. Timber floors/roofs

Exterior walls should be anchored for out-of-plane forces at each diaphragm level with steel anchors or straps that are developed into the diaphragm.

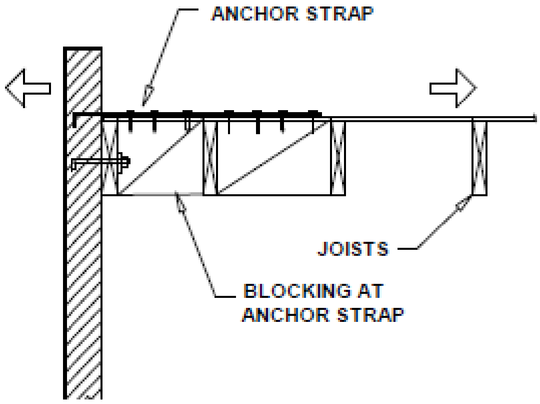


Figure 6-4 Wall anchorage

Wall shall be anchored at the roof and all floor levels at a spacing of equal to or less than 6 foot (1.8 m) center to center. However, anchors shall be provided within 2 feet (0.6m) center to center horizontally from the inside corners of the wall

The connections between the walls and the diaphragm shall not induce cross-grain bending or tension in the wood members (see Figure below). Connections that rely on cross-grain bending in wood members induce tension perpendicular to grain. Failure of such connections is sudden and non-ductile resulting in loss of bearing support and partial or complete collapse of the floors and roof.

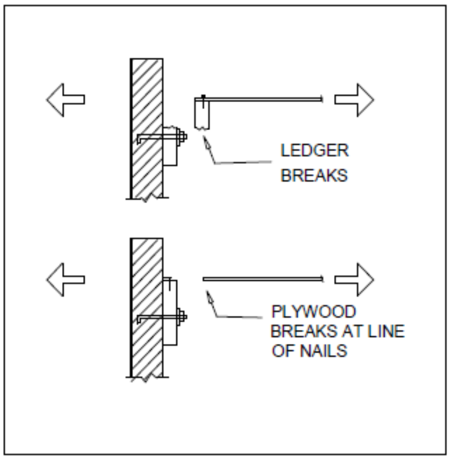


Figure 6-5 Connection relying in cross-grain of timber members induce cross-grain tension causing failure of the connection

Anchors shall be capable of development the maximum of[10]

2.5 S_{D1} times the weight of the wall

3 KN per meter acting normal to the wall at the level of the floor or roof

6.2.3 STIFFENING THE SLOPING ROOF SURFACE[11]

Most of the sloping roof are usually made of rafters, purlins with covering of burnt clay tiles or corrugated galvanized iron (CGI) sheets or asbestos – cement (AC) sheets on top. Sometimes sloping roofs on reinforced concrete slabs are also used. Such roofs push the walls outward during earthquakes. For stiffening such roofs, the rafters should be tied with the seismic belt as in Note 1 below, and the opposite rafters, on both sides of the ridge need to be connected near about mid-height of the roof through cross ties nailed to the rafters (See Figure (5-19). The important point in retrofitting is the provision of seismic belts just below eave level and the gable level.

Note 1:

1. *The mesh should be continuous with 200mm overlap at the corner or elsewhere.*

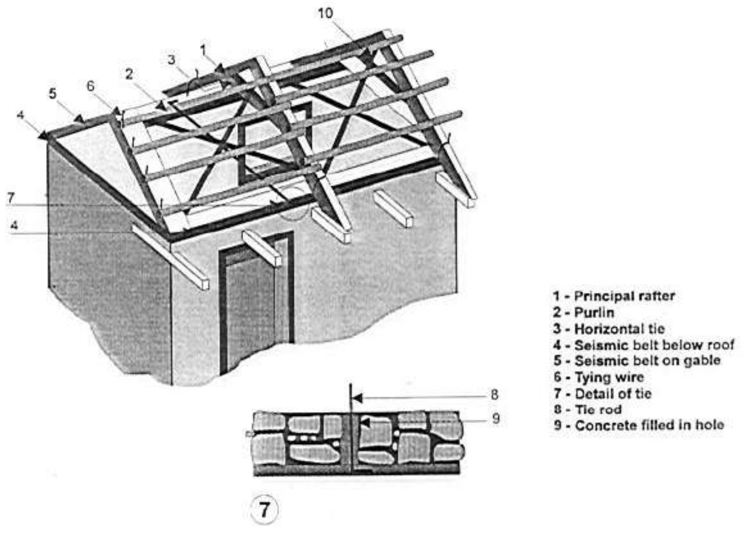


Figure 6-6Stiffening of sloping roof structure

[10] FEMA 310 4.2.6.6

11 GUIDELINES FOR REPAIR, RESTORATION AND SEISMIC RETROFITTING OF MASONRY BUILDINGS, Dr. Anand S. Arya, FNA, FNAE, March 2003

2. *Using galvanized binding wire, tie up the roof rafters with the nails of the eave level belt before applying the plaster over the mesh.*

3. *In brick and Bela stone walls, it will be easy to drill or chisel out holes of 75 mm dia. In that case, instead of the nails, use 3 mm galvanized mild steel wires through the holes to hold and clamp the longitudinal wires every 450 mm c/c.*

6.2.4 REINFORCEMENT AT DIAPHRAGM OPENINGS

There shall be reinforcement around all diaphragm openings greater than 50% of the building width in either major plan dimension as shown below

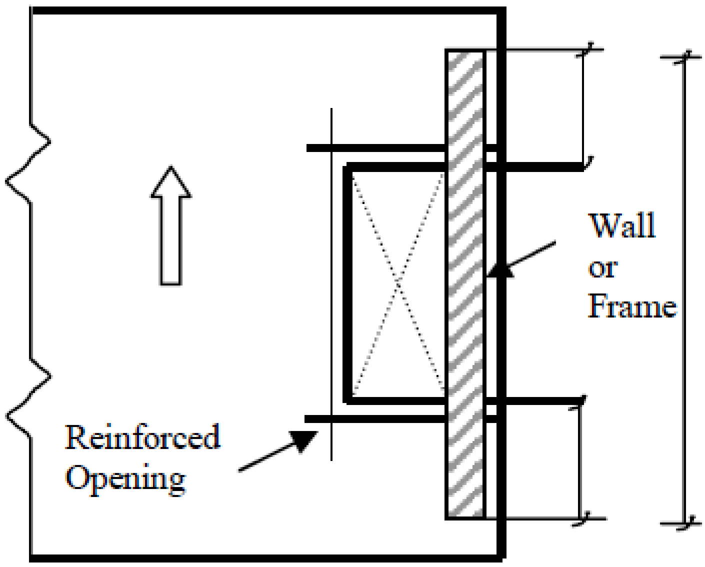


Figure 6-7Opening adjacent to the masonry walls

6.3 STRENGTHENING OF WALL SECTIONS

The walls shall be strengthened for in-plane and out-of-plane loading in order to avoid complete or partial collapse of the walls.

Masonry walls can be reinforced by any of the following or any other suitable measures.

- Steel wire mess with plaster on both faces of the wall
- PP Band with cement or mud plaster on both faces of the wall
- Gabion wire net with or without plaster on both faces of the wall

- The retrofitted walls must be safe against worst combination of lateral forces and designers shall check it before starting the construction.
- A sample calculation for strengthening the wall by using GI wire is shown in Annex 4.
- A step by step approach for application of wire mesh and plaster in masonry building is given in Annex 5.

6.4 WALL OPENINGS

Wall panels with large openings cause the solid wall panels to behave more as frames than as shear walls. Large openings for store fronts and garages, when present, shall be framed by post and beam framing. Lateral force resistance around opening can be provided by steel rigid frames or diagonal bracing.

The openings shall be reinforced by providing a lintel band and vertical reinforcement as shown in following figure.

6.4.1 CONTROL ON DOOR AND WINDOW OPENINGS IN MASONRY WALLS

6.4.1.1 INFILL OPENINGS[12]

A simple method to strengthening a shear wall in-plane is to infill unnecessary window and door openings. This prevents stress concentrations from forming at the corners of openings that initiate cracks. The important thing to consider when infilling an opening is to interlace the new units with the existing or to provide some type of shear connection between the two. This ensures that the existing wall works compositely with the new infill.

*6.4.1.2 SEISMIC BELTS AROUND DOOR / WINDOW OPENING*13

The jambs and piers between window and door openings require vertical reinforcement as in table 5-3:

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[12] A Performance Based Approach to Retrofitting Unreinforced Masonry Structures for Seismic Loads by Keith Bouchard, 2006

[13] GUIDELINES FOR REPAIR, RESTORATION AND RETROFITTING OF MASONRY BUILDINGS IN KACHCHH EARTHQUAKE AFFECTED AREAS OF GUJARAT, GUJARAT STATE DISASTER MANAGEMENT AUTHORITY GOVERNMENT OF GUJARAT, March - 2002

The following mesh reinforcement is recommended to be used for covering the jamb area on both sides of an opening or for covering the pier between the openings.

Table 6-1 Mesh and reinforcement for covering the jamb area

No. of Storeys	**Storeys**	**Reinforcement**		
		Single Bar. mm	**Mesh**	
			N*	**B****
One	One	10	20	500
Two	Top	10	20	500
	Bottom	12	28	700
Three	Top	10	20	500
	Middle	12	28	700
	Bottom	12	28	700

* N = Number of longitudinal wires in the mesh.
**B = Width of the micro concrete belt, half on each all meeting at the corner of T-junction.

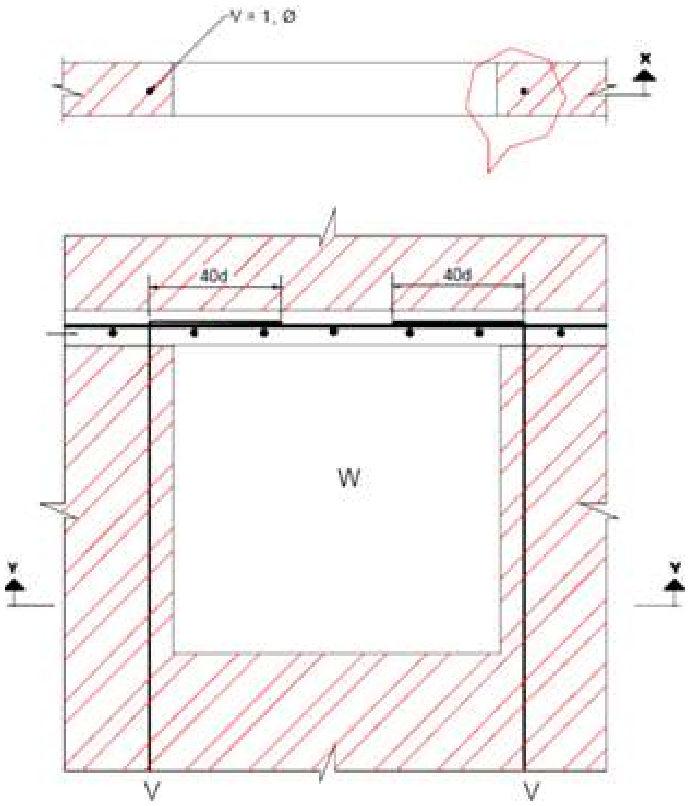

Figure 6-8 Reinforcement around opening in wall (Adapted from NBC 202)

6.5 STIFFENING THE FLAT WOODEN FLOOR / ROOF[14]

Many of the damaged houses have flat floor or roof made of wood logs or timber joists covered with wooden planks and earth. For making such roof/floor rigid, long planks 100mm wide and 25 mm thick should be nailed at both ends of the logs/joists from below. Additionally, similar planks or galvanized metal strips 1.5 mm thick 50 mm wide should be nailed diagonally also. Also see Figure 5-18.

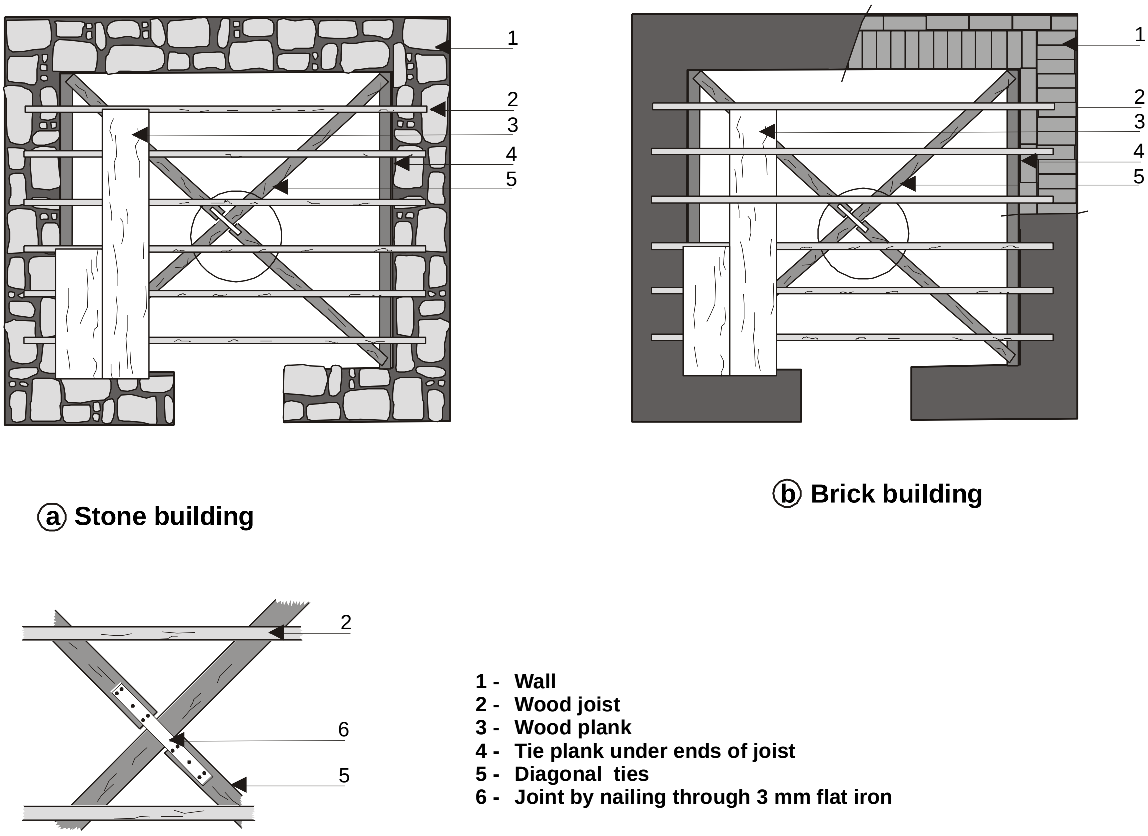

Figure 6-9 Stiffening flat wooden floor / roof

[14] **GUIDELINES FOR REPAIR, RESTORATION AND SEISMIC RETROFITTING OF MASONRY BUILDINGS, Dr. Anand S. Arya,** FNA, FNAE March 2003

6.6 SHEAR WALLS[15]

In this method concrete shear walls are used to retrofit buildings. This method adds significant strength and stiffness to masonry structures. The disadvantages of this method include a considerable increase in the mass of the existing structure and expensive and cumbersome new footings. They can be a major problem on soft soils and in pile-supported structures. The location of new shear walls should be chosen such that they (a) align with the full height of the building, (b) minimize torsion and (c) can be easily incorporated into the existing frame.

Furthermore, the shear walls should be able to maximize the dead weight that can be mobilized to resist overturning uplift.

6.7 STRENGTHENING OF FOUNDATION

The assessment, strengthening/retrofitting/rehabilitation of foundation shall be integral part of the retrofitting and strengthening of an existing building.

The soil condition, condition of existing foundation connectivity of the foundation to the superstructure shall be investigated to ensure that foundation is able to transfer the load safely to the ground.

Foundation rehabilitation schemes shall be evaluated in conjunction with any rehabilitation of the superstructure and according to the general principles and requirements of this standard to assure that the complete rehabilitation achieves the selected building performance level for the selected earthquake hazard level. When new rehabilitation elements are used in conjunction with existing elements, the effects of differential foundation stiffness on the modified structure shall be demonstrated to meet the acceptance criteria16.

In case the foundation is found inadequate, any of the following measures can be adopted

i. Soil material improvements
ii. Shallow foundation improvement techniques
iii. Improvement using deep foundation techniques

A brief guidance is given in Annex

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[15] Canadian Journal on Environmental, Construction and Civil Engineering Vol. 2, No. 8, November 2011, An Investigation into the Interaction of Concrete Shear walls and Masonry Structures in the Seismic Performance of Concrete Shear Walls, M. Kheirollahi, B. Rafezy

[16] FEMA 356

7. ANNEX

EXAMPLE

7.1 STRENGTH BASED ANALYSIS

A fictitious two-storey brick URM building is being used for calculations. The building has 305mm load bearing walls that act as the main lateral force resisting elements. It has a flexible wood diaphragm. Elevation and Plan of the building is shown in figure 7-1. Due to the relative thickness of the spandrel with respect to the piers, this wall is classified as strong spandrel-weak pier (coupled wall). This means that the capacity of the wall will be limited by piers. The flexible diaphragm allows for this wall to be analyzed without considering the other in-plane lateral resisting elements, for example the rear wall. The weight and mass of the building is distributed through tributary area.

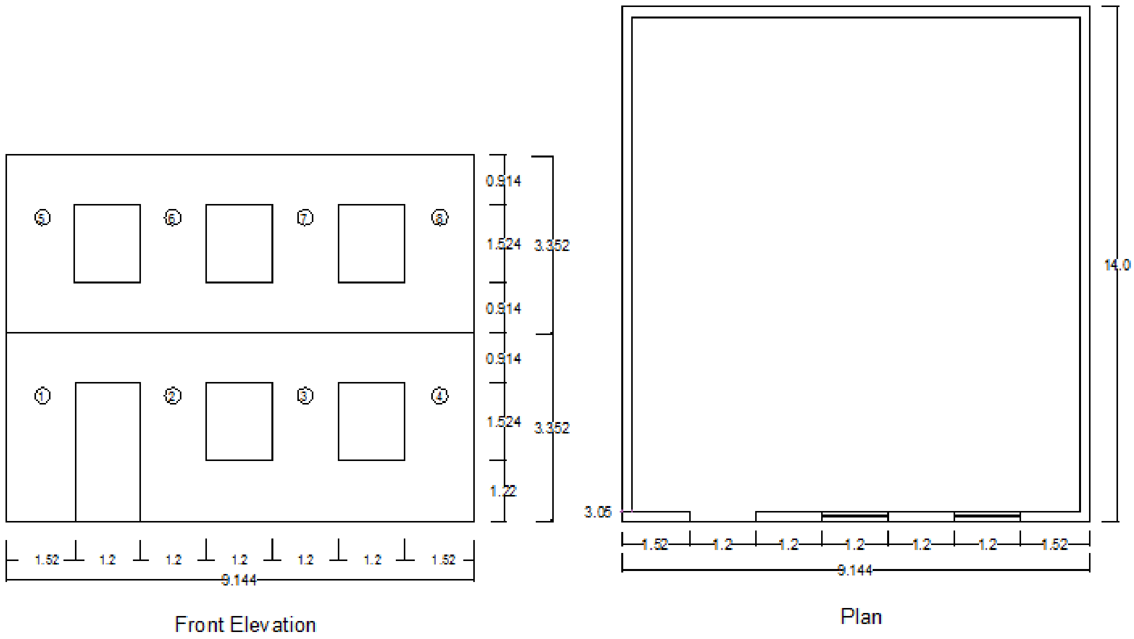


Figure 7-1 Elevation and Plan of a fictitious building

In this analysis the mathematical Formulation is adapted from IITK-GSDMA GUIDELINES for SEISMIC EVALUATION AND STRENGTHENING OF BUILDINGS, Provisions with Commentary and Explanatory Examples, Indian Institute of Technology Kanpur, Gujarat State Disaster Mitigation Authority, August 2005.

