SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL

2013

ADOBE

CORD

MRB & Associates

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LIST OF FIGURES

LIST OF TABLES

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CONTENTS

1. INTRODUCTION

Nepal is one of the hotspots for disaster and is ranked as 11th most vulnerable countries in the world for earthquake. As Nepal lies in the seismic prone area with frequent occurrence of devastating earthquakes, the buildings need to be designed and constructed against seismic safety. On the contrary, the structures built in Nepal are not just seismically unsafe, but not even engineered to meet the basic building codes. In this case of haphazard growth of non-engineered buildings standing together with the old and withered structure, the settlements in Nepal and basically the city cores are extremely vulnerable to earthquake, as witnessed during the previous quakes.

Many of the early structures in Nepal were built of adobe. Earth, as a building material, has been used since ancient times, and is still being used in many part of the country, if not in the city areas. The materials available for construction of early monuments, temples, palace and residential buildings were generally limited to those that were readily available and easily worked by local artisans. Because earth's intensive use during past centuries, currently, there is a great architectural heritage stock and an equally large stock of vulnerable buildings. As a consequence of their age, design and the functions they performed, surviving historic adobe structures are among the most historically and culturally significant structures in their communities and should be preserved for future generation.

The structural damage in this kind of structures manifests, in general, in the form of cracks or voids, caused by drying shrinkage, thermal movements, foundations settlements, plant growth and earthquakes, the last one having devastating consequences. Repairing those cracks is fundamental in order to obtain an improved structural behavior, especially when earth construction was built in a seismic zone like Nepal.

It is neither practical nor feasible to demolish all these 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 code 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.

This guideline is for assisting professionals and authorities in Nepal to retrofit the existing adobe (Low Strength Masonry) public and private buildings in Nepal. The guideline is based on the experiences gained in Nepal in retrofitting as well as on the adaptation of different techniques used in other countries from literatures survey. It includes the buildings typology – adobe (earthen sun-dried bricks) with mud mortar, fired bricks in mud mortar and stone masonry buildings.

1.1 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 adobe buildings in Nepal. It is expected that this document will be used by retrofit design professionals for performing seismic evaluations and retrofit designs.

1.2 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

In addition, these guidelines can assist responsible parties in the planning of seismic retrofitting projects that are consistent with both conservation principles and established public policy; they can help local officials establish parameters for evaluating submitted retrofitting proposals; and they can serve as a resource for technical information and issues to be considered in the design of structural modifications to historic adobe buildings.

1.3 CONCEPT OF REPAIR, RESTORATION AND RETROFITTING[1]

1.3.1 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:

[1] Adapted from IAEE Manual

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 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.

1.3.2 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.

1.3.3 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 economical reasons, generally substituting these weak structures with new earthquake resistant buildings is neglected. This guideline focuses on the seismic retrofitting of adobe structures for sustaining design utilities.

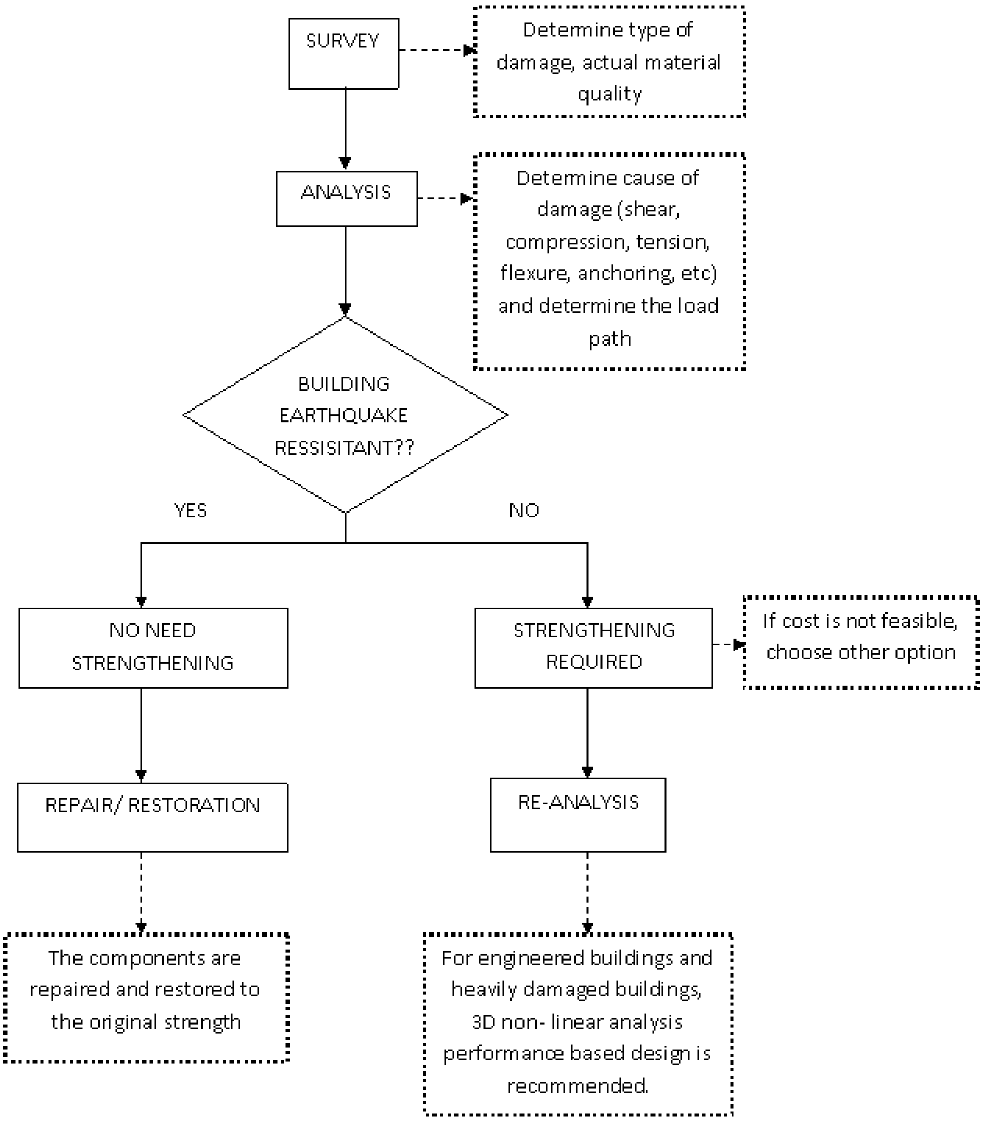


Figure 1-1 Stepwise Process of Seismic Retrofitting Of Building

2. DAMAGE CATEGORIZATION AND USUAL DAMAGE TYPOLOGY

Apart from low cost, simple construction technology, excellent thermal and acoustic properties, adobe structures are vulnerable to the effects of natural phenomena such as earthquakes, rain, and floods. Traditional adobe construction responds very poorly to earthquake ground shaking, suffering serious structural damage or collapse, and causing a significant loss of life and property. Seismic deficiencies of adobe construction are caused by the heavy weight of the structures, their low strength, and brittle behavior. During strong earthquakes, due to their heavy weight, these structures develop high levels of seismic forces that they are unable to resist, and therefore fail abruptly.

The studies on past earthquakes confirmed the considerable damage of adobe buildings and loss of life. In the 2001 earthquakes in El Salvador, more than 200,000 adobe buildings were severely damaged or collapsed, 1,100 people died under the rubble of these buildings, and over 1,000,000 people were made homeless (USAID El Salvador 2001). That same year, the earthquake in the south of Peru caused the death of 81 people, the destruction of almost 25,000 adobe houses and the damage of another 36,000 houses, with the result that more than 220,000 people were left without shelter. (USAID Peru 2001). Adobe buildings were also damaged in the rural areas affected by the 2008 Wenchuan, China earthquake (EERI 2008) and the 2010 Maule, Chile, earthquake (Astroza et al. 2010).

According to MoHA, the recent earthquake that hit Eastern Nepal on 18 September 2011 left 8,792 buildings severely damaged, most of which were adobe buildings. The same earthquake was also responsible for affecting more than 22,000 buildings for partial damages *(source: www.ekantipur.com).*

The seismic damage categorization for adobe construction and its mode of failures are summarized below.

(Source: www.earthquake.usgs.gov)

Figure 2-1 Severe damage to adobe buildings in Chorrillos district in Peru earthquake 1974

(Source: irapl.altervista.org, figure 69-B, U.S. Geological Survey Professional paper 1002)

Figure 2-2 Damage of adobe houses in Gautemala City during Guatemala Earthquake 1976

(Source:http://www.worldhousing.net/whereport1view.php?ID=100130)

Figure 2-3 Collapsed adobe structures 2003 Bam Earthquake

Figure 2-4 Collapsed structure (source: CoRD)

2.1 DAMAGE CATEGORATION

The damage categorizations based on the European Macroseismic Scale (EMS- 98) define building damage to be in Grade 1 to Grade 5. The damage classifications help in evaluation of earthquake intensity following an earthquake.

Table 2-1 Damage Categorization

S.No.	Damage Grade	Wall
1	Grade 1: Negligible to slight damage	No structural damage, slight non- structural damage • Hair line cracks in very few walls. • Fall of small pieces of plaster only. • Fall of loose stones from upper parts of buildings in very few cases
2	Grade 2: Moderate damage	Slight structural damage, moderate non-structural damage • Cracks in many walls.

		• Fall of fairly large pieces of plaster. • Partial collapse of chimneys.
3	Grade 3: Substantial to heavy damage	Moderate structural damage, heavy non- structural damage • Large and extensive cracks in most walls. • Roof tiles detach, chimney fracture at the roof line • Failure of individual non structural elements (partitions, gable walls, etc).
4	Grade 4: Very heavy damage	Heavy structural damage, very heavy non-structural damage • Serious failure of walls (gaps in walls) • Partial structural failure of roof and floors
5	Grade 5: Destruction	Very heavy structural damage • Total or near total collapse of the building

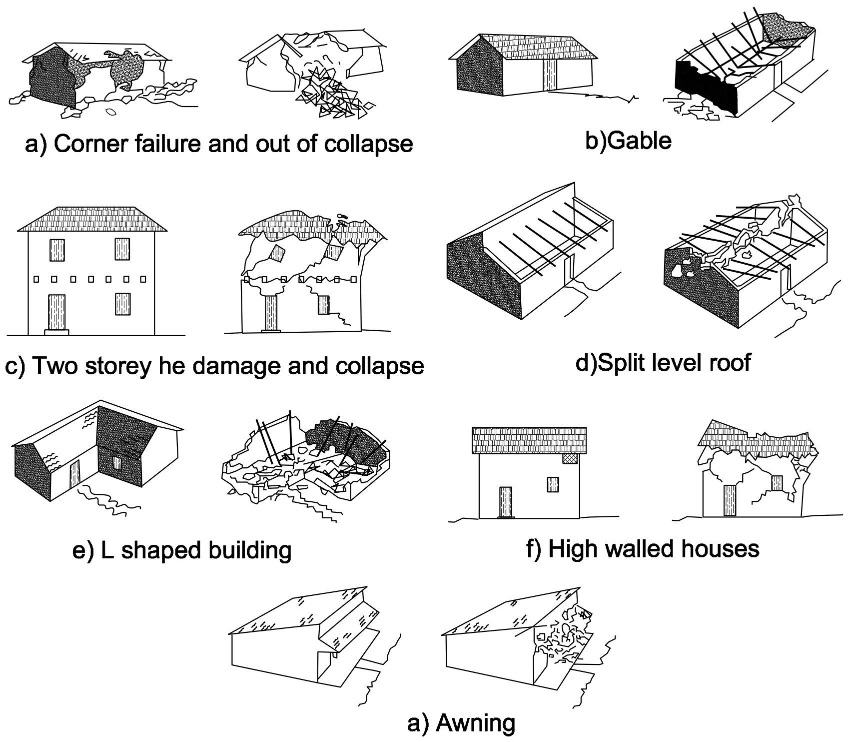
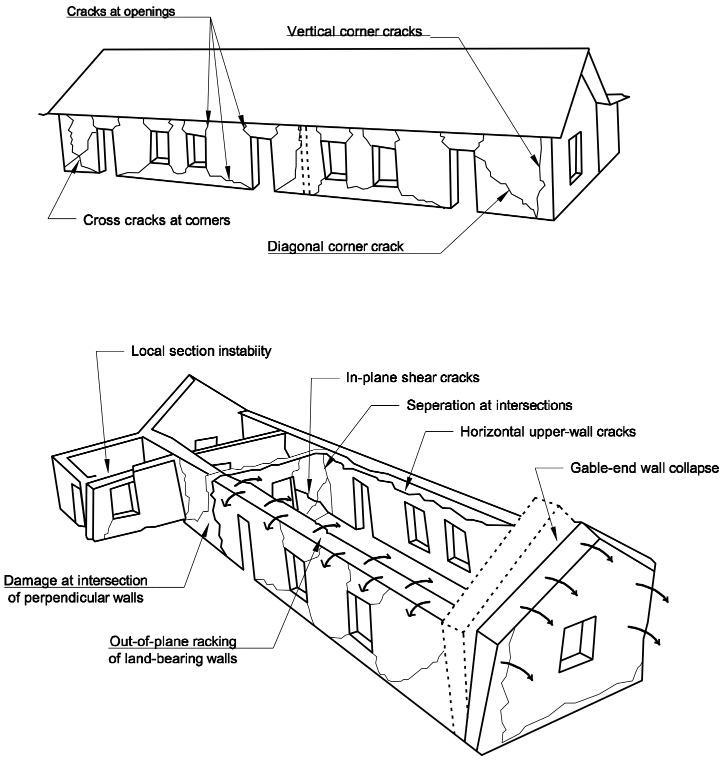