1. Detailed Evaluation

1.1 Floor and Roof Dead Loads

Floor:	1.5	KN/m^2
Roofs:	1.2	KN/m^2

Level	Height(m)	Wall thickness(m)
Second	3.65	0.3048
First	3.65	0.3048

1.2 Unit Weight of Walls

Weight of the wall per meter run:

Wall thickness (m)	KN/m^2
0.3048	6.096
0.3048	6.096
0.3048	6.096

Building length =	14	m
Breadth =	9.144	m
Roof live load=	1.5	KN/m^2
Room live load =	2	KN/m^2

1.3 Seismic Weights

Roof dead load =	153.6192	KN
Side walls =	104.051405	KN
End walls =	67.9604318	KN
Openings =	-33.980216	KN
Total seismic weight on roof =	291.650821	KN

First Storey

Floor dead load =	192.024	KN
Side walls =	311.5056	KN
End walls =	203.457658	KN
opening =	-43.041607	KN
Total seismic weight of first storey =	663.945651	KN
Total seismic weight of the building =	955.596471	KN

1.4 Calculation of A_{hm}

Z =	0.36	
I =	1	
Sa/g =	2.5	
R =	1.5	
Ah =	0.3	
U =	0.67	(From Draft code: Sec 5.3)
A_{hm} =	0.201	

1.5 Calculations of DCR Values

1.5.1 Roof Diaphragm

W_d = Roof load + Load on side walls =	257.670605	KN
Vu =	3.6	KN/m
In-plane width dimension of masonry =	9.144	m
Summation VuDd =	65.8368	KN
K =	0.8	(Draft code)
DCR =	2.45834167	

Since this (L , DCR) falls in region 2 of Figure 5 of the draft code, therefore, the roof diaphragm does not need cross walls.

1.5.2 First Storey diaphragm

Wd =	503.5296	KN
Vu =	7.3	KN/m
In plane width dimension of masonry =	10.15	m
Summation VuDd =	148.19	KN
K =	0.8	(Draft code)
DCR =	2.13428389	

Point (L, DCR) falls in region 2 of the figure 5, therefore fourth storey diaphragm does not need cross walls.

1.6 Design Seismic Base Shear

V_B = 192.074891

Table A7.1: Checking Acceptability of Diaphragm Span (Special Procedure

Level	D,m	Wd, KN	DCR	Region from fig 2 of Draft Code
Roof	9.144	257.6706	2.45834167	Region 2 (14, 2.45)
1	10.15	503.5296	2.13428389	Region 2 (14, 2.13)
3	10.15	0	0	Region 2 (24, 3.86)
2	10.04	0	0	Region 2 (24, 4.34)

Wi (KN)	hi (m)	Wi hi^2	Qi, KN	Sum Qi, KN
291.6508207	7.3	15542.07	122.408745	122.408745
663.9456508	3.65	8845.416	69.6661455	192.074891
	Total	24387.49		

Table A7.2 : Checking strength of the Diaphragm

Level	Qi	Sum Qi, KN	w_b, KN	Sum w_i, KN	w_{px}, KN
Roof	122.4087	122.4087	291.650821	291.650821	257.6706

1	69.66615	192.0749		663.945651	955.596471	503.5296

$0.35ZIw_{px}$	$0.75ZIw_{px}$	$\mathbf{F_{px}}$	$\mathbf{F_{px}/w_{px}}$
32.4664962	69.57106	**108.1469**	0.41970993
63.4447296	135.953	**101.2094**	0.201

1.7 Actual h/t Ratio

Top Storey:	11.97507	Do not satisfy
First Storey:	11.97507	Satisfy
Other Storey:	9.84252	Satisfy
	Top storey needs bracing	

1.8 Diaphragm Shear Transfer

1.8.1 Roof Level

Cp =	0.5	
Ahm =	0.201	
Wd =	257.6706	KN
Vu =	3.6	KN/m
Vd =	38.84384	KN
Also Vd =	32.9184	KN
Adapted Vd =	32.9184	KN

1.8.2 First Storey

Cp =	0.5	
Ahm =	0.201	
Wd =	503.5296	KN
Vu =	7.3	KN/m
Vd =	75.90709	KN
Also Vd =	74.095	KN
Adapted Vd =	74.095	KN

1.9 In-plane Shear of masonry walls

1.9.1 Roof Level

Side(Long)
Walls

Wwx =	467.2584	KN
Wd=	257.6706	KN
Fwx =	119.8148	KN
Also Fwx =	144.3189	KN
Adapted Fwx =	119.8148	KN

End(Short)
Walls

Wwx =	16.99011	KN
Wd=	257.6706	KN

Fwx =	29.31091	KN
Also Fwx =	36.33341	
Adapted Fwx =	29.31091	KN

1.9.1 First Level

Side(Long) Walls

Wwx =	155.7528	KN
Wd=	503.5296	KN
Fwx =	81.91104	KN
Also Fwx =	133.5063	KN
Adapted Fwx =	81.91104	KN

End(Short) Walls

Wwx =	80.20803	KN
Wd=	503.5296	KN
Fwx =	66.72654	KN
Also Fwx =	82.87301	KN
Adapted Fwx =	66.72654	KN

Table A7.3: Summary of Storey Shear Force

Wall	Level	Storey Force Fwx, KN	Wall Storey Shear force, Sum Fwx, KN
Side (Long Wall)	Roof	119.8148	119.814834
	1st	81.91104	201.725872
	3rd	0	201.725872
	2nd	0	201.725872
End (Short wall)	Roof	29.31091	29.3109075
	1st	66.72654	96.0374454
	3rd	0	96.0374454
	2nd	0	96.0374454

1.10.Check for inplane shear strength

Table A 7.4: Summary of Pier Analysis

Storey	Pier	Pd, KN	D,m	H,m	t, m
1	5&8	18.9544	1.52	1.2	0.305
	6&7	14.964	1.2	1.2	0.305
Ground	1&4	48.2448	1.52	1.2	0.305
	2&3	38.088	1.2	1.2	0.305

Va, KN	Vr, KN	Mode
27.68	12.00445	Rocking
21.9	7.482	Rocking
35.18	30.55504	Rocking
27.99	19.044	Rocking

7.2 APPROACH FOR NON-LINEAR ANALYSIS

FEMA356 and ATC 40 provide the procedure for non linear static analysis of masonry structure. Non linear static analysis can be performed by using force controlled method or displacement control method. Pushover analysis required determination of three primary element: capacity, demand (displacement) and performance.

Capacity: The overall capacity of the structure depends on the strength and deformation capacities of the individual components. The pushover capacity curves approximate how structures behave after exceeding their capacity.

Demand: For a given structure and ground motion, the displacement demand is an estimation of the maximum expected response of the building during ground motion.

Performance: Performance verifies that, the structural and non-structural components are not damage beyond the acceptable limit of the performance objective for the force and displacement, implied by the displacement.

Pushover Curve: Non-linear static procedure develops a pushover curve, which is the relationship between the base shear and lateral displacement of the control node. The pushover curve is developed by first applying gravity loads, followed by monotonically increasing lateral forces with specified height wise distribution. Generalized force deformation curve for masonry given in FEMA356 and acceptance criteria is shown below.

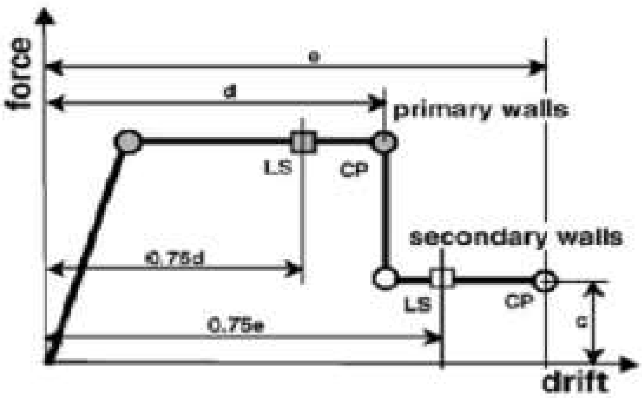


Figure 2.6 Generalized force deformation relation for masonry element and component (FEMA356)

Table 2-1 Acceptance criteria for nonlinear static analysis of masonry (FEMA356)

Behavioral Mode	c%	d%	e%	Performance Level				
				Primary			Secondary	
				IO %	LS %	CP %	LS %	CP %
Bed-Joint Sliding	0.6	0.4	0.8	0.1	0.3	0.4	0.6	0.8
Rocking	0.6	$0.4h_{eff}/L$	$0.8h_{eff}/L$	0.1	$0.3h_{eff}/L$	$0.4h_{eff}/L$	$0.6h_{eff}/L$	$0.8h_{eff}/L$

IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention

Table 2-2 Structural performance level for unreinforced masonry building as define in FEMA356

Description	Type	Structural Performance Level		
		Collapse Prevention	Life Safety	Immediate Occupancy
Unreinforced masonry wall	Primary	Extensive cracking; face course and veneer may peel off. Noticeable in plane and out of plane offset.	Extensive cracking; Noticeable in plane offset of masonry and minor out of plane offset.	Minor (<1/8" width) cracking of veneers. Minor spalling in veneers at a few corner opening. No observable out of plane offsets.
	Drift ratio	1%	0.6%	0.3%

7.3 CALCULATION OF STIFFNESS OF MASONRY WALLS[17]

The lateral stiffness of masonry walls subjected to lateral in-plane forces shall be determined considering both flexural and shear deformations.

The masonry assemblage of units, mortar, and grout shall be considered to be a homogeneous medium for stiffness computations with an expected elastic modulus in compression as given in the table below18.

	Masonry Condition[1]		
Property	**Good**	**Fair**	**Poor**
Compressive Strength (f'_m)	900 psi	600 psi	300 psi
Elastic Modulus in Compression	$550f'_m$	$550f'_m$	$550f'_m$
Flexural Tensile Strength[2]	20 psi	10 psi	0
Shear Strength[3]			
Masonry with a running bond lay-up	27 psi	20 psi	13 psi
Fully grouted masonry with a lay-up other than running bond	27 psi	20 psi	13 psi
Partially grouted or ungrouted masonry with a lay-up other than running bond	11 psi	8 psi	5 psi

1. Masonry condition shall be classified as good, fair, or poor as defined in this standard.

Table 7-2 Factors to Translate Lower-Bound Masonry Properties to Expected Strength Masonry Properties[1]

Property	Factor
Compressive Strength (f_{me})	1.3
Elastic Modulus in Compression[2]	–
Flexural Tensile Strength	1.3
Shear Strength	1.3

1. See Chapter 6 for properties of reinforcing steel.
2. The expected elastic modulus in compression shall be taken as $550f_{me}$, where f_{me} is the expected masonry compressive strength.

For linear procedures, the stiffness of a URM wall or pier resisting lateral forces parallel to its plane shall be considered to be linear and proportional with the geometrical properties of the uncracked section excluding claddings.

Storey shears in walls with openings shall be distributed to piers in proportion to relative lateral un-cracked stiffness of each pier (see Fig. below).

[17] FEMA 356

[18] Adapted from FEMA 356

Laboratory tests of solid shear walls have shown that behavior can be depicted at low force levels using conventional principles of mechanics for homogeneous materials. In such cases, the lateral in-plane stiffness of a solid cantilevered shear wall, k, can be calculated using Equation (C7-1):

$$k = \frac{1}{\dfrac{h_{eff}^3}{3E_m I_g} + \dfrac{h_{eff}}{A_v G_m}} \qquad \text{(C7-1)}$$

where:

h_{eff} = Wall height

A_v = Shear area

I_g = Moment of inertia for the gross section representing uncracked behavior

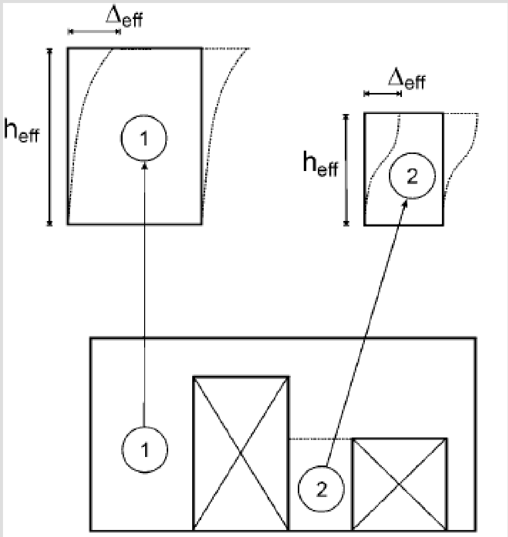
E_m = Masonry elastic modulus

G_m = Masonry shear modulus

Correspondingly, the lateral in-plane stiffness of a pier between openings with full restraint against rotation at its top and bottom can be calculated using Equation (C7-2):

$$k = \frac{1}{\dfrac{h_{eff}^3}{12E_m I_g} + \dfrac{h_{eff}}{A_v G_m}} \qquad \text{(C7-2)}$$

The design professional should be aware that a completely fixed condition is often not present in actual buildings.



h_{eff} = The effective height of the component under consideration

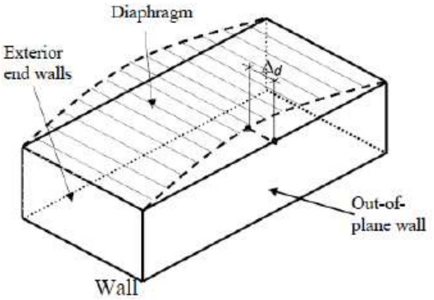
Δ_{eff} = The differential displacement between the top and bottom of the component

Depending on the wall and pier geometry, the elevations at which these parameters are defined may vary in the same wall assembly.

7.4 DEFLECTION CHECK OF DIAPHRAGM[19]

(a) Calculation of Diaphragm Deflection

The deflection of diaphragms as shown in figure C34 (a), should be determined by an adequate engineering analysis. However, it is realized that the calculation of diaphragm deflection is quite complex and also imprecise for various types of prevalent diaphragm construction. It is therefore necessary that a proper care is exercised in the choice of analysis method.

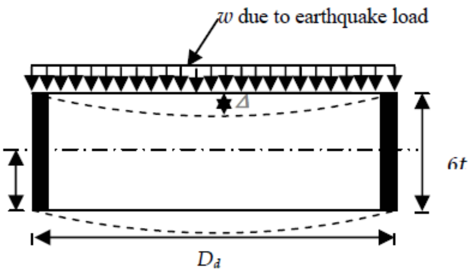


The behaviour of a flexible diaphragm under lateral load can be approximated as that of a wic flanged I beam with a large depth. The web of tl I beam resists shear while the flange contributes resisting moment generated by the uniform later load.

With this I beam approximation, deflection Δ_d o diaphragm of span, *L*, due to uniform lateral loa as shown in figure C34 (b) is given by equation 7.1

$$\Delta_d = \frac{5wL^4}{384EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(7.1)$$

For use in equation 7.1, the dimension of the diaphragm along the direction of lateral load is taken as the depth, D_d, of the I beam and the width of the flange is equal to six times the thickness of the supporting wall including dept of the diaphragm, measured equally above and below the centre of the diaphragm as shown in Figure C34(c).



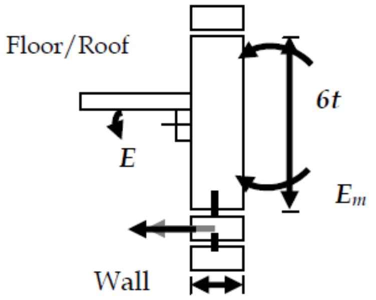


Figure C34(c): Diaphragm Deflection-I beam approximation

The moment of inertia of this beam about an axis perpendicular to the diaphragm is computed using equation 7.2.

$$I = n\frac{dD_d^3}{12} + \sum A\left(\frac{D_d}{2}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(7.2)$$

$$n = \frac{E_d}{E_m} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(7.3)$$

where Ed and Em are modulus of elasticity of diaphragm and masonry materials respectively

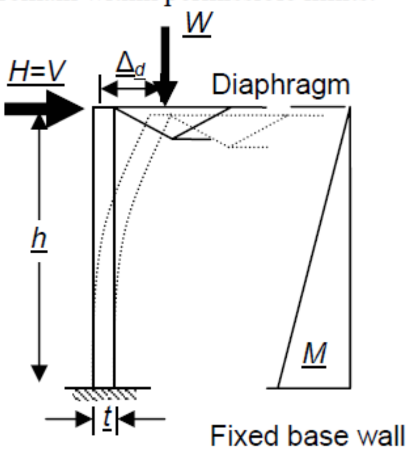
(b) Control of Diaphragm Deflection

As shown in Figure C34 (a) the in-plane deflection of a diaphragm due to seismic inertia forces causes out-of-plane deflection of some walls. The out-of-plane walls are thus subjected to flexural stresses in addition to stresses due to vertical loads; Excessive deflection of a diaphragm can seriously undermine the load carrying capacity of out-of-plane walls. The magnitude of diaphragm deflection should be limited so that walls are not subjected to extreme and damaging deflections.

One method to ensure that in-plane deflections of diaphragm are acceptable is by checking that flexural stresses so induced in the walls are within the permissible limits specified for the masonry as per IS 1905.

[19] Adapted from IITK guideline (Rai, 2005)

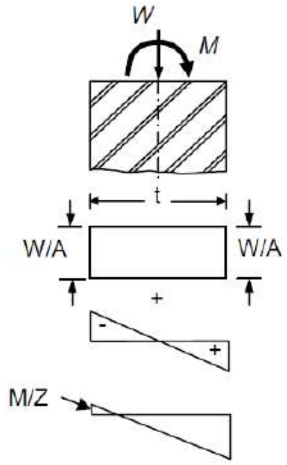
Top of the wall undergo lateral deflection Δ_d with the diaphragm which is the sum of the deflections due to bending moment (Δ_b) and deflections due to shear (Δ_v). For one storey building the deflection is caused primarily by bending. This deflection is caused by shear force at the level of diaphragm, which also generates a linearly varying bending moment up the height of the wall. To prevent masonry from developing tensile cracks and thereby making it unstable it is necessary that resultant stress remain within permissible limits.



Moment at the base of wall is calculated using equation

$$M = Vh = \frac{3E_m I \Delta_d}{h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(7.4)$$

and the resulting stresses are shown in Figure C39



Hence,

$$\frac{M}{Z} - \frac{W}{A} = f_t \qquad \text{(Tensile)}$$

$$-\left(\frac{M}{Z} + \frac{W}{A}\right) = fc \qquad \text{(Compressi}$$

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

$$\frac{3EI}{h^2}\left(\frac{\Delta_d t}{2I}\right) = \left(f_t + \frac{W}{A}\right)$$

$$\Delta_d = \frac{2h^2}{3Et}\left(f_t + \frac{W}{A}\right)$$

Figure C36: Stresses at the base of wall

Taking allowable values of stresses in masonry, permissible deflection of diaphragm (wall) can be obtained as shown in figure C36. The net diaphragm deflection should be less than or equal to the permissible wall deflection.

Deflections can be controlled by controlling (h/t) ratio of the walls.

7.5 ADDITION OF ELEMENTS

i. Addition of cross wall

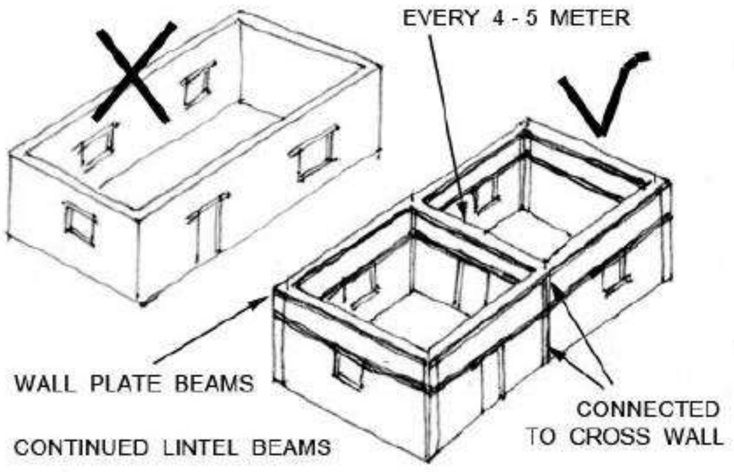


Figure 7-2 Addition of cross-wall (Advies, 2012)

7.6 ALTERNATE APPROACH FOR ROOF REPLACEMENT/ STRENGTHENING /RETROFITTING

While repairing and retrofitting the house, if it is desired to replace the roof structure or tiles with AC or CGI roof, take the following steps:-

i) Complete the repair and retrofitting work of the first storey including provision of 'through' elements.

ii) Stiffen the first floor wooden deck.

iii) Complete the horizontal and vertical seismic belts in the first storey, keep the vertical mesh reinforcement extending beyond the first storey by 300mm and leave uncovered.

iv) Complete the repair and retrofitting of the second storey.

v) Now open the roof structure and remove gable portion up to eave level.

vi) It will be preferable to use seismic bands, instead of belts in this case. Therefore, construct the vertical seismic belts and the eave level and gable bands together, taking the vertical steel in the bands. Anchor steel wires in the bands and extend out for tying down the rafters and purlins.

vii) Now erect the rafters, tie them with bands and fix the tie to make A-frames.

viii) Install diagonal bracing in the plane of the rafters.

ix) Erect the purlins over the rafters, tie them with rafters and gable bands. Bolt down the AC or CGI sheets to the purlins using J or U bolts with iron and bitumen

7.7 STRENGTHENING OF WALLS USING GI WIRE (SAMPLE CALCULATION)

Gabion wire to strengthen masonry walls

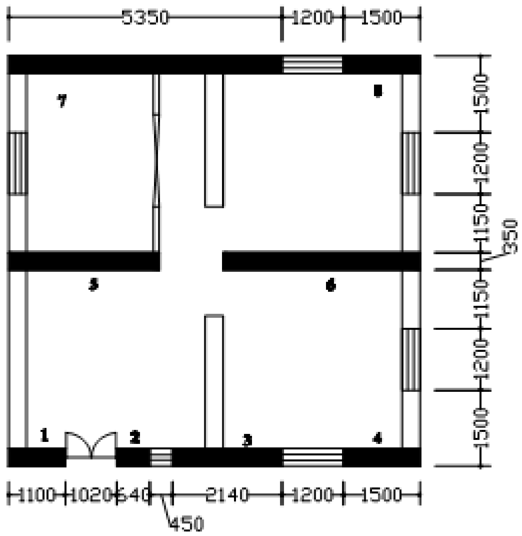


Figure 7-3 : A sample masonry building

For a two storey building

Assume base shear	:	500 KN
Storey level shear, 1st storey	:	300 KN
2nd Storey	:	200 KN

If the outer wall is supposed to carry 1/3rd of the 1st storey shear then shear in the wall is 100 KN.

If the shear strength of the wall is assumed to be zero, the shear force has to be taken by GI wire placed on both sides of the wall.

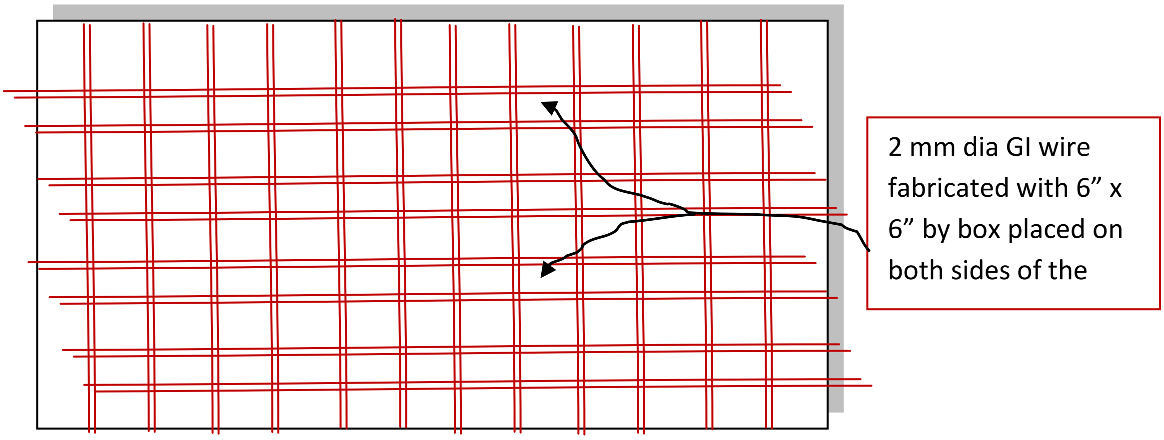
Supposing 2.0 mm GI wire with x-sectional area of 3.1 mm^2 with 0.45KN/mm^2 strength[20], total number of wires required

$= \frac{100}{3.1\ x\ 0.45*0.75}$ [25% strength reduction for knotting of the GI wire]

[20] British standard wire gauge

= 100 wires to be placed equally on both faces of the wall i.e. 50 wires are required in each face of the wall which has to be placed in the length of 8000 mm

= 6” x 6” GI wire mesh on both sides will provide sufficient shear capacity for the wall



7.8 DIFFERENT TECHNIQUES FOR STRENGTHENING OF WALLS

7.8.1 APPLICATION OF STEEL WIRE MESH/FERRO-CEMENT PLATING IN MASONRY BUILDING (ADAPTED FROM A. S. ARYA)

To strengthen a half brick thick load bearing wall

The welded wire mesh may be of 14 gauge wires @ 35 to 40 mm apart both ways. Provision of mesh on external or internal faces with an overlap of 30 cm at the corners will suffice for upto 3 m long walls. For longer walls, ferro-cement plating be provided on both faces.

DETAILS OF RETROFITTING ELEMENTS

Ferro-Cement Plating:-

It consists of a *galvanized iron mesh* fixed to the walls through nails or connector-links drilled through the wall thickness and the mesh is covered by rich mix of cement-sand mortar in the ratio of 1:3. To achieve good results, the following step-wise procedure is to be followed:-

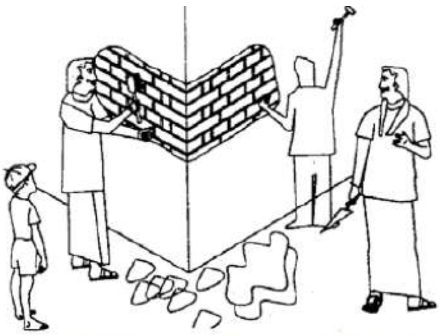


Fig.1:- Removal of Plaster & Raking of Joints

(i) Mark the height or width of the desired plating based on the weld mesh number of longitudinal wires and the mesh size (see table 1).

(ii) Cut the existing plaster at the edge by a mechanical cutter for neatness, and remove the plaster (see fig.1).

(iii) Rake the exposed joints to a depth of 20 mm. Clean the joints with water jet.

(iv) Apply neat cement slurry and plaster the wall with 1:3 cement – coarse sand mix by filling all raked joints fully and covering the wall with a thickness of 15 mm. Make the surface rough for better bond with the second layer of plaster.

(v) Fix the mesh to the plastered surface through 15 cm long nails driven into the wall at a spacing of 45 cm tying the mesh to the nails by binding wire (see fig.2).

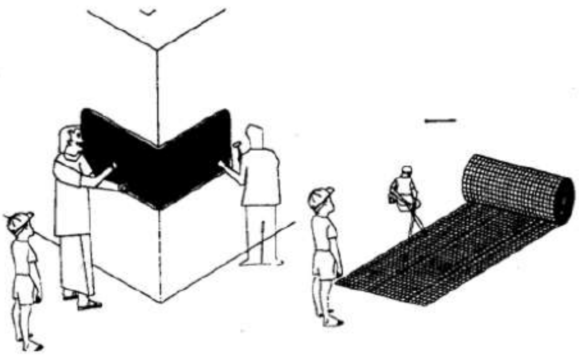


Fig. 2 :- Galvanised steel wire mesh

(vi) Now apply the second layer of plaster with a thickness of 15 mm above the mesh. Good bonding will be achieved with the first layer of plaster and mesh if neat cement slurry is applied by a brush to the wall and the mesh just in advance of the second layer of plaster.

7.8.2 GROUTING

Grouting is defined as the injection of fluid mortars or adhesives to fill discontinuities and cracks and reintegrate detached wall sections, is seen as a more promising solution to the problem21.

21 Interdisciplinary Experts Meeting on Grouting Repairs for Large-scale Structural Cracks in Historic Earthen Buildings in Seismic Areas, *The Getty Conservation Institute Pontificia Universidad Católica del Perú August 13-16, 2007*

Many voids exist in masonry walls; hence an appropriate method for strengthening the walls is by filling these voids by injecting cementitious grout. After hardening, the injected grout will bond the loose parts of the wall together into a solid structure.

7.8.2 METHODOLOGY FOR GROUTING OF CRACKS [22]

7.8.2.1 Minor and medium cracks (crack width 0.5 mm to 5.0mm)

Material / Equipment required

i) Plastic/ Aluminum nipples of 12mm dia.(30 to 40 mm long)

ii) Non-shrink cement (shrinkomp of ACC or equivalent).

iii) Polyester putty of 1:3 cement sand mortar for sealing of the cracks.

iv) Compressor for injecting the slurry.

Procedure:- See Figure 5 - 3

Step-1 Remove the plaster in the vicinity of crack exposing the cracked bare masonry.

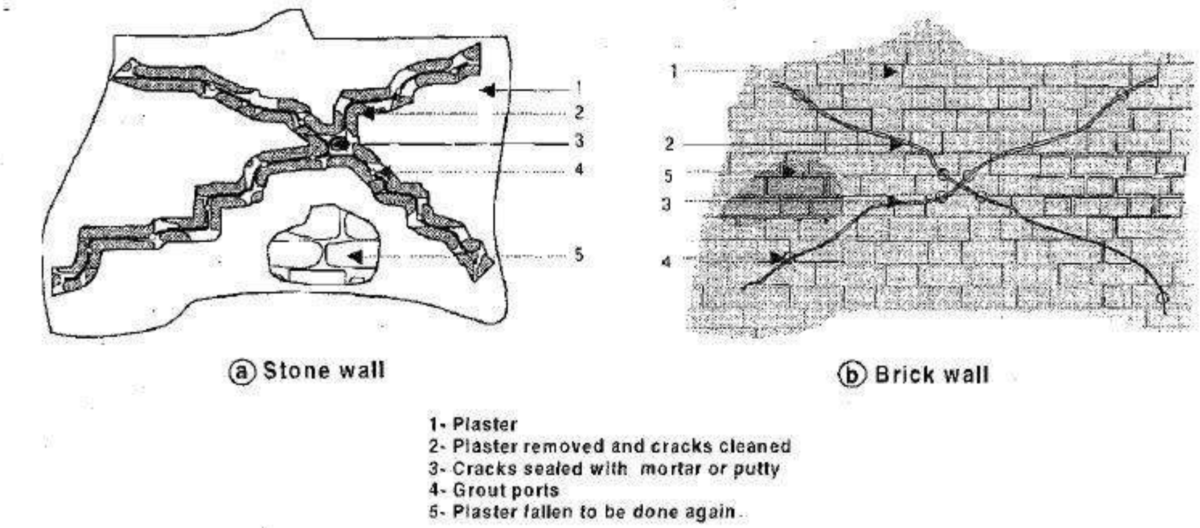


Figure 7-4 Filling grout in cracks

Step-2 Make the shape of crack in the V-shape by chiseling out.

Step-3 Fix the grouting nipples in the V-groove on the faces of the wall at spacing of 150-200 mm c/c.

22 GUIDELINES FOR REPAIR, RESTORATION AND RETROFITTING OF MASONRY BUILDINGS IN KACHCHH EARTHQUAKE AFFECTED AREAS OF GUJARAT, GUJARAT STATE DISASTER MANAGEMENT AUTHORITY GOVERNMENT OF GUJARAT March - 2002

Step-4 Clean the crack with the Compressed air through nipples to ensure that the fine and loose material inside the cracked masonry has been removed.

Step-5 Seal the crack on both faces of the wall with polyester putty or cement mortar 1:3(1-cement: 3-coarse sand) and allowed to gain strength.

Step-6 Inject water starting with nipple fixed at higher level and moving down so that the dust inside the cracks is washed off and masonry saturated with water.

Step-7 Make cement slurry with 1:1(1-non shrink cement:1-water) and start injecting from lower most nipple till the cement slurry comes out from the next higher nipple and then move to next higher nipple.

Step-8 After injection grouting through all the nipples is completed, replaster the finish the same.

7.8.2.2 Major Crack (Crack width more than 5.0mm)

Material / equipment required

i) Plastic/ Aluminum nipples of 12 mm dia. (30 to 40 mm long).

ii) Polyester putty of 1:3 cement-sand mortar for sealing of cracks.

iii) Non-Shrink cement (shrinkomp of ACC or equivalent).

iv) Compressor for injecting the slurry.

v) Galvanized steel wire fabric (16 to 14 gauge i.e. 1.5 to 2.03 mm dia. wire) with 25 mm x 25mm.

vi) Galvanized steel clamping rod of 3.15 mm dia, or 5 mm dia 150 mm long wire nails.

Procedure:-

Step-1 Remove the plaster in the vicinity of crack exposing the cracked bare masonry.

Step-2 Make the shape of crack in the V-Shape by chiseling out.

Step-3 Clean the crack with compressed air.

Step-4 Fix the grouting nipples in the V-groove in both faces of the wall at spacing of 150-200 mm c/c.

Step-5 Clean the crack with the compressed air through nipples to ensure that the fine and loose material inside the cracked masonry has been removed.

Step-6 Seal the crack on both the faces of the wall with polyester putty or cement mortar 1:3 (1-cement: 3-coarse sand) and allowed to gain strength.

Step-7 Inject water starting with nipples fixed at higher level and moving down so that the dust inside the crack is washed off and masonry is saturated with water.

Step-8 Make cement slurry with 1:2:W (1-non shrink cement: 2- fine sand: just enough water) and start injecting from lower most nipple till the slurry comes out from the next higher nipple and then move to next higher nipple.

Step-9 After injection grouting through all the nipples is completed, replaster the surface and finish the same.

Alternative Procedure: See Figure 5-4

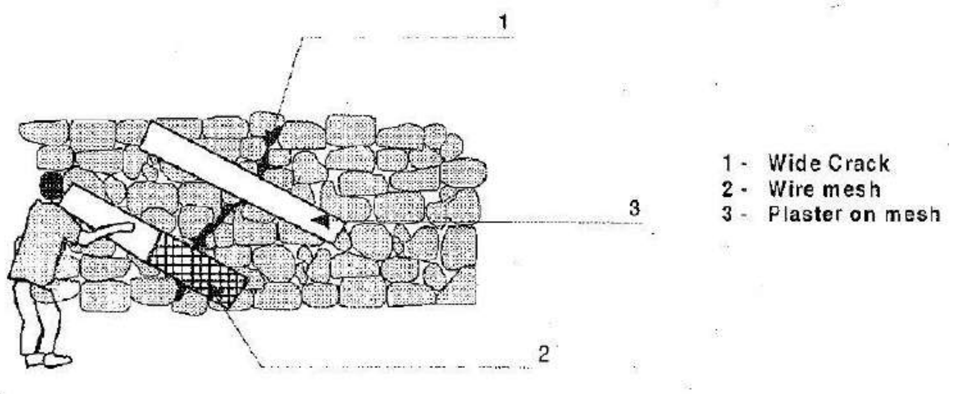


Figure 7-5 Fixing mesh across wide cracks

Step-1 Remove the plaster in the vicinity of crack exposing the cracked bare masonry.

Step-2 Make the shape of crack in the V-shape by chiseling out.

Step-3 Clean the crack with compressed air.

Step-4 Fill the crack with cement mortar 1:3 (1-non shrink cement: 3-fine sand: necessary water) from both sides as deed as feasible.

Step-5 Provide wire mesh on both the faces of wall after removal of plaster in the region of repair to a width of 150mm on each side of the crack.

Step-6 Clamp the mesh with the wall using clamps or wire nails at the spacing of 300mm c/c.

Step-7 Plaster the meshed area with cement sand mortar of 1:3, covering the mesh by a minimum of 12mm.

7.8.3 JACKETING

Jacketing consists of covering the wall surface with a thin layer of reinforced mortar, micro-concrete, or shotcrete overlays interconnected by means of through-wall anchors[23]. One of the most used traditional techniques for strengthening existing residential masonry Buildings, by improving its lateral resistance and energy dissipation capacity of the system is application of reinforced-cement coating (jacket) on one or both sides of the walls (Fig.5-5)[24].

The method of jacketing can be summarized as below:

1. Remove the existing plaster from the wall and in the joints between the bricks or blocks, 10 to 15mm deep, and grouting of the cracks in the wall.
2. The wall surface is cleaned, water moistened and spattered with cement milk.
3. The welded steel reinforcement mesh with 4-6 mm bars at 100-150 mm intervals in vertical and horizontal directions is placed in between two cement coatings with 10-15 mm thick cement mortar layer with compressive strength of 20-30 MPa.

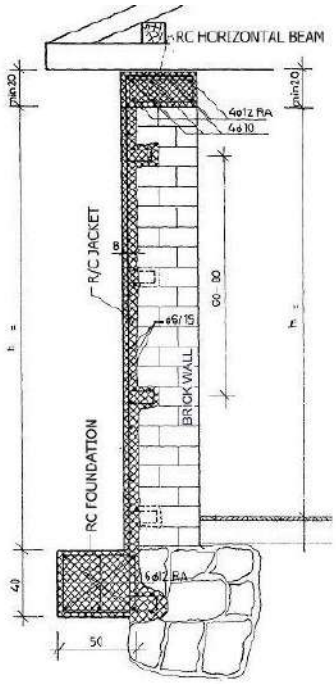


Figure 7-6 Application of RC coating

23 A TUTORIAL: Improving the Seismic Performance of Stone Masonry Buildings, Jitendra Bothara • Svetlana Brzev, First Edition, April 2011

24 Experimental and Analytical Research of Strengthening Techniques for Masonry, Sergey Churilov, Elena Dumova-Jovanoska

4. The wire mesh is connected with steel anchors 6mm diameter bars placed in pre-drilled holes and cement of epoxied on the wall surface, with quantity of 4-6 pieces per m^2.

5. Except for connecting and securing the coating reinforcement to the existing masonry with steel anchors, the connection can be achieved by inserting shear connectors from cage reinforcement. The total thickness of the jacket should not exceed 30 mm.