Figure 2-5 Damage Typology (Source Arya A. S et al, 2012)

2.2 DAMAGE TYPOLOGY

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

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.



(source: Manual of The Getty Conservation Institute)

Figure 2-6 Typical damage modes observed in adobe buildings

2.2.1 OUT OF PLANE WALL DAMAGE

Adobe walls are very susceptible to cracking from flexural stresses caused by out-of-plane ground motions. These cracks are usually occurring in a wall between two transverse walls. The cracks often start at each intersection, extend downward vertically or diagonally to the base of the wall, and then extend horizontally along its length. The wall rocks back and forth out of plane, rotating about the horizontal crack at the base. Cracks due to out-of-plane motions are typically the first type of damage to develop in adobe buildings. Out-of-plane cracks develop in an undamaged adobe wall when peak ground accelerations reach approximately 0.2g.

Although wall cracks that result from out-of-plane forces occur readily, the extent of damage is often not particularly severe, as long as the wall is prevented from overturning. The principal factors that affect the out-of-plane stability of adobe walls are as follows:

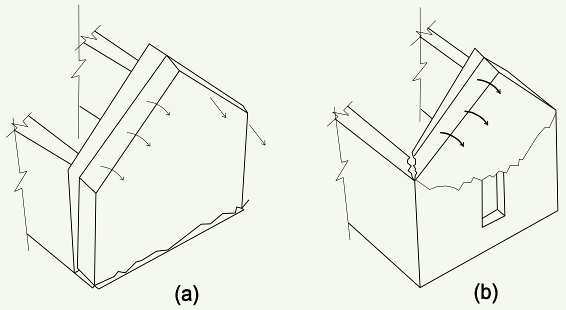
- Wall thickness and the slenderness ratio (SL)
- The connection between the walls and the roof and/ or floor system
- Whether the wall is load bearing or non load bearing
- The distance between intersecting walls and
- The condition of the base of the wall

(Source: Report #52 in EERI/ IAEE World Housing Encyclopedia)
Figure 2-7 Out-of-plane wall collapse – 1996 Nazca earthquake, Peru

(*photo: M. Blondet)*
Figure 2-8 Out-of-plane wall collapse after the 2007 Pisco, Peru earthquake

2.2.1.1 Gable End wall Collapse

Gable end wall damage is a special case of out-of-plane failure that needs specific discussion as these walls are very susceptible to damage in adobe buildings. Gable-end walls are tall and thin, non-load-bearing, and usually not well connected to the structure at the floor, attic, or roof level. Their overturning is caused by ground motions that are perpendicular (out of plane) to the walls. Instability problems can also result from in-plane ground motions when sections of the wall slip along diagonal cracks and then become unstable out of plane, especially at corners.



. *(source: Manual of The Getty Conservation Institute)*

Figure 2-9 Gable-end wall collapse: (a) overturning at base of wall, and (b) mid-height collapse

Figure 2-10 Gable end wall mid-height collapse (Sinam, Eastern Nepal Earthquake, September 1st, 2011)

Figure 2-11 Some other examples of Gable wall damage during Eastern Nepal Earthquake, 2011 (Photo: CoRD)

2.2.1.2 Out of plane flexural cracks and collapse

Out-of-plane flexural cracking is one of the first crack types to appear in an adobe building during a seismic event. This damage type and the associated rocking motion are illustrated in Figure 2-12. 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.

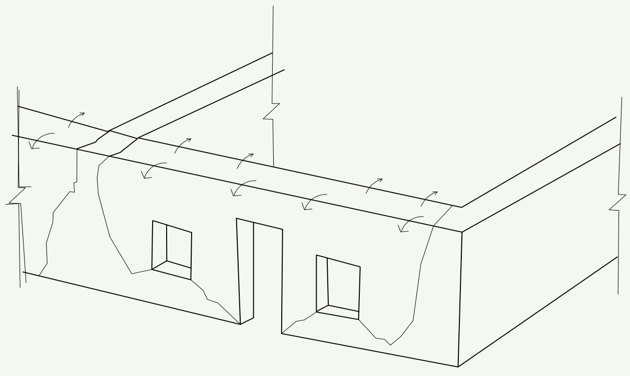


Figure 2-12 Out-of-plane flexure of load bearing wall (source: Manual of The Getty Conservation Institute)

2.2.1.3 Mid height out-of-plane flexural damage

For the most part, historic adobe buildings are not susceptible to mid-height, out-of-plane flexural damage because the walls are usually thick and have small slenderness ratios. However, horizontal cracks may develop when load-bearing walls are long and the top of the wall is restrained by a bond beam or a connection to a roof or ceiling system (Figures 2-13 and 2-14). This type of damage and potential failure mechanism is usually observed only in thin-walled (SL. 8) masonry buildings.