6. Ideally, jacketing should be applied to both interior and exterior wall surfaces, but this may not always be possible due to functional or financial constraints. In the case of a single- surface application, steel dowels of adequate size and spacing should be provided to ensure that the existing stone wall and the new jacket act in unison (Figure 5-6). However, it should be noted that the effectiveness of single-surface jacketing is significantly inferior to double-sided application because a single-sided jacket cannot confine the wall.

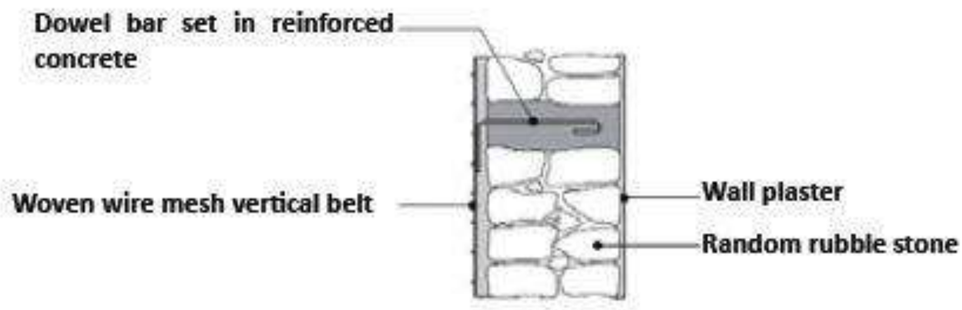


Figure 7-7 Single-sided jacketing showing steel dowels

7.8.4 SEISMIC BAND AND BELT

7.8.4.1 Seismic Bands (Ring Beams)[25]

A seismic band is the most critical earthquake-resistant provision in a stone masonry building. Usually provided at lintel, floor, and/or roof level in a building, the band acts like a ring or belt, as shown in Figure 5-7. Seismic bands are constructed using either reinforced

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25 A TUTORIAL: Improving the Seismic Performance of Stone Masonry Buildings, Jitendra Bothara • Svetlana Brzev, First Edition, April 2011 concrete (RC) or timber. Proper placement and continuity of bands and proper use of materials and workmanship are essential for their effectiveness.

Seismic bands hold the walls together and ensure integral box action of an entire building. Also, a lintel band reduces the effective wall height. As a result, bending stresses in the walls due to out-of-plane earthquake effects are reduced and the chances of wall delamination are diminished.

During earthquake shaking, a band undergoes bending and pulling actions, as shown in Figure 5-6. A portion of the band perpendicular to the direction of earthquake shaking is subjected to bending, while the remaining portion is in tension.

Seismic bands can be provided at plinth, lintel, floor, and roof levels (Figure 5-7). In some cases, a lintel band is combined with a floor or roof band. An RC plinth band should be provided a top the foundation when strip footings are made of unreinforced masonry and the soil is either soft or uneven in its properties.

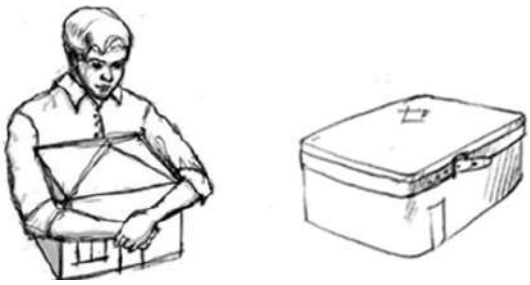

Figure 7-9 A seismic band acts like a belt (adapted from: GOM 1994)

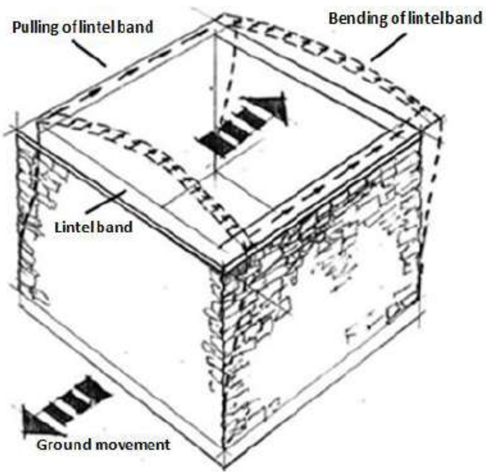

Figure 7-8 Pulling and bending of a lintel band in a stone masonry building (adapted from: Murty 2005)

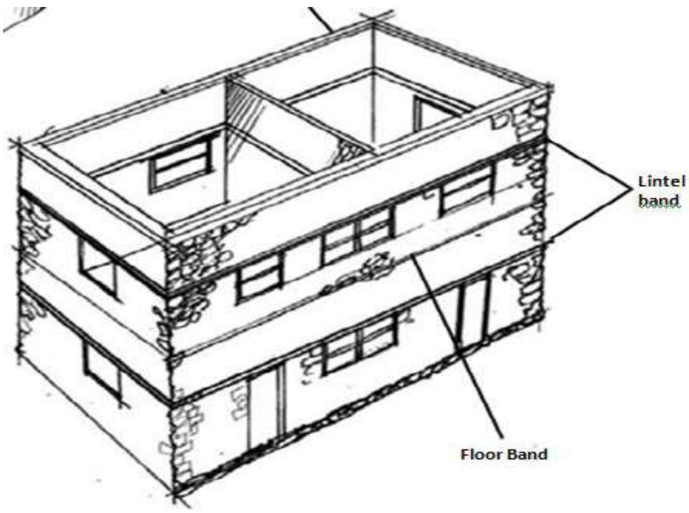

Figure 7-10 Location of seismic bands in a stone masonry building (roof omitted for clarity) (adapted from: UNCRD 2003)

Seismic bands are required at lintel and floor level when the floor and roof structures are flexible, the vertical distance between lintel and floor level is more than 400 mm, or when the total story height exceeds 2.5 m (the same is true of roof bands as well). Otherwise, the provision of a lintel band is sufficient. A floor/roof band is not required in buildings with RC floor/roof structures. In such cases, the slab itself ties the walls together.

Seismic bands must be continuous (like a loop or a belt), otherwise they are inefficient. Some examples of undesirable discontinuities in lintel band construction are illustrated in Figures 5-10 and 5-11.

Lintel beams (commonly known as lintels) are required a top all the openings in a wall. However, if a band is provided at the lintel level, a lintel beam can be cast as an integral part of the lintel band to minimize construction costs, as illustrated in Figure 5-12. Details for combining a lintel and floor/roof band are shown in Figure 5-13. The band must be continuously reinforced at the wall intersections, as shown in Figure 5-14.

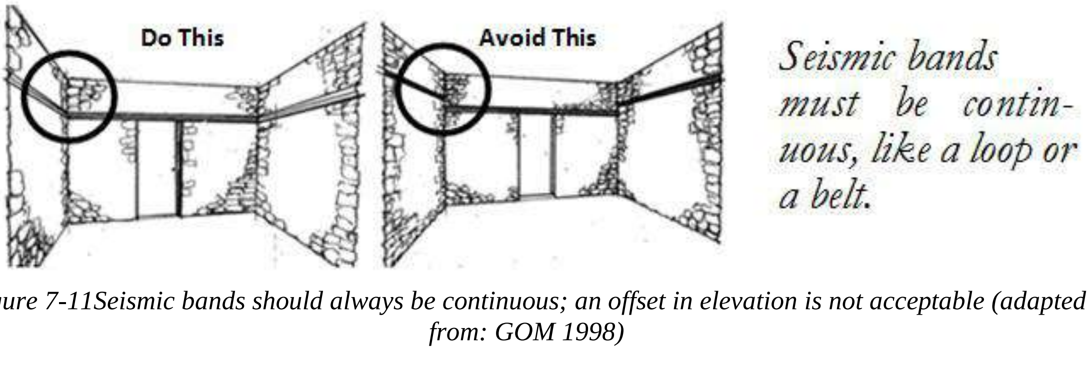

Figure 7-11Seismic bands should always be continuous; an offset in elevation is not acceptable (adapted from: GOM 1998)

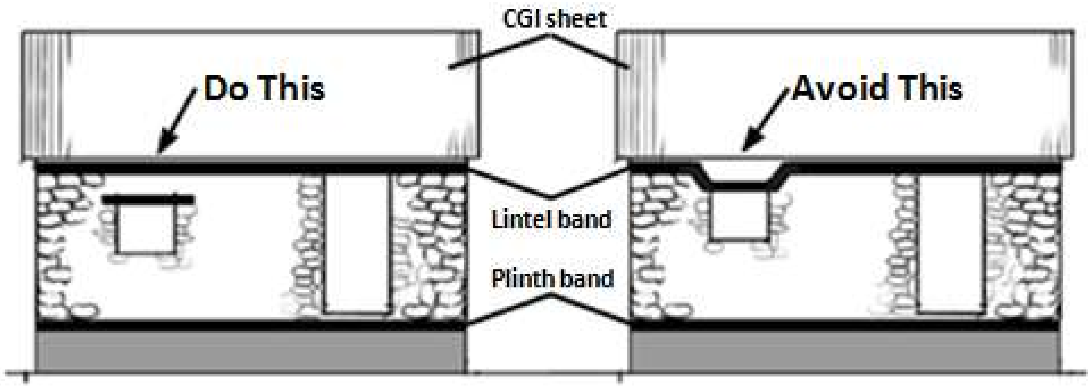

Figure 7-12RC seismic bands should always remain level without any dips or changes in height (adapted from: GOM 1998)

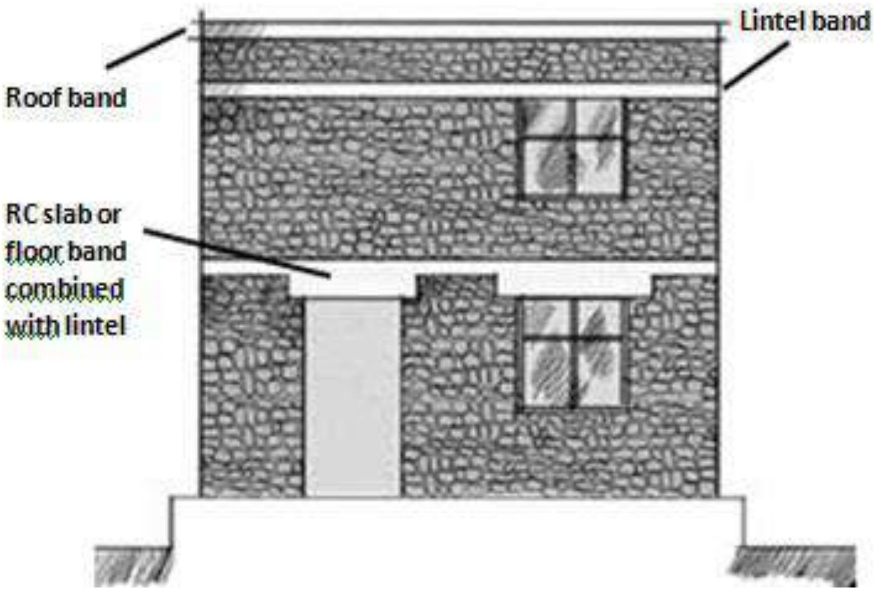

Figure 7-13 Merging RC floor and lintel bands

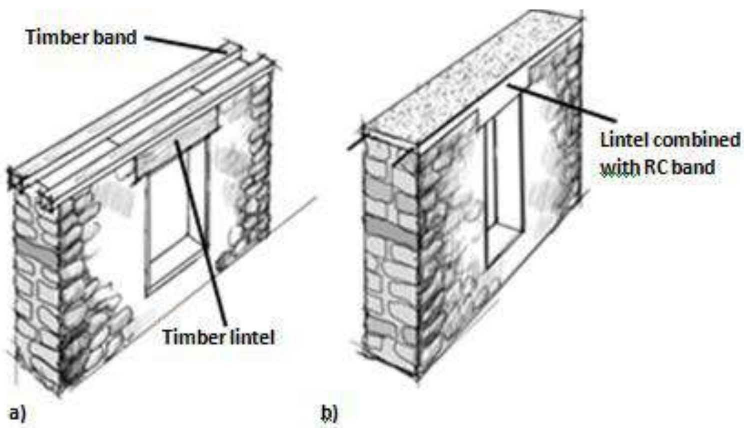

Figure 7-14 Combining floor/roof and lintel band: a) timber band, b) RC band

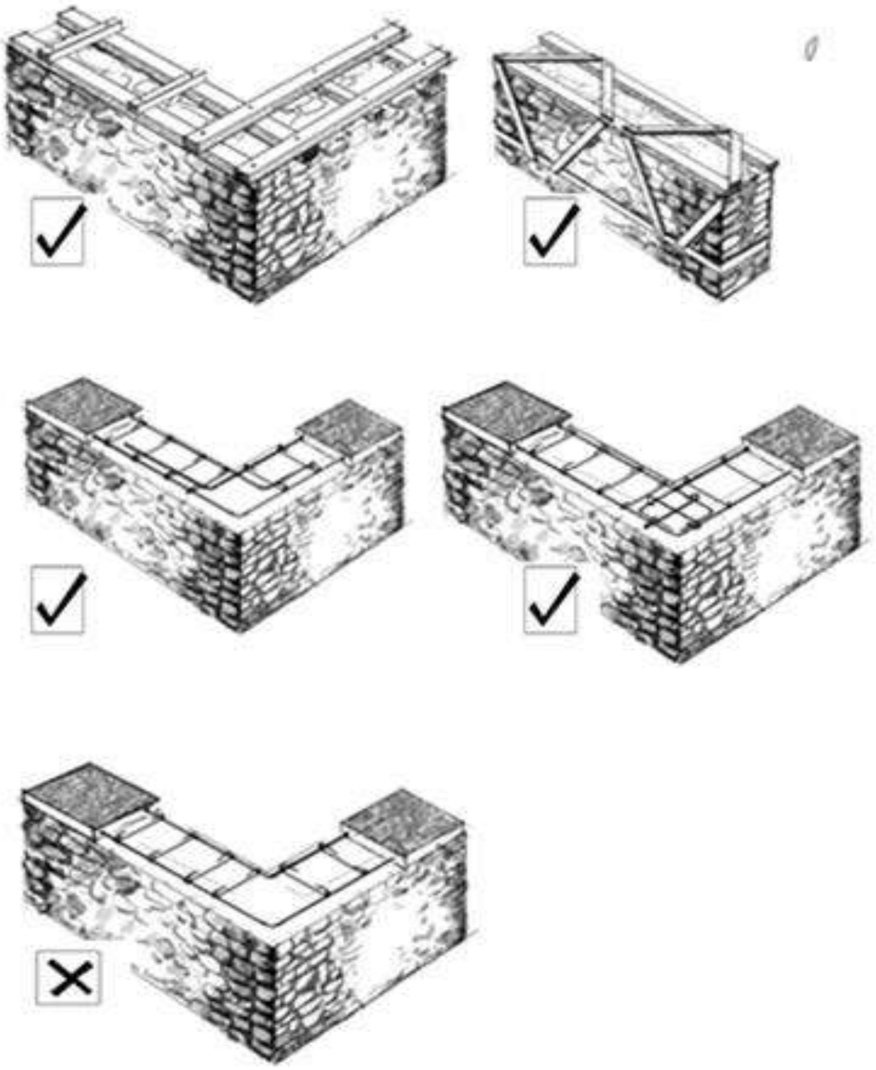

Figure 7-15Recommended detailing of timber and RC bands (adapted from: T. Schacher and C.V.R Murty)

7.9 VERTICAL REINFORCEMENT AT CORNERS AND THE JUNCTIONS OF WALLS[26]

The vertical reinforcement consisting of TOR bar as per Table 5-1 or equivalent shall be provided on the inside corner of room starting from 750 mm below the ground floor going up to the roof slab, passing through each middle floor through holes made in the slabs. (As in fig. shown) The reinforcement will be connected to the walls by using L shape dowels of 8 mm TOR bar, the vertical leg of 400 mm length firmly tied to the vertical reinforcement bars and the horizontal leg of minimum 150 mm length embedded in the walls through 75 mm dia.

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holes drilled in the wall into which the 8 mm dia. leg of the dowel will be grouted using non-shrink cement cum polymer grout. Such dowels will be provided, first one just above plinth level and then at about every 1 m distance apart. The corner reinforcement will be covered with 1:3 cement mortars or 1:1 ½:3 micro concrete fully bonded with the walls giving a minimum cover of 15 mm on the bar.

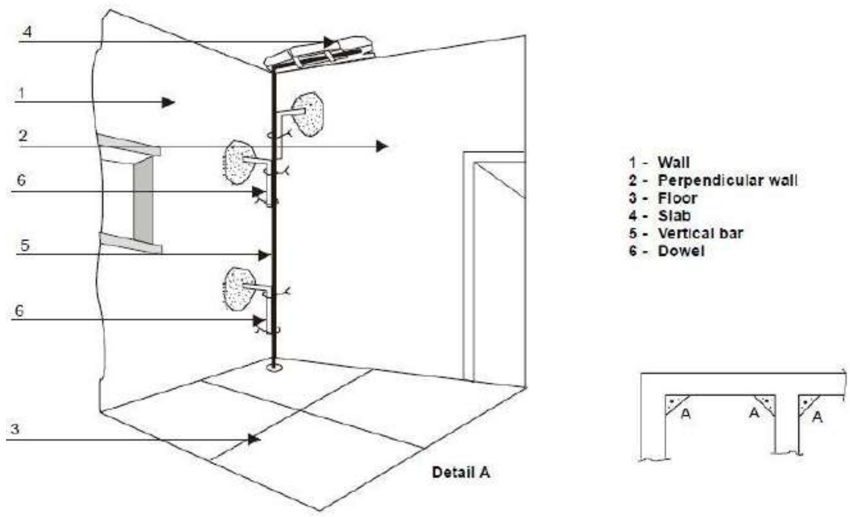

Figure 7-16 Vertical Reinforcement at corner and Junctions of Walls

Table 7-1 Vertical Bar of Mesh Reinforcement in Vertical Belt at inside Corners of Rooms

No. of storeys	Storeys	Single Bar, mm	Mesh (g 10)	
			N	B
One	One	12	14	400
Two	Top	12	14	400
	Bottom	16	14	400
			With 1 bars of 12Φ	
Three	Top	12	14	400
	Middle	16	14	400
			With 1 bars of 12Φ	
	Bottom	16	14	400
			With 1 bars of 12Φ	

1) Gauge 10 (3.25 mm dia) galvanized mesh with 25 mm spacing of wires shall be used.
2) Single bar, if used, shall be HSD or TOR type. If two bars are used at a T-junction, the diameter can be taken as follows. For one of 10 or 12 mm take 2 of 8 mm, and for one of 16 mm take 2 of 12 mm.
3) N = Number of longitudinal wires in the mesh.
4) B = Width of the micro concrete belt, half on each all meeting at the corner of T-junction.
5) The transverse wire in the mesh could be at spacing up to 150 mm.

7.9.1 HOW TO INSTALL VERTICAL BAR IN A CORNER[27]

1. Identify the inside corner for installation of vertical bar. Select appropriate location to maintain vertical continuity between storeys in case of a multi-storey structure
2. Mark the area where the bar is to be installed. Using plumb-bob, demarcate a 100 mm (4") wide patch at the corner on both walls as the limits of concreting for encasing the rod.
3. Use electric grinder if available, cut the plaster along vertical boundary of both the patches to restrict the removal of plaster.
4. Remove the plaster from the marked area and expose the walling material. Rake all the mortar joints to the depth of 12 mm (½”). Clean the surface with a wire brush.
5. Remove flooring within 300 mm x 300 mm patch at the corner and excavate to 450 mm depth.
6. Make holes for installing shear connectors in both walls, starting on one wall at 150 mm (6") from the floor, with successive holes at approximately every 600 mm (2') but in alternate walls, and the last hole 150 mm below the ceiling level or 150 mm below eave level. Clean all the holes with wire brush to remove loose material.
7. Place appropriate diameter bar in the floor excavation with the lower 150 mm (6") bent in ‘L’ shape. In a structure with CGI roof, the top end can be connected to one of the principal elements of the attic floor or the roof. In case of an RC slab roof, the top end can be bent into ‘L’ shape for connecting to the slab reinforcement. The rod will pass through each intermediate floor.

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8. Place appropriately shaped 8 mm TOR bar in the holes made for shear connectors and connect them to the vertical bar making sure that the vertical bar is 35 to 50 mm (½" to 2") from each wall.
9. With vertical bar plumb and at right distance from the walls pour concrete in 1:2:4 proportion in the hole excavated in the floor, with continuous rodding, to completely encase the bottom of the steel rod in concrete.
10. Clean all the shear connector holes by splashing water and wetting the surface of the holes thoroughly. Fill up the holes with non-shrink cement cum polymer grout. Make sure that the grout completely encases the shear connector bar.
11. Once all the shear connectors are grouted, clean the exposed surfaces of the wall with wire brush and water.
12. Install centering for concreting around the vertical bar. This can be done with GI sheet or timber plank. The concreting must be done in stages with the height of each new stage not exceeding 900 mm (3'). Pour 1:1½:3 micro-concrete into the form work, with continuous rodding to prevent honeycombing. Once the concrete is set, move the formwork upwards and continue concreting. Encase the entire length of the vertical bar in this manner. The bar must have the minimum concrete cover of 15 mm. Connecting top bent end of vertical rod to slab reinforcement
13. Where the roof is of RC slab, in the vicinity of the vertical bar, break the bottom concrete cover to expose the slab reinforcing bars. Connect the top bent portion of the vertical bar to the exposed bars of the slab using binding wires providing a minimum of 300 mm (12") overlap. Wet the exposed surface of the slab and then apply neat cement slurry. Finally apply cement mortar in 1:4 proportions and finish the joint to match the surrounding area.
14. Cure all concrete work for 15 days.

7.10 FOUNDATION REHABILITATION (BASED ON FEMA 356)

1. Soil Material Improvements. Improvement in existing soil materials may be effective in the rehabilitation of foundations by achieving one or more of the following results: (a) improvement in vertical bearing capacity of footing foundations, (b) increase in the lateral frictional resistance at the base of footings, (c) and increase in the passive resistance of the soils adjacent to foundations or grade beams.

Soil improvement options to increase the vertical bearing capacity of footing foundations are limited. Soil removal and replacement and soil vibratory densification usually are not feasible because they would induce settlements beneath the footings or be expensive to implement without causing settlement. Grouting may be considered to increase bearing capacity. Different grouting techniques are discussed in *FEMA 274* Section C4.3.2. Compaction grouting can achieve densification and strengthening of a variety of soil types and/or extend foundation loads to deeper, stronger soils. The technique requires careful control to avoid causing uplift of foundation elements or adjacent floor slabs during the grouting process. Permeation grouting with chemical grouts can achieve substantial strengthening of sandy soils, but the more fine-grained or silty the sand, the less effective the technique becomes. Jet grouting could also be considered. These same techniques also may be considered to increase the lateral frictional resistance at the base of footings.

Soil improvement by the following methods may be effective in increasing the passive resistance of soils adjacent to foundations or grade beams; removal and replacement of existing soils with stronger, well-compacted soils or with treated (e.g., cement-stabilized) soils; in-place mixing of existing soils with strengthening materials (e.g., cement); grouting, including permeation grouting and jet grouting; and in-place densification by impact or vibratory compaction. In-place densification by impact or vibratory compaction should be used only if the soil layers to be compacted are not too thick and vibration effects on the structure are tolerable.

2. Shallow Foundation Rehabilitation. The following measures may be effective in the rehabilitation of shallow foundations:

2.1 New isolated or spread footings may be added to existing structures to support new structural elements such as shear walls or frames.

2.2 Existing isolated or spread footings may be enlarged to increase bearing or uplift capacity. Consideration of existing contact pressures on the strength and stiffness of the modified footing may be required unless uniform distribution is achieved by shoring and/or jacking.

2.3 Existing isolated or spread footings may be underpinned to increase bearing or uplift capacity. Underpinning improves bearing capacity by lowering the contact horizon of the footing. Consideration of the effects of jacking and load transfer may be required.

2.4 Uplift capacity may be improved by increasing the resisting soil mass above the footing.

2.5 Mitigation of differential lateral displacement of different portions of a building foundation may be carried out by provision of interconnection with grade beams, reinforced grade slab or ties.

3. Deep Foundation Rehabilitation. The following measures may be effective in the rehabilitation of deep foundation consisting of driven piles made of steel, concrete, or wood, or cast-in-place concrete piers, or drilled shafts of concrete.

3.1 Shallow foundation of spread footings or mats may be provided to support new shear walls or frames or other new elements of the lateral force-resisting system, provided the effects of differential foundation stiffness on the modified structure are analyzed and meet the acceptance criteria.

3.2 New wood piles may be provided for an existing wood pile foundation. A positive connection should be provided to transfer the uplift forces from the pile cap or foundation above to the new wood piles. Existing wood piles should be inspected for deterioration caused by decay, insect infestation, or other signs of distress prior to undertaking evaluation of existing wood pile foundation.

3.3 Driven piles made of steel, concrete, or wood, or cast-in-place concrete piers or drilled shafts of concrete, may be provided to support new structural elements such as shear walls or frames.

3.4 Driven piles made of steel, concrete, or wood, or cast-in-place concrete piers or drilled shafts of concrete, may be provided to supplement the vertical and lateral capacities of existing pile and pier foundation groups.

7.11 OTHER ADVANCED APPROACHES FOR RETROFITTING OF MASONRY STRUCTURES

7.11.1 ENERGY DISSIPATION DEVICES FOR EARTHQUAKE RESISTANCE (DAMPER)[28]

Another approach for controlling seismic damage in buildings is to install Seismic Dampers in place of some structural elements, such as diagonal braces. These dampers act like the hydraulic shock absorbers in cars that absorb sudden jerks. When seismic energy is transmitted through them, dampers absorb part of the energy, thus damping the vibration of the building. By equipping a building with devices that have high damping capacity, the seismic energy entering the building is greatly decreased. This system has also been used in historic buildings such as City Hall in San Francisco.

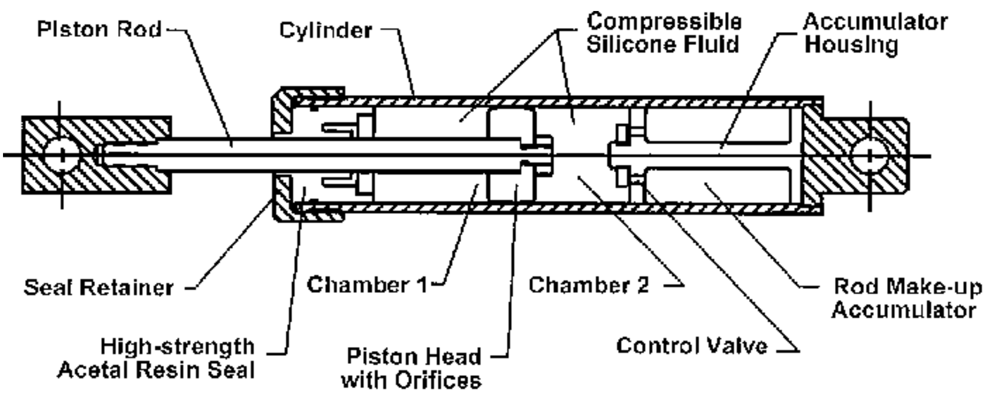

Figure 7-17 Typical Damper

[28] Two Activities—Base Isolation for Earthquake Resistance, TOTLE.

Commonly used types of seismic dampers include:

I) **Viscous Dampers** *(energy is absorbed by silicone-based fluid passing between piston cylinder arrangement)*

II) **Friction Dampers** *(energy is absorbed by surfaces with friction between them rubbing against each other*

III) **Yielding Dampers** *(energy is absorbed by metallic components that yield)*

IV) **Viscoelastic dampers** *(energy is absorbed by utilizing the controlled shearing of solids)*

7.11.2 BASE ISOLATION[29]

Base isolation is the most powerful tool of earthquake engineering. It is meant to enable a building to survive a potentially devastating seismic impact through a proper initial design or subsequent modifications. Contrary to popular belief base isolation does not make a building earthquake proof.

The seismic base isolation technology involves placing flexible isolation systems between the foundation and the superstructure. By means of their flexibility and energy absorption capability, the isolation systems reflect and absorb part of the earthquake input energy before this energy is fully transmitted to the superstructure, reducing the energy dissipation demand on the superstructure.

Many base isolators look like large rubber pads, although there are other types that are based on sliding of one part of the building relative to other. Base isolation is particularly effective for retrofitting low to medium height unreinforced masonry buildings, such as historic buildings. Portland’s historic Pioneer Courthouse has been seismically retrofitted using base isolation. Experiments and observations of base-isolated buildings in earthquakes indicate that building acceleration can be reduced to as little as one-quarter of the ground acceleration.

i) **Lead-rubber bearings** are frequently used for base isolation. A lead rubber bearing is made from layers of rubber sandwiched together with layers of steel.

[29] Two Activities—Base Isolation for Earthquake Resistance, TOTLE.

The bearing is very stiff and strong in the vertical direction, but flexible in the horizontal direction.

ii) **Spherical sliding isolation** uses bearing pads that have a curved surface and low-friction materials similar to Teflon. During an earthquake the building is free to slide both horizontally and vertically on the curved surfaces and will return to its original position after the ground shaking stops. The forces needed to move the building upwards limit the horizontal or lateral forces that would otherwise cause building deformations.

iii) **Working Principle** To get a basic idea of how base isolation works, first examine the diagrams above that illustrate traditional earthquake mitigation methods. When an earthquake vibrates a building with a fixed foundation, the ground vibration is transmitted to the building. The buildings displacement in the direction opposite the ground motion is actually due to inertia. In addition to displacing in a direction opposite to ground motion, the un-isolated building is deformed. If the deformation exceeds the constraints of the building design, the structure of the building will fail. This failure often occurs in the ground floor because most of the building's mass is above that level. Also many buildings have "soft" ground floors with many windows or unreinforced spaces for parking or lobbies.

7.11.3 NON-METALLIC FIBRE COMPOSITES/FIBRE REINFORCED COMPOSITES (FRC)[30]

Commonly used forms of FRC viz. Precured CFRC (Carbon Fibre Reinforced Composite), Glass Fibre Reinforced polymer Composites (GFRC) rebar, glass fibre roll, etc.

Fibre Reinforced Polymer (FRP) composites comprise fibres of high tensile strength within a polymer matrix such as vinylester or epoxy. FRP composites have emerged from being exotic materials used only in niche applications following the Second World War, to common engineering materials used in a diverse range of applications such as aircraft, helicopters, space-craft, satellites, ships, submarines, automobiles, chemical processing equipment,

[30] New and emerging technologies for retrofitting and repairs, Dr Gopal Rai, CEO, R & M International Group of companies

sporting goods and civil infrastructure. The role of FRP for strengthening of existing or new reinforced concrete structures is growing at an extremely rapid pace owing mainly to the ease and speed of construction, and the possibility of application without disturbing the existing functionality of the structure. FRP composites have proved to be extremely useful for strengthening of RCC structures against both normal and seismic loads. The FRP used for strengthening of RC structures can be mainly categorized as :

(i) Laminates, for flexural strengthening

The laminates are generally made up of Carbon fibres blended in an epoxy matrix. These when applied with epoxy, act as external tension reinforcements to increase the flexural strength of the RCC members.

The main advantages of Fibre reinforced composite laminates No corrosion, No transportation problem, High ultimate strength, High Young’s modulus, Very good fatigue properties, Low weight and Endless tapes available so no joints.

Figure 7-18 Use of Laminates in for strengthening of slabs in a bridge and a building.

Fibre wraps, for shear and axial strengthening

Fibre wraps are made up of three different materials namely Carbon, Aramid and Glass. Carbon fibre is the strongest, most inert and the most expensive one; glass is the cheapest and has low elastic modulus and strength. Aramid fibre is used mainly for impact resistance. The concept of flexural and shear strengthening of RC beams using FRP composites is quite straight forward and exactly similar to the steel reinforcement used for normal RC construction. For flexural strengthening, the laminates act as longitudinal reinforcement and for shear strengthening, the wraps act as shear reinforcement (stirrups).

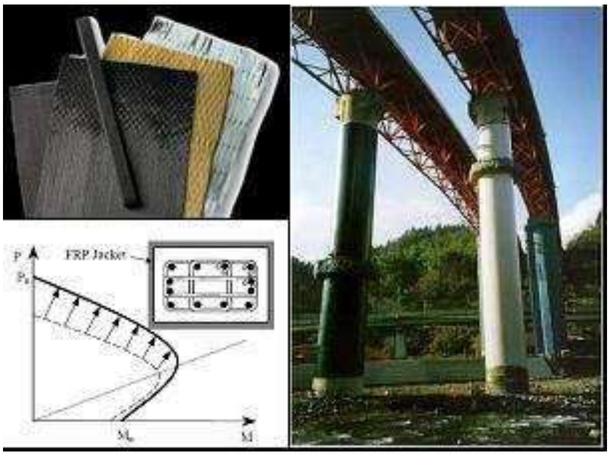

Figure 7-19 Different types and uses of Fibre wraps

7.12 ALTERNATIVE APPROACH FOR ANALYSIS

7.12.1 PERFORMANCE BASED BEHAVIOR OF MASONRY[31]

7.12.1.1 Building Performance Levels

In case where sufficient technical know-how is available, following alternative approach can be adapted.

i. ***Immediate Occupancy (IO)***
ii. ***Life Safety (LS)***
iii. ***Collapse Prevention (CP)***

Target Building Performance Levels

FEMA 356 defines a number of target building performance levels that can be used to assess an existing building. The main performance levels are as follows:

- Operational (O)
- Immediate occupancy (IO)
- Life-safety (LS)
- Collapse prevention (CP)

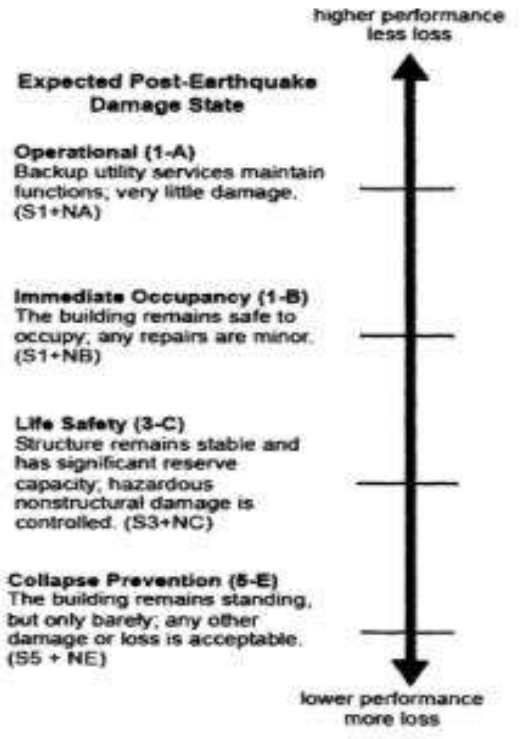

Figure 7-20 Range of performance levels (FEMA 356-2000)

31 Adapted from FEMA 356 - 2000

These performance levels are fairly self-explanatory and based on the desired condition of structural and architectural components in the building after an earthquake. Figure 7-2 shows the range of performance levels and the expected damage state after the seismic event. These performance levels are combined with the earthquake hazard level of the site to obtain the rehabilitation objective for the project.

The following damage levels are allowed in an unreinforced masonry building for each performance level. The collapse prevention level allows extensive cracking, peeling off of face course and veneer, and noticeable in-plane and out-of-plane offsets in the main shear/load bearing walls. Non-structural walls can completely dislodge but drift must not exceed 1%. For the life safety level, extensive cracking and noticeable in-plane offsets are allowed in both structural and non-structural elements. Out-of-plane offsets must be minor and drift cannot exceed 0.6%. Finally, for the immediate occupancy and operational performance level, only minor cracking and spalling of the veneer is allowed with no noticeable out-of-plane offsets. Drift must not exceed 0.3%.

7.13 PROPERTIES OF MASONRY WALLS

7.13.1 IN PLANE PROPERTIES OF URM WALL

7.13.1.1 Masonry Shear Strength

For URM components, expected masonry shear strength, v_{me}, shall be measured using an approved in place shear test. Expected shear strength shall be determined in accordance with Equation (7-1a):

$$v_{me} = \frac{0.75\left(0.75v_{te} + \frac{P_{CE}}{A_n}\right)}{1.5} \quad \text{.. (7-1a)}$$

where,

P_{CE} = Expected gravity compressive force applied to a wall or pier component considering load combinations of gravity load and earthquake load

A_n = Area of net mortared/grouted section of a wall or pier

v_{te} = Average bed-joint shear strength, v_{to}, given in Equation (7-1b)

Values for the mortar shear strength, v_{te}, shall not exceed 100 psi (690 Kpa) for the determination of v_{me} in Equation (7-1a). The 0.75 factor on v_{te} shall not be applied for single wythe masonry walls. Individual bed joint shear strength test values, v_{to}, shall be determined in accordance with Equation (7-1b):

$$v_{to} = \frac{V_{test}}{A_b} - P_{D+L} \quad \text{…………………………………………………. (7-2)}$$

where,

V_{test} = Test load at first movement of a masonry unit

A_b = Sum of net mortared area of bed joints above and below the test unit

P_{D+L}= Stress due to gravity loads at the test location

The in-place shear test shall not be used to estimate shear strength of reinforced masonry components.

7.13.1.2 Expected Lateral Strength of Unreinforced Masonry Walls and Piers (Deformation Controlled)

Expected lateral strength, QCE , of existing and enhanced URM walls or pier components shall be the lesser of the lateral strength based on expected bed-joint sliding shear strength or expected rocking strength, calculated in accordance with Equations (7-2) and (7-3), respectively:

$$Q_{CE} = V_{bjs} = v_{me} A_n \quad \text{……………………………………………. (7-2)}$$

$$Q_{CE} = V_r = 0.9\alpha P_E \left(\frac{L}{h_{eff}} \right) \quad \text{……………………………………………. (7-3)}$$

where,

A_n = Area of net mortared/grouted section

h_{eff} = Height to resultant of lateral force

L = Length of wall or pier

P_E = Expected axial compressive force due to gravity loads

v_{me} = Expected bed-joint sliding shear strength

V_{bjs} = Expected shear strength of wall or pier based on bed-joint sliding shear strength

Vr = Strength of wall or pier based on rocking

α = Factor equal to 0.5 for fixed-free cantilever wall, or equal to 1.0 for fixed- fixed pier

7.13.1.3 Lower Bound Lateral Strength of Unreinforced Masonry Walls and Piers (Force controlled)

Lower bound lateral strength, Q_{CL}, of existing and enhanced URM walls or pier components shall be taken as the lesser of the lateral strength values based on diagonal tension stress or toe compressive stress calculated in accordance with Equations (7-4) and (7-5), respectively. L / h_{eff} shall not be taken less than 0.67 for use in Equation (7-5).

$$Q_{CL} = V_{dt} = f'_{dt} A_L \left(\frac{L}{h_{eff}}\right)\sqrt{1+\frac{f_a}{f'_{dt}}} \quad \text{……………………………………..(7-4)}$$

$$Q_{CL} = V_{tc} = \alpha P_L \left(\frac{L}{h_{eff}}\right)\left(1-\frac{f_a}{0.7 f'_m}\right) \quad \text{………………………………………. (7-5)}$$

where A_n, h_{eff}, L, and α are the same as given for Equations (7-4) and (7-5) and:

f_a = Axial compressive stress due to gravity loads

f'_{dt} = Lower bound masonry diagonal tension strength

f'_m = Lower bound masonry compressive strength

P_L = Lower bound axial compressive force due to gravity loads

V_{dt} = Lower bound shear strength based on diagonal tension stress for wall or pier

V_{tc} = Lower bound shear strength based on toe compressive stress for wall or pier

7.13.2 OUT OF PLANE PROPERTIES OF URM WALL

Walls shall be evaluated for out-of-plane inertial forces as required by this section. Forces specified in this section shall be considered force-controlled actions.