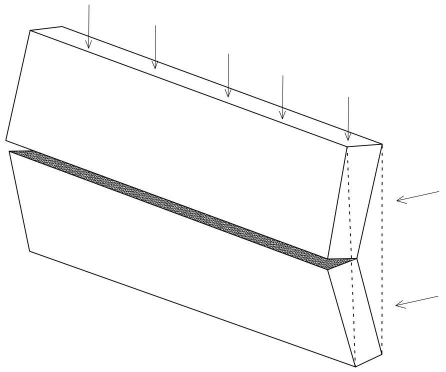


Figure 2-13 Sketch of mid-height out of plane failure (source: Manual of The Getty Conservation Institute)

Figure 2-14 Mid-height Crack (photo:CoRD)

2.2.2 IN-PLANE SHEAR CRACKS

Diagonal cracks (Figures 2-15a, b) 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 (Figure 2-15c). 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.

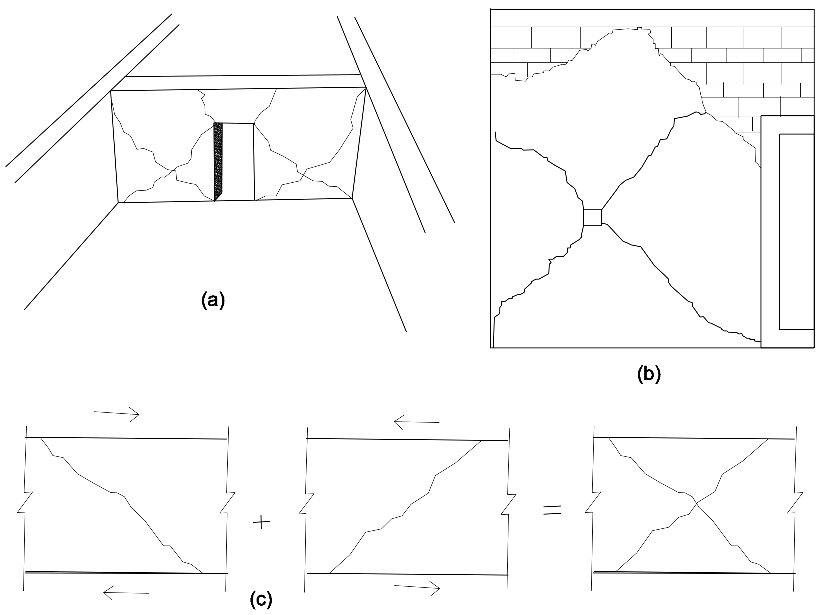


Figure 2-15 Illustrations show (a) drawing of X-shaped shear cracks in an interior wall; (b) typical X pattern (Leonis Adobe, Calabasas, Calif.); and (c) how X-shaped cracks result from a combination of shear cracks caused by alternate ground motions in opposite directions. (source: Manual of The Getty Conservation Institute)

However, while in-plane shear is not considered hazardous 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.

2.2.3 CORNER DAMAGE

Damage often occurs at the corners of buildings due to the stress concentrations that occur at the intersection of perpendicular walls. Instability of corner sections often occurs because the two walls at the corner are unrestrained and therefore the corner section is free to collapse outward and away from the building.

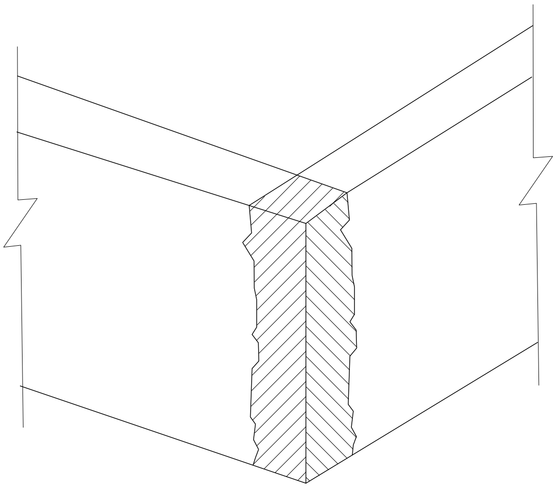


Figure 2-16 Illustrations showing (a) how vertical cracks at corner can lead to instability of intersection (source: Manual of The Getty Conservation Institute)

Figure 2-17 Corner damage during Eastern Nepal Earthquake, 2011 (source: CoRD)

Figure 2-18 Some other examples of corner damages (Source: CoRD)

2.2.3.1 Vertical cracks at corners

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 (Figure 2- 19).

Source: Photos by Hari. D. Shrestha Other than stated

Figure 2-19 Vertical cracking and separation of adobe walls after the 1997 Jabalpur, India earthquake (source: Kumar 2002)

2.2.3.2 Diagonal 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. Illustrations of this damage type are shown in Figure 2-20.

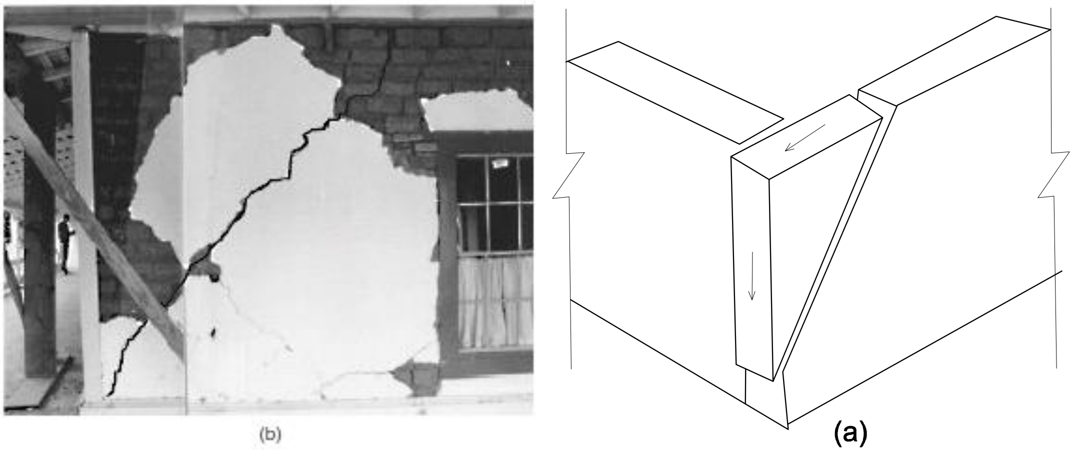


Figure 2-20 Corner cracks: (a) illustration of vertical downward and horizontal displacement of a corner wall section, and (b) example of displaced wall section (Leonis Adobe) (source: Manual of The Getty Conservation Institute)

2.2.4 COMBINATIONS WITH OTHER CRACKS OR PREEXISTING DAMAGE

A combination of diagonal and vertical cracks can result in an adobe wall that is severely fractured, and several sections of the wall may be susceptible to large offsets or collapse. An example of a wall section that is highly vulnerable to serious damage is illustrated in Figure

2-21. The diagonal cracking at that location allows the cracked wall sections freedom to move outward. Corners may be more susceptible to collapse if vertical cracks develop and the base of the wall has already been weakened by previous moisture damage.

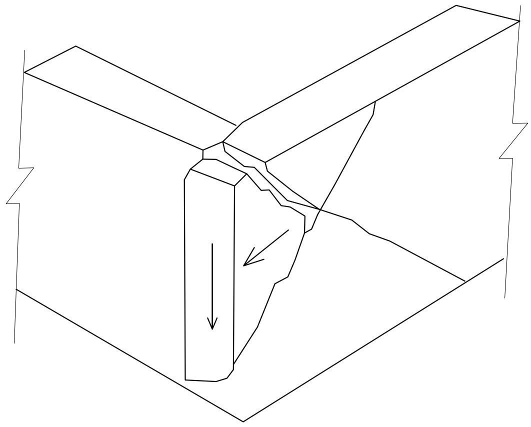


Figure 2-21 Illustration showing how combination of shear and flexural cracks can result in corner displacement or collapse (source: Manual of The Getty Conservation Institute)

2.2.5 CRACKS AT OPENINGS

Cracks occur at window and door openings more often than at any other location in a building. In addition to earthquakes, foundation settlement and slumping due to moisture intrusion at the base can also cause cracking. Cracks at openings develop because stress concentrations are high at these locations and because of the physical incompatibility of the adobe and the wood lintels. Cracks start at the top or bottom corners of openings and extend diagonally or vertically to the tops of the walls, as illustrated in Figure 2-22 and 2-24.

Cracks at openings are not necessarily indicative of severe damage. Wall sections on either side of openings usually prevent these cracks from developing into large offsets. However, in some cases, these cracks result in small cracked wall sections over the openings that can become dislodged and could represent a life-safety hazard.

2.2.6 INTERSECTION OF PERPENDICULAR WALLS

Damage often occurs at the intersection of perpendicular walls. One wall can rock out of plane while the perpendicular in-plane wall remains very stiff. Damage at these locations is inevitable during large ground motions and can result in the development of gaps between the in-plane and out-of- plane walls (Figure 2-23a) or in vertical cracks in the out-of-plane wall (Figure 23-b). Damage may be significant when large cracks form and associated damage occurs to the roof or ceiling framing. Anchorage to the horizontal framing system or other continuity elements can greatly reduce the severity of this type of damage.

Damage at the intersection of perpendicular walls is normally not serious from a life-safety perspective. However, in the same way that corner damage occurs, adjacent walls can become isolated and behave as freestanding walls. When they reach this state, the possibility of collapse or overturning is greatly increased, and a serious life-safety threat can arise. In addition, if significant permanent offsets occur, repair may be difficult and expensive.

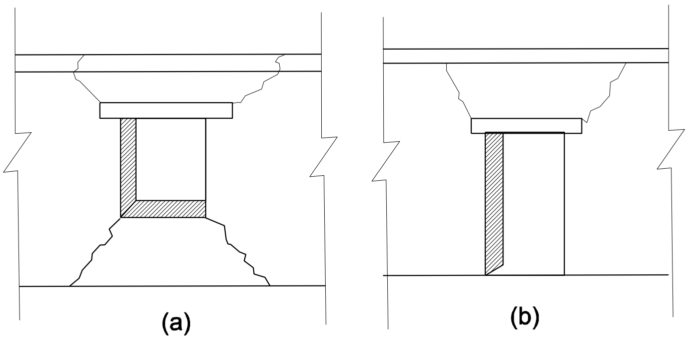


Figure 2-22 Illustration of cracks originating at stress concentration locations: (a) cracks appearing first at upper corners of window opening followed by lower corner cracks; and (b) cracks at upper corners of door opening. (source: Manual of The Getty Conservation Institute)