7.13.2.1 Out-of-Plane Anchorage to Diaphragms

Walls shall be positively anchored to all diaphragms that provide lateral support for the wall or are vertically supported by the wall. Walls shall be anchored to diaphragms at horizontal distances not exceeding 200 mm, unless it can be demonstrated that the wall has adequate capacity to span horizontally between the supports for greater distances. Anchorage of walls to diaphragms shall be designed for forces calculated using Equation (7-6), which shall be developed in the diaphragm. If sub-diaphragms are used, each sub diaphragm shall be capable of transmitting the shear forces due to wall anchorage to a continuous diaphragm tie. Sub-diaphragms shall have length-to-depth ratios not exceeding 3:1. Where wall panels are stiffened for out-of-plane behavior by pilasters or similar elements, anchors shall be provided at each such element and the distribution of out-of-plane forces to wall anchors and diaphragm ties shall consider the stiffening effect and accumulation of forces at these elements. Wall anchor connections shall be considered force-controlled.

$$F_p = S_{xs} w \chi \qquad \text{(7-6)}$$

where,

F_p = Design force for anchorage of walls to diaphragms

χ = Factor from Table 7-1 for the selected Structural Performance Level. Increased values of χ shall be used when anchoring to flexible diaphragms

S_{XS} = Spectral response acceleration parameter at short periods for the selected hazard level and damping adjusted for site class

W = Weight of the wall tributary to the anchor

Exceptions:

1. *Fp* shall not be less than the 6 KN/m.

Structural Performance Level	χ^1 Flexible Diaphragms	χ^1 Other Diaphragms
Collapse Prevention	0.9	0.3
Life Safety	1.2	0.4
Immediate Occupancy	1.8	0.6

Table 7-1 Coefficient χ for Calculation of Out of Plane Wall Forces

7.13.2.2 Out-of-Plane Strength

Wall components shall have adequate strength to span between locations of out-of-plane support when subjected to out-of-plane forces calculated using Equation (7-7).

$$F_p = S_{xs} w \chi \qquad \text{(7-7)}$$

where,

F_p = Out-of-plane force per unit area for design of a wall spanning between two out-of-plane supports

χ = Factor from Table 7-1 for the selected performance level. Values of χ for flexible diaphragms need not be applied to out-of-plane strength of wall components

S_{XS} = Spectral response acceleration at short periods for the selected hazard level and damping

adjusted for site class

W = Weight of the wall per unit area

i. Stiffness

The out-of-plane stiffness of walls shall be neglected in analytical models of the global structural system in the orthogonal direction.

ii. Strength

Unless arching action is considered, flexural cracking shall be limited by the expected tensile stress values measured using one of the following three methods:

1. Test samples shall be extracted from an existing wall and subjected to minor-axis bending using the bondwrench method of *ASTM C1072-99*.
2. Test samples shall be tested in situ using the bondwrench method.
3. Sample wall panels shall be extracted and subjected to minor-axis bending in accordance with *ASTM E518-00*.

Flexural tensile strength for unreinforced masonry (URM) walls subjected to in-plane lateral forces shall be assumed to be equal to that for out-of-plane bending, unless testing is done to define the expected tensile strength for in-plane bending.

Arching action shall be considered only if surrounding floor, roof, column, or pilaster elements have sufficient stiffness and strength to resist thrusts from arching of a wall panel, and a condition assessment has been performed to ensure that there are no gaps between a wall panel and the adjacent structure.

The condition of the collar joint shall be considered when estimating the effective thickness of a wall for out-of-plane behavior. The effective void ratio shall be taken as the ratio of the collar joint area without mortar to the total area of the collar joint. Wythes separated by collar joints that are not bonded, or have an effective void ratio greater than 50% shall not be considered part of the effective thickness of the wall.

iii. Acceptance Criteria

For the Immediate Occupancy Structural Performance Level, flexural cracking in URM walls due to out-of plane inertial loading shall not be permitted as limited by the tensile stress requirements of Section 7.2.1.4.4. For the Life Safety and Collapse Prevention Structural Performance Levels, flexural cracking in URM walls due to out-of-plane inertial loading shall be permitted provided that cracked wall segments will remain stable during dynamic excitation. Stability shall be checked using analytical time-step integration models considering acceleration time histories at the top and base of a wall panel. For the Life Safety and Collapse Prevention Structural Performance Levels, stability need not be checked for walls spanning vertically with a height-to-thickness (h/t) ratio less than that given in Table 7-2.

Wall Types	S_{X1} $\leq 0.24g$	$0.24g < S_{X1}$ $\leq 0.37g$	S_{X1} $> 0.37g$
Walls of one-story buildings	20	16	13
First-story wall of multistory building	20	18	15
Walls in top story of multistory building	14	14	9
All other walls	20	16	13

Table 7-2 Permissible h/t Ratios for URM Out-of-Plane Walls

7.13.3 IN PLANE PROPERTIES OF REINFORCED MASONRY WALL

i. Stiffness

The stiffness of a reinforced masonry wall or pier component in-plane shall be determined as follows:

1. The shear stiffness of RM wall components shall be based on uncracked section properties.
2. The flexural stiffness of RM wall components shall be based on cracked section properties. Use of a cracked moment of inertia equal to 50 percent of Ig shall be permitted. In either case, veneer wythes shall not be considered in the calculation of wall component properties. Stiffnesses for existing and new walls shall be assumed to be the same.

Use of a cracked moment of inertia equal to 50 percent of Ig shall be permitted. In either case, veneer wythes shall not be considered in the calculation of wall component properties. Stiffnesses for existing and new walls shall be assumed to be the same.

ii. Strength

The strength of RM wall or pier components in flexure, shear, and axial compression shall be determined in accordance with the requirements of this section. The assumptions, procedures, and requirements of this section shall apply to both existing and new RM wall or pier components.

iii. Flexural Strength of Walls and Piers

Expected flexural strength of an RM wall or pier shall be determined based on the following assumptions:

1. Stress in reinforcement below the expected yield strength, f_{ye}, shall be taken as the expected modulus of elasticity, E_{se}, times the steel strain. For reinforcement strains larger than those corresponding to the expected yield strength, the stress in the reinforcement shall be considered independent of strain and equal to the expected yield strength, f_{ye}.
2. Tensile strength of masonry shall be neglected when calculating the flexural strength of a reinforced masonry cross-section.
3. Flexural compressive stress in masonry shall be assumed to be distributed across an equivalent
rectangular stress block. Masonry stress of 0.85 times the expected compressive strength, f_{me}, shall be distributed uniformly over an equivalent.
compression zone bounded by the edge of the cross-section and a depth equal to 85% of the depth from the neutral axis to the extreme fiber of the cross-section.
4. Strains in the reinforcement and masonry shall be considered linear through the cross-section. For purposes of determining forces in reinforcing bars distributed across the section, the maximum compressive strain in the masonry shall be assumed to be equal to 0.003

iv. Shear strength of walls and piers

The lower bound shear strength of an RM wall or pier, V_{CL}, shall not exceed the value computed in accordance with Equations (7-1) and (7-2). For intermediate values of *M/Vdv*, interpolation shall be used.

For *M/Vdv* less than 0.25:

$$V_{CL} \leq 6\sqrt{f'_m A_n} \quad \text{……………………………………….} \quad (7\text{-}8)$$

For *M/Vdv* greater than or equal to 1.00:

$$V_{CL} \leq 4\sqrt{f'_m A_n} \quad \text{………………………………………….} \quad (7\text{-}9)$$

where:

A_n = Area of net mortared/grouted section

f'_m = Lower bound compressive strength of masonry

M = Moment on the masonry section

V = Shear on the masonry section

dv = Wall length in direction of shear force

The lower-bound shear strength, V_{mL}, provided by the masonry shall be determined using Equation (7-10)

$$V_{mL} = \left[4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right] An\sqrt{f'_m} + 0.25P_L \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(7-10)}$$

where M/Vdv shall be limited to 1.0, and PL is the lower-bound vertical compressive force in pounds due to gravity loads, specified in Equation (7-2).

The lower-bound shear strength, V_{sL}, resisted by the reinforcement shall be determined using Equation (7 – 11)

$$V_{sL} = 0.5\left(\frac{A_v}{s}\right) fyd_v \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(7-11)}$$

where:

Av = Area of shear reinforcement

s = Spacing of shear reinforcement

fy = Lower bound yield strength of shear reinforcement

For RM walls or piers in which shear is considered a deformation-controlled action, expected shear strength, V_{CE}, shall be calculated using Equations (7-1) through (7-5) substituting expected material properties in lieu of lower-bound.

v. Strength considerations for flanged walls

Wall intersections shall be considered effective in transferring shear when either condition (1) or (2) and condition (3) are met:

1. The face shells of hollow masonry units are removed and the intersection is fully grouted.

2. Solid units are laid in running bond, and 50% of the masonry units at the intersection are interlocked.

3. Reinforcement from one intersecting wall continues past the intersection a distance not less than 40 bar diameters or 24 inches.

The width of flange considered effective in compression on each side of the web shall be taken as the lesser of six times the thickness of the web, half the distance to the next web, or the actual flange on either side of the web wall.

The width of flange considered effective in tension on each side of the web shall be taken as the lesser of 3/4 of the wall height, half the distance to an adjacent web, or the actual flange on either side of the web wall.

vi. Vertical compressive strength of walls and piers

Lower bound vertical compressive strength of existing RM wall or pier components shall be determined using Equation (7-12):

$$Q_{CL} = P_{CL} = 0.8\left[0.85 f'_m \left(A_n - A_s\right) + A_s f_y\right] \quad \text{………………………. (7-12)}$$

where:

f'_m = Lower bound masonry compressive strength

f_y = Lower bound reinforcement yield strength

vii. Acceptance Criteria

The shear required to develop the expected strength of reinforced masonry walls and piers in flexure shall be compared to the lower bound shear strength defined in Section 7.2.1.5.4. For reinforced masonry wall components governed by flexure, flexural actions shall be considered deformation-controlled. For reinforced masonry components governed by shear, shear actions shall be considered deformation-controlled. Axial compression on reinforced masonry wall or pier components shall be considered force-controlled.

Expected strength in flexure shall be determined in accordance with Section 7.2.1.5.3, and lower bound strength in axial compression shall be determined in accordance with Section 7.2.1.5.6.

viii. Default Properties

Default lower-bound values for masonry compressive strength, elastic modulus in compression, flexural tensile strength, and masonry shear strength shall be based on Table 7-3. Default expected strength values for masonry compressive strength, elastic modulus in compression, flexural tensile strength, and masonry shear strength shall be determined by multiplying lower-bound values by an appropriate factor taken from Table 7-4.

Table : Default Lower-Bound Masonry Properties

	Masonry Condition[1]		
Property Good Fair Poor	**Good**	**Fair**	**Poor**
Compressive Strength (f'_m)	6205 Kpa	4135 Kpa	2070 Kpa
Elastic Modulus in Compression	$550f'_m$	$550f'_m$	$550f'_m$
Flexural Tensile Strength	140 Kpa	70 Kpa	0
Shear Strength			
Masonry with a running bond lay-up	185 Kpa	140 Kpa	90 Kpa
Fully grouted masonry with a lay-up other than running bond	185 Kpa	140 Kpa	90 Kpa
Partially grouted or ungrouted masonry with a lay-up other	75 Kpa	55 Kpa	35 Kpa
1. Masonry condition shall be classified as good, fair, or poor as defined in this standard.			

Table 7-2 Factors to Translate Lower-Bound Masonry Properties to Expected Strength Masonry Properties

Property	**Factor**
Compressive Strength (f_{me})	1.3
Elastic Modulus in Compression[2]	-
Flexural Tensile Strength	1.3

The expected elastic modulus in compression shall be taken as $550f_{me}$, where f_{me} is the expected masonry compressive strength.

7.14 ANALYSIS METHODS

FEMA 356 specifies four procedures that can be used to analyze an existing building. These are:*32*

- Linear Static Procedure
- Linear Dynamic Procedure
- Nonlinear Static Procedure
- Nonlinear Dynamic Procedure

7.14.1 ELASTIC ANALYSES

Elastic analyses assume linear behavior during a seismic event. This is clearly a stretch when considering URM buildings but the idea is to provide a quick estimate for the engineer to give him an idea as to what he is dealing with. FEMA 356 specifies two acceptable elastic analyses: the linear static procedure and the linear dynamic procedure. They are detailed here.

7.14.2 LINEAR STATIC PROCEDURE (LSP)

A special equation is given to approximate the fundamental period for URM buildings with flexible diaphragms (FEMA 356-2000):

$T = 0.0254(0.078\Delta_d)^{0.5}$ ………………………………..(7 – 13)

Where, Δ_d is the maximum in-plane diaphragm displacement (meter). This equation assumes that the in-plane deflection of the masonry walls is negligible compared to that of the flexible diaphragm.

Once the period is determined, the next step is to calculate the pseudo-lateral load from the following equation (FEMA 356-2000):

$V = C_1 C_2 C_3 C_m S_a W$ ………………………………. (7 – 14)

Where:

V = Pseudo lateral load

C_1 = Modification factor relating expected inelastic displacements to the calculated elastic response.

C_2 = Modification factor for stiffness degradation and strength deterioration (1.0 for LSP)

C_3 = Modification factor to account for increased displacements due to P-Delta effects

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32 A Performance Based Approach to Retrofitting Unreinforced Masonry Structures for Seismic Loads by Keith Bouchard, 2006

C$_m$ = Effective mass factor to account for higher mode mass participation (1.0 for URM)

S_a = Response spectrum acceleration at fundamental period and damping ratio of building (estimated at 5%)

W = Effective weight of the building

For URM buildings with flexible diaphragms and a fundamental period estimated from equation 7-13, the pseudo-lateral load is calculated for each span of the building and for each floor. It is then distributed to the vertical seismic-resisting elements (walls) according to tributary area. Forces in the diaphragm can then be calculated using these results.

The forces for each story determined from the pseudo lateral load are then compared to the story strengths to determine if they are acceptable. For elements that are limited by force-controlled failure modes, the governing equation is (FEMA 356-2000):

$\boldsymbol{KQ_{cL} \geq Q_{UF}}$ …………………………………..(7 – 15)

Where:

K = Knowledge factor

$\mathbf{Q_{CL}}$ = Lower-bound strength of component

$\mathbf{Q_{UF}}$ = Force-controlled design action

The knowledge factor is obtained from **FEMA 356** and depends on both the method used to determine component properties (testing vs. default) and the desired performance level. Table 7-6 shows the knowledge factor for different scenarios.

T*able 7-3 Knowledge factor according to acquired data (FEMA 356-2000)***)**

Data	Level of Knowledge							
	Minimum		Usual				Comprehensive	
Rehabilitation Objective	BSO or Lower		BSO or Lower		Enhanced		Enhanced	
Analysis Procedures	LSP, LDP		All		All		All	
Testing	No Tests		Usual Testing		Usual Testing		Comprehensive Testing	
Drawings	Design Drawings	Or Equivalent	Design Drawings	Or Equivalent	Design Drawings	Or Equivalent	Construction Documents	Or Equivalent
Condition Assessment	Visual	Compre-hensive	Visual	Compre-hensive	Visual	Compre-hensive	Visual	Compre-hensive
Material Properties	From Drawings or Default Values	From Default Values	From Drawings and Tests	From Usual Tests	From Drawings and Tests	From Usual Tests	From Documents and Tests	From Compre-hensive Tests
Knowledge Factor (κ)	0.75	0.75	1.00	1.00	0.75	0.75	1.00	1.00

For elements that are limited by deformation-controlled mechanisms, the governing equation also takes into account the ability of the wall to resist lateral loading after yield. For these piers, the equation is as follows (FEMA 356- 2000):

$mKQ_{CE} \geq Q_{UD}$ ……………………………….(7 – 16)

Where:

m = Modification factor to account for expected ductility of failure mode

Q_{CE} = Expected strength of component

$\mathbf{Q_{UD}}$ = Deformation-controlled design action

The "m" factor is obtained from FEMA 356 and again depends on the failure mode (only for deformation-controlled mechanisms) and the performance level of the building. Table 7-7 shows the factors for URM walls according to limit state and performance level.

Table 7-4 m-factor for URM walls (FEMA 356- 2000)

Limiting Behavioral Mode	*m*-factors				
	Performance Level				
		Primary		Secondary	
	IO	LS	CP	LS	CP
Bed-Joint Sliding	1	3	4	6	8
Rocking	$1.5h_{eff}/L$ (not less than 1)	$3h_{eff}/L$ (not less than 1.5)	$4h_{eff}/L$ (not less than 2)	$6h_{eff}/L$ (not less than 3)	$8h_{eff}/L$ (not less than 4)

Interpolation shall be used between table values.

It should be noted that the LSP is not applicable for all buildings. The standard designates that the procedure should not be used for buildings with a fundamental period greater than 3.5 times **Ts** or for buildings with significant structural or geometrical irregularities. For these structures, the linear dynamic or nonlinear procedures should be used.

7.14.3 LINEAR DYNAMIC PROCEDURE (LDP)

The linear dynamic procedure again assumes linear elastic stiffness and equivalent viscous damping values to model a structure. A modal spectral analysis that is not modified for nonlinear response is then used to find internal displacements and forces. As in the LSP, the idea is to approximate the actual displacements expected during an earthquake but produce conservative force values. The first step in the LDP is to characterize the ground motion. This can either be done through a response spectrum or a more in depth ground acceleration time history analysis. For the response spectrum analysis, enough modes need to be included to total 90% of the participating mass of the building in each direction. Modal responses are then combined using the "square root sum of squares" rule or the "complete quadratic combination" rule to determine peak member forces, displacements, story shears, and base reactions. The time-history method requires a time-step by time-step evaluation of a building response using recorded ground motions (FEMA 356-2000).

Forces and deformations obtained using the LDP should be modified using the C1**,** C2, and C3 factors defined in the previous section. The design forces are then compared to the expected or lower-bound wall strengths using the same acceptance criteria as in the linear static procedure.

7.14.4 INELASTIC ANALYSES

Inelastic analyses take into account the nonlinear behavior that a structure undergoes during a seismic event. This is much more accurate for URM buildings that are sure to exhibit this type of behavior post-cracking. FEMA 356 specifies two acceptable inelastic analyses: the nonlinear static procedure and the nonlinear dynamic procedure. They are detailed here.