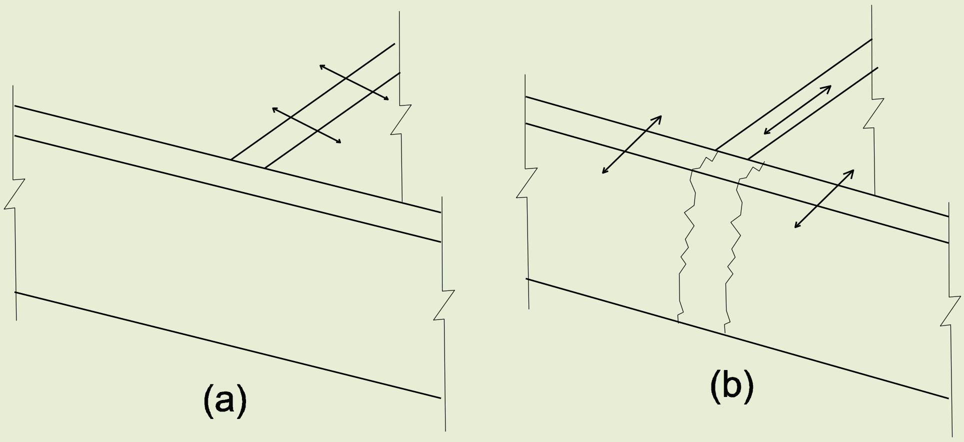


Figure 2-23 Illustrations showing (a) how separation can occur between in-plane and out-of-plane walls, and (b) how vertical cracks develop in out-of-plane walls at the intersection with perpendicular, in-plane walls. (source: Manual of The Getty Conservation Institute)

Figure 2-24 Some examples of Cracks at opening (source: CoRD)

3. ASSESSMENT OF DAMAGE/VULNERABILITY

UNDP and Government of Nepal have already developed the guidelines "*Seismic Vulnerability Evaluation Guideline for Private and Public Buildings*'. The vulnerability assessment of adobe buildings can be performed as described in the guidelines. In addition refer ANNEX I for detail assessment.

4. RETROFITTING TECHNIQUES FOR DIFFERENT ELEMENTS

4.1 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:[2]

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.
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

[2] Adapted from IAEE Manual

regular. The different forms of recommended geometrical configurations are illustrated in Figure 4-1.

vi. Openings in load bearing walls should be restricted as shown in figure 4-2.

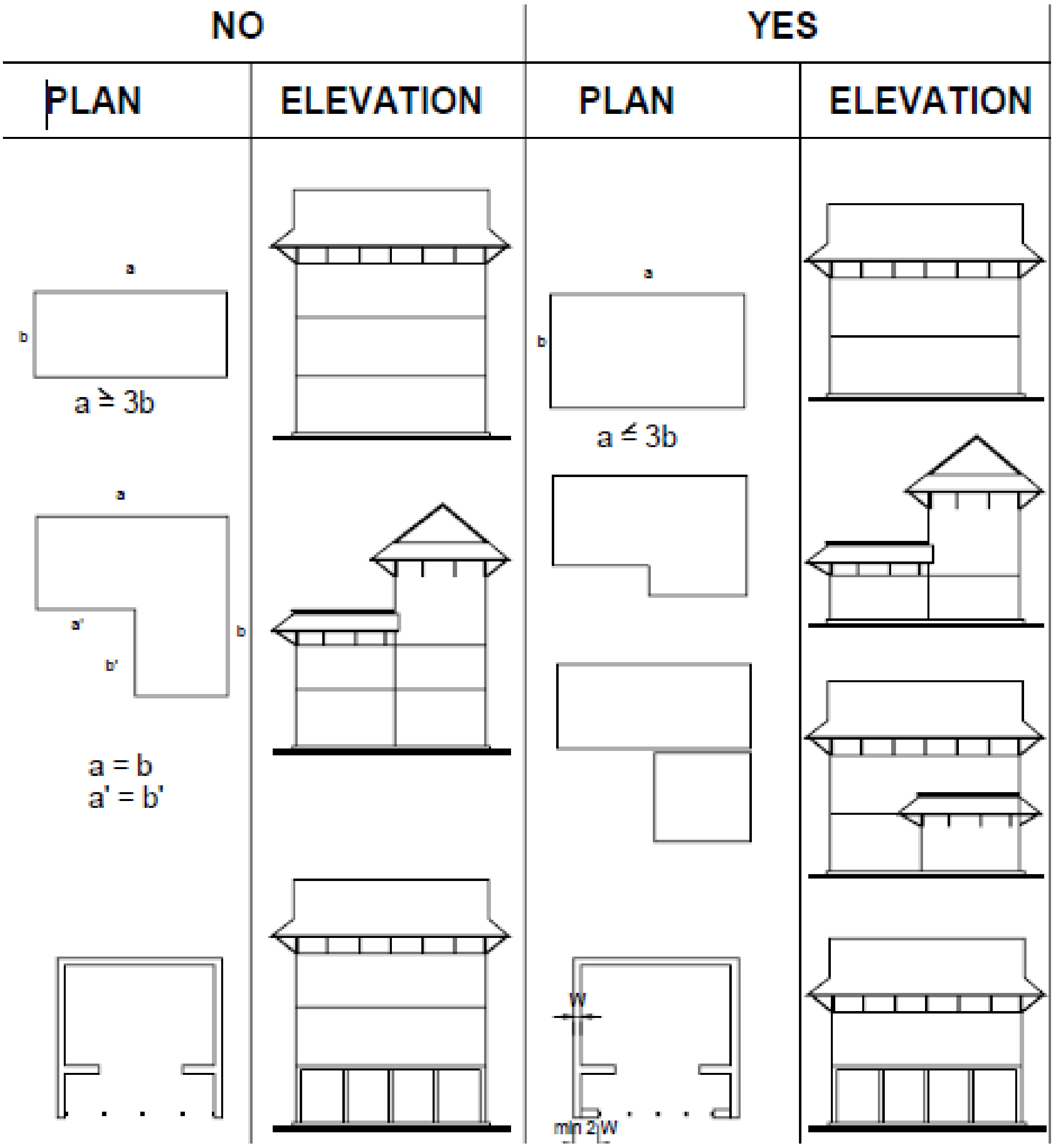


Figure 4-4-1 Recommended forms of buildings (Adapted from NBC 203)

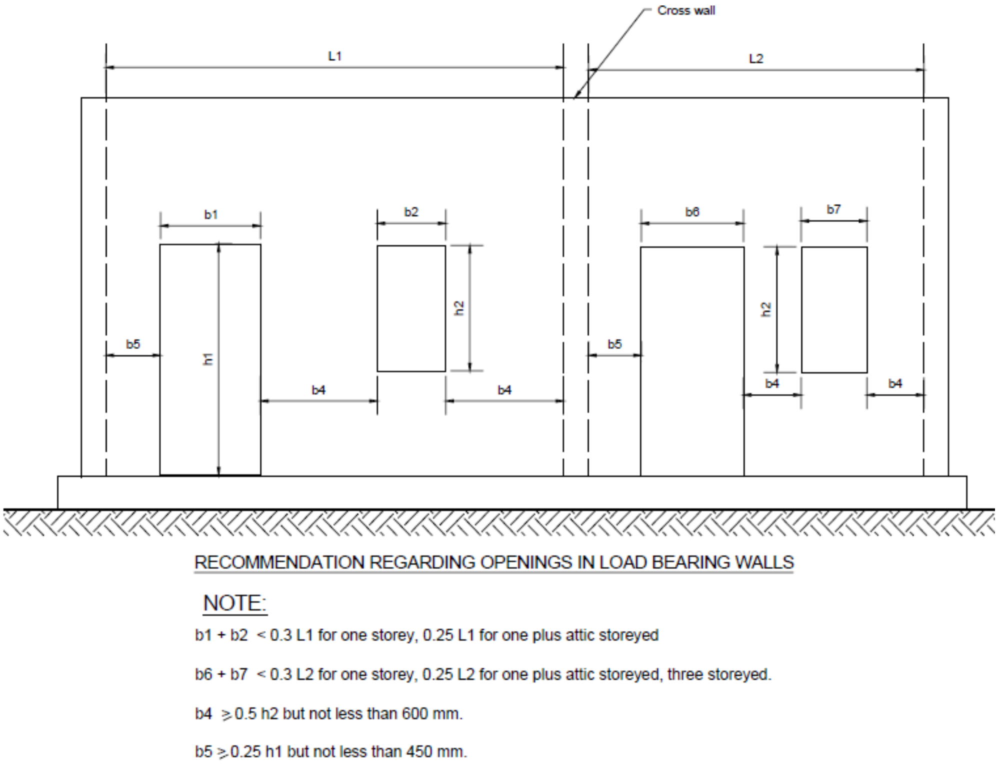


Figure 4-2 Location of Opening (Adopted from NBC 203*)*

4.2 Strengthening of Walls

4.2.1 SEISMIC BELTS

Aims: prevents failure due to overturning providing anchorage to the roof- floor, out of plane strength and stiffness. Establish in plane continuity. Prevent cracked wall section from kicking out in plane.

Seismic belts are the most critical earthquake-resistant provision in an adobe building. They act like a ring or belt, as shown in figure below. Seismic belts hold the walls together and ensure integral box action of an entire building. They are to be provided on all walls on both faces (a) just above lintels of door and window openings and (b) just below floor or roof. 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 delaminating are reduced.

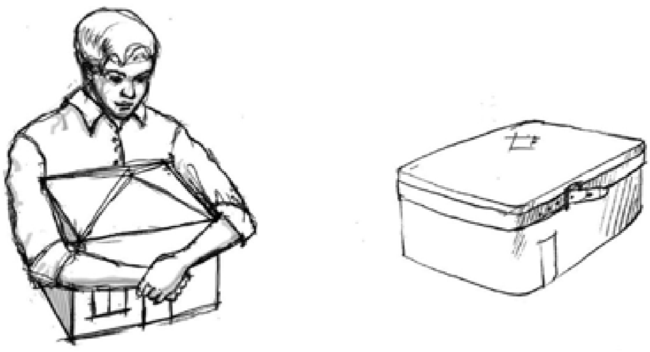


Figure 4-3 A seismic band acts like a belt (adopted from: GOM 1994*)*

The seismic belts are divided into two basic elements:

- Upper wall element
- Lower wall element

Upper wall element is the most important part of a retrofit of the adobe building[3] as it prevents failure due to overturning. It provides anchorage to the roof or floor and out of plane strength and stiffness. The elements like horizontal straps, cables or bond beam establish in plane continuity preventing crack propagation (cracked wall section from moving apart in the plane of the wall).

The lower wall elements prevent the kicking out of cracked wall section along the length of the wall. Wall may be displaced into a door and window openings. However more serious problems tend to occur at the ends of the walls where cracked walls are unrestrained leading to the outward movement of the wall at the base. Such basal displacement is prevented from the lower wall elements. Proper placement, continuity of belts and proper use of materials and workmanship are essential for their effectiveness.

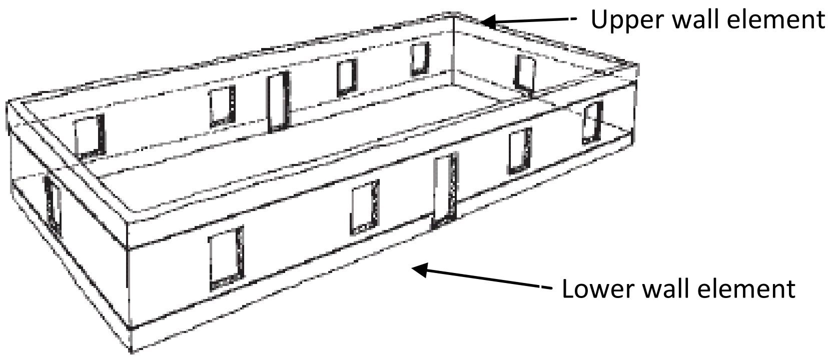


Figure 4-4 Seismic belt showing upper and lower wall elements (source: Manual of The Getty Conservation Institute)