7.14.5 NONLINEAR STATIC PROCEDURE (NSP)

The basis of the NSP is to incorporate the nonlinear load-deformation properties of a building into a mathematical model and then add incremental loading to that model until a target displacement is reached. This is sometimes called a "static pushover analysis." Since the nonlinear characteristics of the components are included in the model, the calculated forces at the target displacement should be accurate unlike in the linear procedures. The NSP model should include gravity loads on the components, should be discretized, and should include all primary and secondary lateral force resisting elements. A simplified version of the NSP is also allowed by FEMA 356 in which only primary elements are considered and the force-deformation properties of those elements are modeled as bilinear (FEMA 356- 2000).

The first step in the procedure is to designate a control node for the building. The standard states that this node should be at the center of mass at the roof of the structure. Lateral loads are then applied at diaphragm levels in proportion to the inertia forces in the structure. Two distributions should be considered for all NSP analyses: one that is proportional to the fundamental mode of the building or a story shear distribution and one that is either a uniform distribution or an adaptive load distribution that changes for nonlinear properties of the yielded structure.

The next step for the NSP is to generate nonlinear force-deformation relationships for each of the pier elements. A generalized force-deformation relationship is given in the standard and can be seen in Figure 7-3. These relationships are then used to develop a global force-displacement relationship for the building. An idealized bilinear curve is then fit over the actual building curve with the slope of the first section equal to an effective lateral stiffness, which is taken as the secant stiffness at 60% of the effective yield strength of the structure. This portion lasts until the effective yield strength of the building is reached. The second line has a slope of **a** which is a fraction of the effective lateral stiffness. This line ends when a target displacement is reached (FEMA 356-2000).

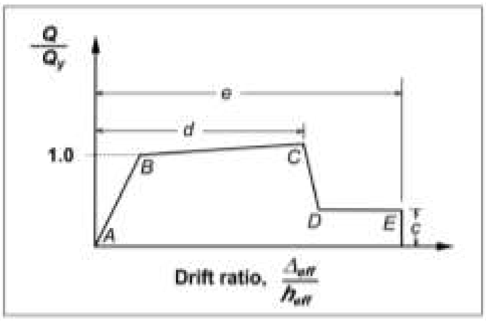


Figure 7-21 Generalized Force-Deformation Relationship for Deformation Controlled Masonry Elements or Components (FEMA 356-2000)

Once the idealized force-displacement relationship is determined, an effective fundamental period must be calculated for each orthogonal direction. The equation for this is as follows (FEMA 356-2000):

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.(7-17)$$

Where:

T_i = Elastic fundamental period calculated by elastic dynamic analysis

K_i = Elastic lateral stiffness

K_e = Effective lateral stiffness

FEMA 356 specifies an empirical formula to calculate the target displacement, d_t. For URM buildings with flexible diaphragms, this target displacement must be calculated for each line of vertical seismic resisting elements with masses calculated by tributary area. The equation is (FEMA 356-2000):

$$\delta_t = C_o C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots.(7-18)$$

Where:

C_o = Factor to relate spectral displacement of equivalent SDOF system to the control node of the actual MDOF building

C_1, C_2, C_3 = Same factors as LSP

T_e = Effective fundamental period

S_a = Response spectrum acceleration

g = Acceleration of gravity

The forces and deformations obtained through the analyses are then modified to consider the effects of horizontal torsion and then compared to the acceptance criteria found in the standard.

7.14.6 NONLINEAR DYNAMIC PROCEDURE (NDP)

The nonlinear dynamic procedure involves creating a finite element model of a building that incorporates the nonlinear load-deformation properties of individual components and then subjecting that model to a ground motion time history. The procedure is similar to that of the NSP with the exception that time histories are used instead of spectral accelerations.

7.14.6.1 Calculation Example of Performance based approach:

The same building used for analysis of strength based analysis in Annex Section 7.1 is analysed by performance based approach. Calculation is shown below:

i. Without Retrofitting

wall thickness (m)	0.3048				
Masonry Wt. (KN/m3):	18.8505		Modal DOF:	2	
Number of storey:	2		Component Properties:		
Masonry Condition :	fair			Lower bound	expected
			f'(KN/m2):	4136.85436	5377.91
			Em(KN/m2):	2275269.90	2957851
			Gm(KN/m2):	910107.961	1183140
	First Storey		Second storey		
story Height (m):	3.6576		story Height (m):	3.6576	
No. of opening:	3		No. of opening:	3	
No. of piers:	4		No. of piers:	4	

Pier one	Length(m):	1.524		Pier Five	Length(m):	1.524		
	h_{eff}(m):	0.2286			h_{eff}(m):	1.524		
	I_a(m^4)	0.08990			I_a(m^4)	0.08990		
	A_v(m^2)	0.46451			A_v(m^2)	0.46451		
		L.B	Exp			L.B	Exp	
	k(kN/m):	1832.84	2382.7		k(kN/m):	198.143	257.586	
Pier Two	Length(m):	1.2192		Pier Six	Length(m):	1.2192		
	h_{eff}(m):	1.524			h_{eff}(m):	1.524		
	I_a(m^4)	0.04603			I_a(m^4)	0.04603		
	A_v(m^2)	0.37161			A_v(m^2)	0.37161		
		L.B	Exp			L.B	Exp	
	k(kN/m):	136.566	177.53		k(kN/m):	136.566	177.536	
Pier Three	Length(m)	1.2192		Pier Seven	Length(m):	1.2192		
	h_{eff}(m):	1.524			h_{eff}(m):	1.524		
	I_a(m^4)	0.04603			I_a(m^4)	0.04603		
	A_v(m^2)	0.37161			A_v(m^2)	0.37161		
		L.B	Exp			L.B	Exp	
	k(kN/m):	136.566	177.53		k(kN/m):	136.566	177.536	
Pier Four	Length(m)	1.524		Pier Eight	Length(m):	1.524		
	h_{eff}(m):	1.524			h_{eff}(m):	1.524		
	I_a(m^4)	0.08990			I_a(m^4)	0.08990		
	A_v(m^2)	0.46451			A_v(m^2)	0.46451		
		L.B	Exp			L.B	Exp	
	k(kN/m):	198.143	257.58		k(kN/m):	198.143	257.586	
Total Story One stiffness (KN/m):		L.B	EXP	Total Story One stiffness (KN/m):		L.B	EXP	
		2304.12	2995.3			669.420	870.246	

wall thickness (m):	0.3048
Masonry Wt. (KN/m3):	18.85049568
Number of storie:	2
Masonry Condition :	fair
Running Bond:	yes

α:factor equal to 0.5 for fixed-free pier, 1.0 for fixed-fixed

Modal DOF: 2

Component Properties:

	Lower bound	expected
f'(KN/m2) :	4136.8	5377.91
Em(KN/m2):	2275269.	2957850.9
Gm(KN/m2):	910107.961	1183140.3
Vte(KN/m2):	137.8951456	179.26369

GRAVITY LOADS

First Floor

Height of masonry above(m):	0.381	Height of masonry above(m):	0.1016
Dead load from masonry(KN/m):	0.0579132	Dead load from maonry(KN/m):	0.015443
Dead load from building(KN/m):	0.0120652	Dead load from building(KN/m):	0.012065
live/snow load from building(KN/m):	0.0120652	live/snow load from building(KN/m):	0.012065
Qg (Factored additive)(KN/m):	0.0902482	Qg (Factored additive)(KN/m):	0.043531
Qg (Factored counteractive)(KN/m)	0.0629807	Qg (Factored counteractive)(KN/m)	0.024757

WALL THICKNESS

Expected Strength

First Storey					Second Storey				
Pier One:	Length(m)	0.127			Pier Five:	Length(m):	0.127		
	heff(m):	0.228				heff(m):	0.127		
	An(m^2)	0.038				An(m^2)	0.0387		
	Vte(KN/m2)	179.2				Vte(KN/m2):	179.26		
	Pce(KN):	0.011				Pce(KN):	0.0055		
	Vme(KN/m2)	67.37				Vme(KN/m2):	67.295		
	α:	1				α:	1		
	Q_{CE}:	V_{bjs}(KN):	2.60794			Q_{CE}:	V_{bjs},KN	2.6049739	
		V_r(KN)	0.005731	Control			V_r,KN	0.0049756	Controls!
Pier Two:	Length(m):	0.1016			Pier Six:	Length(m):	0.1016		
	heff(m):	0.127				heff(m):	0.127		
	An(m^2)	0.030968				An(m^2)	0.0309677		
	Vte(KN/m2):	179.2637				Vte(KN/m2):	179.26369		
	Pce(KN):	0.009169				Pce(KN):	0.0044228		
	Vme(KN/m2):	67.37193				Vme(KN/m2):	67.295293		
	α:	1				α:	1		
	Q_{CE}:	V_{bjs}(KN):	2.086352			Q_{CE}:	V_{bjs}(KN):	2.0839791	
		V_r(KN)	0.006602	Controls!			V_r(KN)	0.0031844	Controls!

Pier Three:	Length(m):	0.1016		
	heff(m):	0.127		
	An(m^2)	0.030968		
	Vte(KN/m2):	179.2637		
	Pce(KN):	0.009169		
	Vme(KN/m2):	67.37193		
	α:	1		
Q_{CE}:	V_{bjs}(KN):	2.086352		
	V_r(KN)	0.006602	Controls!	

Pier Three:	Length(m):	0.127		
	heff(m):	0.127		
	An(m^2)	0.03871		
	Vte(KN/m2):	179.2637		
	Pce(KN):	0.011462		
	Vme(KN/m2):	67.37193		
	α:	1		
Q_{CE}:	V_{bjs}(KN):	2.60794		
	V_r(KN)	0.010315	Controls!	
Exp. Story Strength (Q_{CE}):	V_{bjs}(KN):	9.388585		
	V_r(KN)	0.02925	Controls!	

Pier Seven:	Length(m):	0.1016	
	heff(m):	0.127	
	An(m^2)	0.0309677	
	Vte(KN/m2):	179.26369	
	Pce(KN):	0.0044228	
	Vme(KN/m2):	67.295293	
	α:	1	
Q_{CE}:	V_{bjs}(KN):	2.0839791	
	V_r(KN)	0.0031844	Controls!

Pier Eight:	Length(m):	0.127	
	heff(m):	0.127	
	An(m^2)	0.0387096	
	Vte(KN/m2):	179.26369	
	Pce(KN):	0.0055285	
	Vme(KN/m2):	67.295293	
	α:	1	
Q_{CE}:	V_{bjs}(KN):	2.6049739	
	V_r(KN)	0.0049756	Controls!
Exp. Story Strength (Q_{CE}):	V_{bjs}(KN):	9.377906	
	V_r(KN)	0.0163201	Controls!

wall thickness (m)	0.3048
Masonry Wt. (KN/m3)	18.8505
Number of storie:	2
Masonry Condition	fair
Running Bond?:	yes

Modal DOF: 2

Component Properties:

	Lower bound	expected
f'(KN/m2):	4136.854368	5377.911
Em(KN/m2):	2275269.902	2957851
Gm(KN/m2):	910107.961	1183140
Vte(KN/m2):	137.8951456	179.2637

Wall thickness

Lower Bound Strength

	First Storey						Second Storey				
Pier One:	Length(m):	1.52				Pier Five:	Length(m)	1.52			
	h_{eff}(m):	2.74					h_{eff}(m):	1.52			
	An(m^2):	0.46					An(m^2):	0.46			
	f'_a(KN/m2):	110.32					f'_a(KN/m2):	34.47			
	P_L(KN):	0.10					P_L(KN):	0.04			
	f'_{dt}(KN/m2):	67.37					f'_{dt}(KN/m2):	67.30			
	α:	1.00					α:	1.00			
	l/h_{eff}:	0.56					l/h_{eff}:	1.00			
	Q_{CL}:		Vdt(KN):	28.23553			Q_{CL}:		Vdt(KN):	38.44149	
			Vtc(KN):	0.05129	Controls!				Vtc(KN):	0.03728	Controls!
			P_{CL}(KN):	1306.70958					P_{CL}(KN):	1306.710	
Pier Two:	Length(m):	1.22		14.63		Pier Six:	Length(m)	1.22			
	h_{eff}(m):	1.52		18.29			h_{eff}(m):	1.52			
	An(m^2):	0.37					An(m^2):	0.37			
	f'_a(KN/m2):	110.32					f'_a(KN/m2):	34.47			
	P_L(KN):	0.08					P_L(KN):	0.03			
	f'_{dt}(KN/m2):	67.37					f'_{dt}(KN/m2):	67.30			
	α:	1.00					α:	1.00			
	l/h_{eff}:	0.80					l/h_{eff}:	0.80			
	Q_{CL}:		Vdt(KN):	32.52734			Q_{CL}:		Vdt(KN):	24.60255	
			Vtc(KN):	0.05909	Controls!				Vtc(KN):	0.02386	Controls!
			P_{CL}(KN):	1045.36766					P_{CL}(KN):	1045.368	
Pier Three	Length(m)	1.22				Pier Seven:	Length(m):	1.22			
	h_{eff}(m):	1.52					h_{eff}(m):	1.52			
	An(m^2):	0.37					An(m^2):	0.37			
	f'_a(KN/m2):	110.32					f'_a(KN/m2):	34.47			
	P_L(KN):	0.08					P_L(KN):	0.03			

	f'_{dt}(KN/m2):	67.37			f'_{dt}(KN/m2):	67.30	
	α:	1.00			α:	1.00	
	l/h_{eff}:	0.80			l/h_{eff}:	0.80	
Q_{CL}:	Vdt(KN)	32.52734	Controls!	Q_{CL}:	Vdt(KN):	24.60255	Controls!
	Vtc(KN)	0.05909			Vtc(KN):	0.02386	
	P_{CL}(KN):	1045.36766			P_{CL}(KN):	1045.368	
Pier Four:	Length(m):	1.52		Pier Eight:	Length(m):	1.52	
	h_{eff}(m):	1.52			h_{eff}(m):	1.52	
	An(m^2):	0.46			An(m^2):	0.46	
	f'_a(KN/m2):	110.32			f'_a(KN/m2):	34.47	
	P_L(KN):	0.10			P_L(KN):	0.04	
	f'_{dt}(KN/m2):	67.37			f'_{dt}(KN/m2):	67.30	
	α:	1.00			α:	1.00	
	l/h_{eff}:	1.00			l/h_{eff}:	1.00	
Q_{CL}:	Vdt(KN):	50.82396		Q_{CL}:	Vdt(KN):	38.44149	
	Vtc(KN):	0.09233	Controls!		Vtc(KN):	0.03728	Controls!
	P_{CL}(KN):	1306.70958			P_{CL}(KN):	1306.710	
L.B Story Strength (Q_{CL}):	V_{dt}(KN):	144.11417		L.B Story Strength (Q_{CL}):	V_{dt}(KN):	126.08808	
	V_{tc}(KN)	0.26180	Controls!		V_{tc}(KN)	0.12228	Controls!

Determine Period

Diaphragm Span(m):	9.144
Diaphragm Length(m):	9.144
Diaphragm Thickness (m):	0.0254
Diaphragm Mod. (KN/m2):	10342.1359
Diaphragm I (m^4)	1.61830778
Floor Dead Load (KN/m2):	0.00263895
Inertial Diaphragm Force(KN):	0.22064963
Max. Diaphragm Deflection(mm):	0.16016381

Approximate Period T(s): 0.11177109

Trib. Weight of Building (KN)
Floor one: 370.314448
Floor Two: 370.314448

Calculate Spectral Acceleration:			
BSE-1:		**BSE-2:**	
S_a	0.127	S_a	0.448

Calculate Pseudo Lateral Load:			
BSE-1		**BSE-2**	
Factors:		Factors:	
C_1	1.00	C_1	1.01
C_2	1.00	C_2	1.00
C_3	1.00	C_3	1.00
C_m	1.00	C_m	1.00
Pseudo Lateral Load (KN)		**Pseudo Lateral Load (KN)**	
Floor One(KN):	47.02993492	Floor One(KN):	167.5598815
Floor Two(KN):	47.02993492	Floor Two(KN):	167.5598815

Design Forces			
BSE-1		**BSE-2**	
Deformation Controlled		Deformation Controlled	
Story Shear:		**Story Shear:**	
Q_{ud}:Floor Two (KN):	47.02993492	Q_{ud}:Floor Two (KN):	167.5598815
Q_{ud}:Floor One (KN):	94.05986984	Q_{ud}:Floor One (KN):	335.119763
Force Controlled		Force Controlled	
J Factor	1.0	J Factor	1.6
Story Shear:		**Story Shear:**	
Q_{UF}:Floor Two (KN):	47.02993492	Q_{UF}:Floor Two (KN):	104.724926
Q_{UF}:Floor One (KN):	94.05986984	Q_{UF}:Floor One (KN):	209.4498519

Accceptance Criteria					
BSE-1			**BSE-2**		
Deformation Controlled			Deformation Controlled		
Limit State:	Rocking		**Limit State:**	Rocking	
Performance Level:	IO		Performance Level:	LS	
Knowledge Factor (k):	0.75		Knowledge Factor (k):	0.75	
m factor:	1		m factor:	3	
mkQ_{CE}			mkQ_{CE}		
Floor Two(KN):	62.2751024	GOOD!	Floor Two(KN):	199.8585965	GOOD!
Floor One(KN):	154.5312184	GOOD!	Floor One(KN):	463.5936552	GOOD!
Force Controlled			Force Controlled		
Limit state	Toe Crushing		**Limit state**	Toe Crushing	
Knowledge factor(k):	0.75		Knowledge factor(k):	0.75	
kQ_{CL}:			kQ_{CL}:		
Floor Two(KN):	41.54638974	NO GOOD!	Floor Two(KN):	41.54638974	NO GOOD!
Floor One(KN):	108.6700537	GOOD!	Floor One(KN):	108.6700537	NO GOOD!

7.14.6.2 Analysis by Steel retrofitting

Determine Period

Diaphragm Span(m):	9.144
Diaphragm Length(m):	9.144
Diaphragm Thickness (m):	0.0254
Diaphragm Mod. (KN/m2):	10342.13592
Diaphragm I (m^4)	1.618307783
Floor Dead Load (KN/m2):	0.002638948
Inertial Diaphragm Force(KN):	0.220649625
Max. Diaphragm Deflection(mm):	0.160163806

Approximate Period T(s): 0.111771091

Trib. Weight of Building (KN)

Floor one: 370.3144482

Floor Two: 370.3144482

Calculate Spectral Acceleration:			
BSE-1:		BSE-2:	
S_a	0.127	S_a	0.448

Calculate Pseudo Lateral Load:			
BSE-1		BSE-2	
Factors:		Factors:	
C_1	1.00	C_1	1.01
C_2	1.00	C_2	1.00
C_3	1.00	C_3	1.00
C_m	1.00	C_m	1.00
Pseudo Lateral Load (KN)		**Pseudo Lateral Load (KN)**	
Floor One(KN):	47.02993492	Floor One(KN):	167.5598815
Floor Two(KN):	47.02993492	Floor Two(KN):	167.5598815

Design Forces			
BSE-1		BSE-2	
Deformation Controlled		Deformation Controlled	
Story Shear:		**Story Shear:**	
Q_{ud}:Floor Two (KN):	47.02993492	Q_{ud}:Floor Two (KN):	167.5598815
Q_{ud}:Floor One (KN):	94.05986984	Q_{ud}:Floor One (KN):	335.119763
Force Controlled		Force Controlled	
J Factor	1.0	J Factor	1.6
Story Shear:		**Story Shear:**	
Q_{UF}:Floor Two (KN):	47.02993492	Q_{UF}:Floor Two (KN):	104.724926
Q_{UF}:Floor One (KN):	94.05986984	Q_{UF}:Floor One (KN):	209.4498519

Accceptance Criteria		
BSE-1		
Deformation Controlled		_
Limit State:	Rocking	

Performance Level:	IO	
Knowledge Factor (k):	0.75	
m factor:	1	
mkQ_{CE}		
Floor Two(KN):	473.1128494	GOOD!
Floor One(KN):	461.2805799	GOOD!
Force Controlled		
Limit state	Toe Crushing	
Knowledge factor(k):	0.75	
kQ_{CL}:		
Floor Two(KN):	858.8181443	GOOD!
Floor One(KN):	881.7264856	GOOD!

BSE-2		
Deformation Controlled		
Limit State:	Rocking	
Performance Level:	LS	
Knowledge Factor (k):	0.75	
m factor:	3	
mkQ_{CE}		
Floor Two(KN):	1419.338548	GOOD!
Floor One(KN):	1383.797258	GOOD!
Force Controlled		
Limit state	Toe Crushing	
Knowledge factor(k):	0.75	
kQ_{CL}:		
Floor Two(KN):	858.8181443	GOOD!
Floor One(KN):	881.7264856	GOOD!

7.14.7 COMPUTER AIDED ANALYSIS

A local building in Kathmandu was chosen for the Analysis. The plan and elevation of the building is shown in figure 7 – 4.

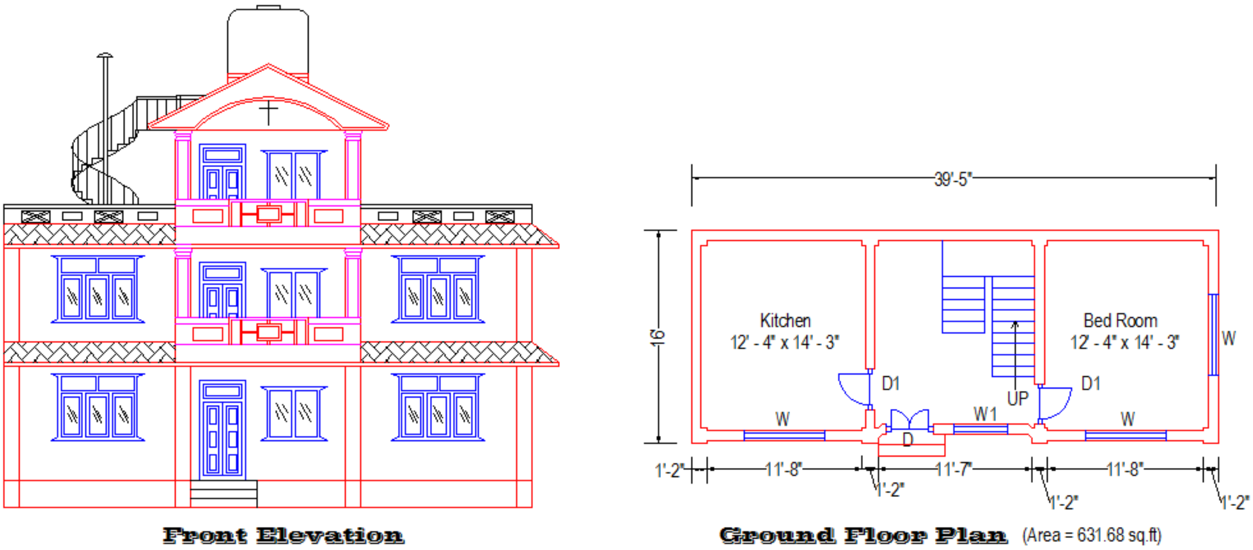


Figure 7-22 Elevation and Plan of Local Building.

The vertical structure is a single layer non retrofitted masonry made of brick masonry, while the floors is of concrete slabs. In order to reduce the computational burden of the dynamic analyses needed for the vulnerability assessment, only the facade wall was analysed using the proposed SAP2000 v.14.0.0 model. The design values assumed for the mechanical properties are based on the mean values measured in brick masonry from different researches conducted in Pulchowk Campus, for brick masonry, f_d=1.82N/mm2, E=509N/mm2, and G=203.6N/mm2. Only the in-plane seismic performance of the wall was investigated, assuming that the wall was effectively connected to the floors.

Laurent Pasticier, Claudio Amadio and Massimo Fragiacomao (July 2007) carried out Non Linear Push over Analysis of masonry structure using SAP 2000 v.10. The study has established that the equivalent frame method for the masonry could be adapted for Non - Linear Push over Analysis, by providing different hinges at the different section in the members.

The masonry pier was modeled as elastoplastic. Hinges were determined according to the failure mechanisms of masonry. The standard force-displacement curve that can be implemented in the SAP 2000 plastic hinges in depicted in Figure 7-5(a) . The masonry piers

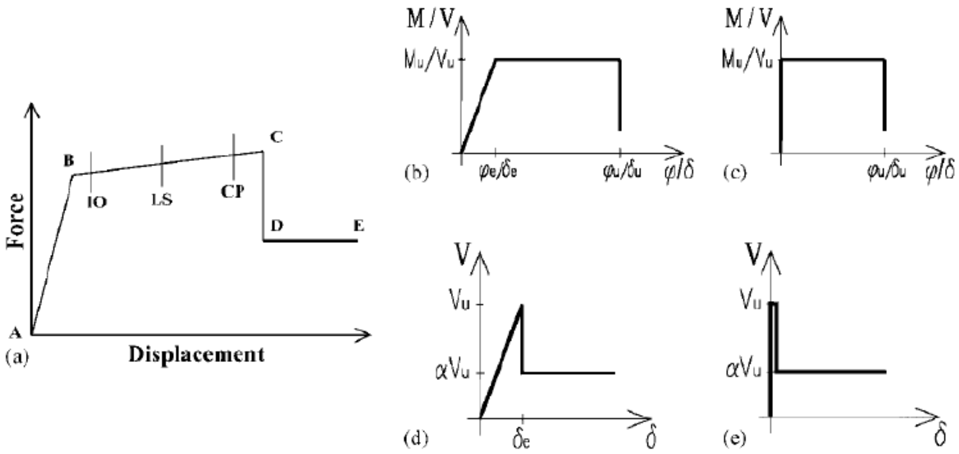


Figure 7-23 (a) Standard shape of the force vs displacement curve in SAP2000® v.10 for the plastic hinge element ; (b) and (c): behavior assumed, respectively, for the entire pier and the correspondent plastic hinge; (d) and (e): behavior assumed, respectively, for the entire spandrel beam and the correspondent plastic hinge.

were modeled as elastoplastic with final brittle failure (Figure 7-5 (b)) by introducing two 'rocking hinges' at the end of the deformable parts and one ‘shear hinge’ at mid-height . A rigid- perfectly plastic behavior with final brittle failure was assumed for all these plastic hinges (Figure 7-5(c)).

7.14.7.1 Pushover Analysis

i. *Without retrofitting*

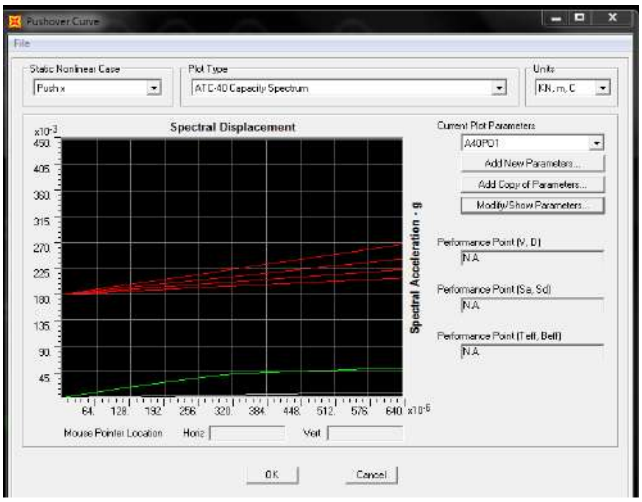


Figure 7-24 Push Over Curve of Local Building without retrofitting, from SAP 2000 v 14.0.0 Analysis

As the model is assumed to be without lintel, both the shear hinges (at the middle) and flexural hinges (at the ends) are assigned in the spandrels. Base Shear calculated here for the building is 88.29 KN, but the structure seems to be collapsed at 25.553 KN base shear. No performance point is found for DBE (Ca = 0.18, Cv = 0.3). Pushover curve is shown in figure 7-6.

ii. *Retrofitting with lintel*

Only shear hinge is assigned in spandrels at the mid span while piers are modeled with two flexural hinges at the ends and one shear hinge at the mid span. Slight in base shear capacity was found but performance point was not found. Pushover curve is shown in figure 7-7.

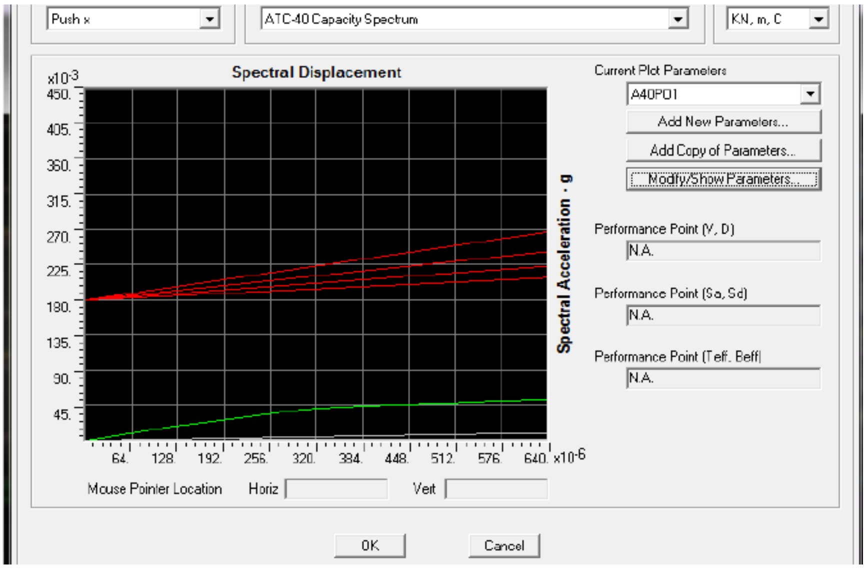


Figure 7-25 Push Over Curve of Local Building with lintel, from SAP 2000 v 14.0.0 Analysis

iii. *Retrofitting with lintel + Rigid Diaphragm*

Rigid diaphragm was assigned at the floor level but no increase in base shear capacity in found in in-plane analysis. No performance point was found. Pushover curve is shown in figure 7-8.

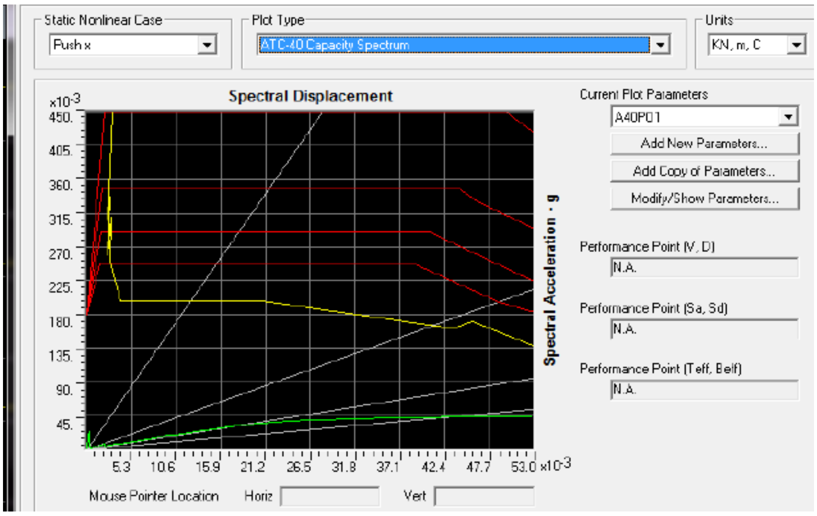


Figure 7-26 Push Over Curve of Local Building with lintel and rigid diaphragm, from SAP 2000 v 14.0.0 Analysis

iv. *Retrofitting with lintel + rigid diaphragm + Columns*

Assigning five RCC columns of size 230mm x 230mm the base shear capacity of the structure was as increased but masonry wall fails far before the RCC columns fails. But structure as a whole has large base shear capacity. Performance point of the structure with Ca = 0.18 and Cv = 0.3 is found to be (V, D) = (230.53 KN, 0.027m). Pushover curve is shown in figure 7-9.

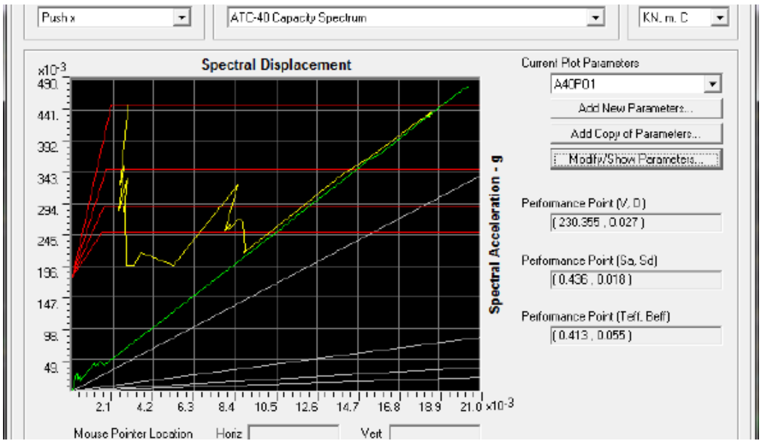


Figure 7-27 Push Over Curve of Local Building with lintel, rigid diaphragm and columns, from SAP 2000 v 14.0.0 Analysis

v. *Retrofitting with lintel + rigid diaphragm + wire meshing*

Rebar used; 16mm bar as vertical bars @ 150 mm c/c, 8mm bar as horizontal bars @ 150 mm c/c.

base shear = 88.289 KN, (Z = 0.36, I = 1, Sa/g =2.5, R =3)

Performance point of the structure with DBE (Ca = 0.18, Cv = 0.3) is (107 KN, 0.016m)

Although the base shear capacity of the structure is lesser than while assigning RC columns, this option seems to be more reliable as in this case the masonry structure is covered with wire mesh which enable the masonry members to remain stable without the formation of the hinges approximately upto the performance point. Pushover curve is shown in figure 7-10.

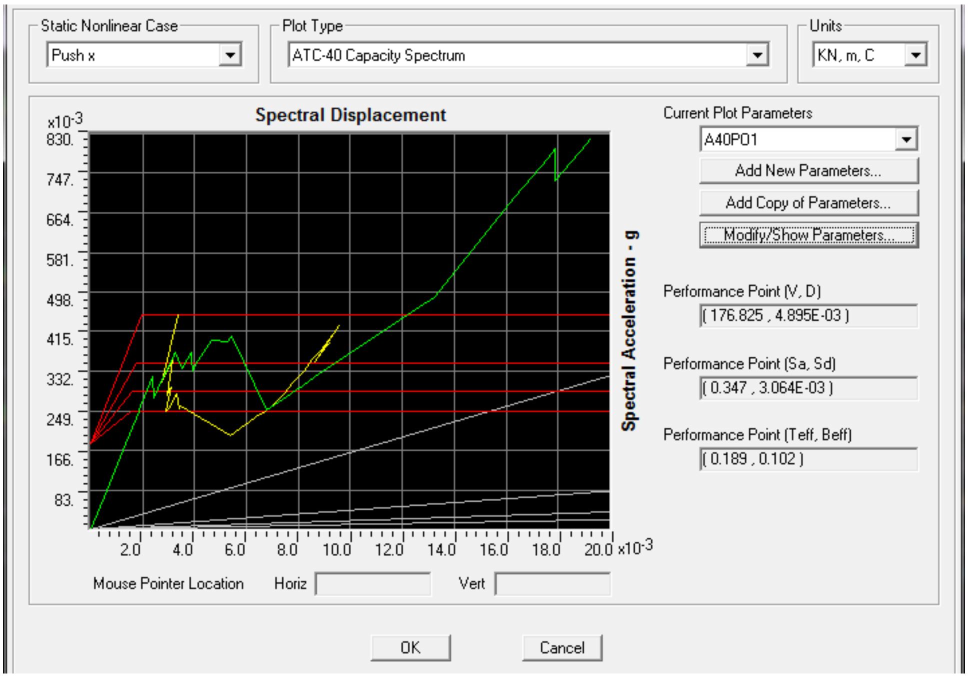
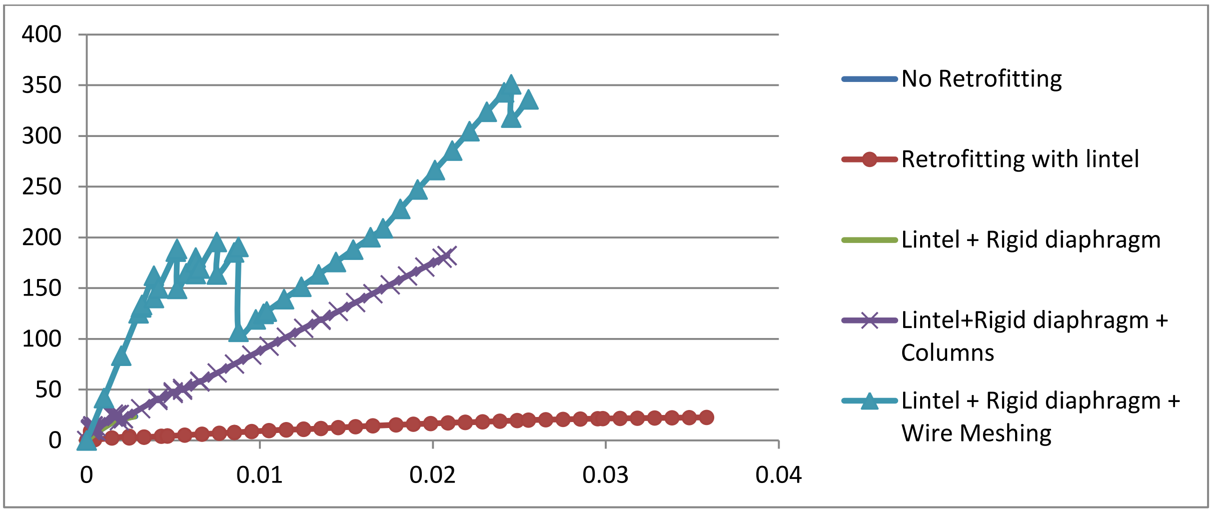


Figure 7-28 Push Over Curve of Local Building with lintel, rigid diaphragm and meshing, from SAP 2000 v 14.0.0 Analysis

Table 7-5 Comparison table for different retrofitting options



7.15 CONCLUSION FROM IN PLANE ANALYSIS

Retrofitting techniques adapted	**Model**	**Result**	**Remark**
NO retrofitting	Existing (without retrofit)	Fails before DBE	Building fails in DBE
Retrofitting with lintel	Lintel assigned on the openings	No improvement in the base shear capacity	Building fails in DBE
Retrofitting with lintel + rigid diaphragm	Existing (with rigid diaphragm)	No improvement in the base shear capacity	Building fails in DBE
Retrofitting with lintel + rigid diaphragm + Columns	Columns are inserted in the walls	Increase in base shear capacity	Performance point of the building increases, and as a whole structure remain stable but masonry walls fails too earlier although the whole structure is stable, so this technique is not so good
Retrofitting with lintel + rigid diaphragm + Wire meshing	Jacketing is done by wire meshing	Increase in base shear capacity	As each wall unit has been strengthened, wall does not fail, so this technique seems to be most reliable of all above

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