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

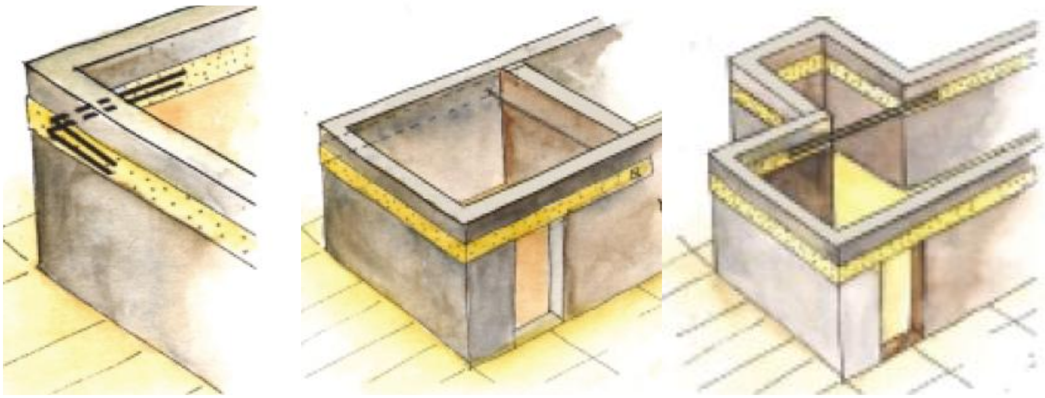
Specifications of Seismic Belts

The seismic belt is made with reinforcement consisting of galvanized welded wire mesh (WWM) and TOR/MS bars that are anchored to the wall and fully encased in cement plaster or micro-concrete. The width of the belt should be 30 mm more than the width of the WWM.

According to the specification of National Disaster Management Division, Government of India Guidelines for J&K, 13 gauge 250 mm wide with 8 longitudinal wires WWM and 2-6 mm dia. MS bars are used in the seismic belts.

- Seismic belts should be connected on both face of the wall.
- Ensuring belt continuity across small masonry projections from the main wall.
- Install the belt reinforcement, including the WWM on three walls. Extend the reinforcement of the belts as close to the fourth wall as possible.
- Make sure that corners do overlap.

a)



b)

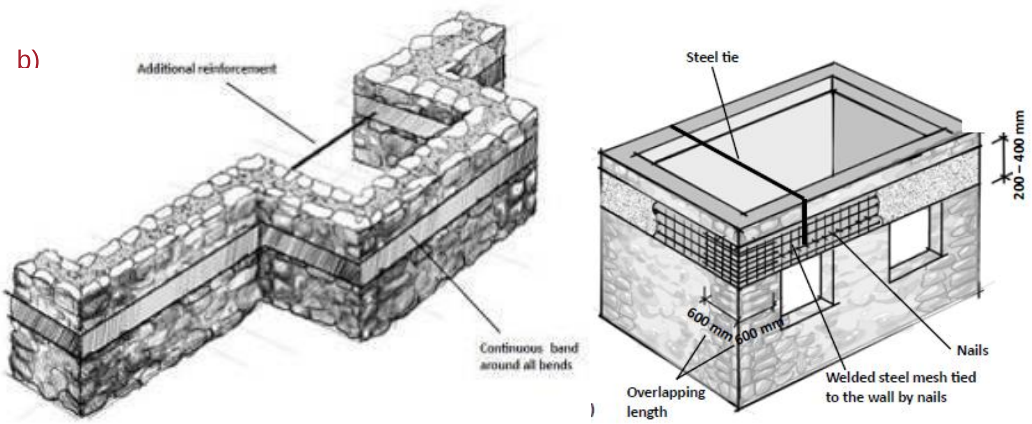


Figure 4-5 a) Seismic belts around various locations (source: UNDP, UNECO & GOI, 2007) b) Additional reinforcement (source: R. Desai and GOM 1998)

Steps for construction of the belts:

Mark the location of belts and remove plaster in the mark places

i. Rake out mortar joints

ii. Clean the surface and wet it with water

iii. Apply neat cement slurry and apply first coat of 12 mm thickness. Roughen its surface after initial set

iv. Installing mesh with bars to walls nailing at about 300 mm apart.

v. Apply second coat of plaster of 16 mm thickness.

4.2.1.1 Gable-wall bands

Gable walls are typically non-load bearing, and the roof, attic, and/or floor framing provides little restraint against outward motion. The walls are taller than others in the building, but are usually of the same thickness. This makes the gable wall more susceptible to collapse. Hence it should be securely anchored to the building at the roof and the attic floor levels for out-of-plane stability. In case of new structure, it is compulsory to provide gable band and roof band. In existing structures however, this can be achieved by cross bracing two gables.

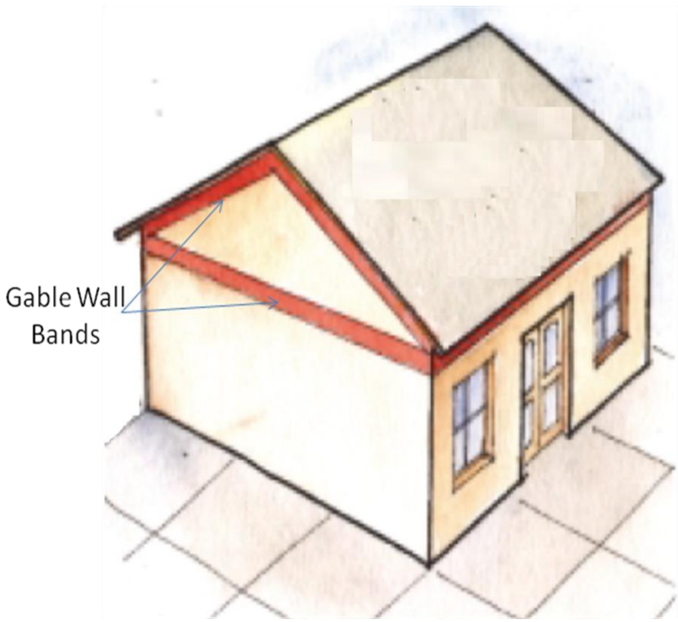


Figure 4-6Gable band

4.2.1.2 Vertical Reinforcing

Due to the weaknesses in brick or stone masonry walls, poor storey-to-storey bonding, Poor wall-to-roof bonding and inadequate resistance to vertical bending in masonry, adobe and stone masonry buildings have horizontal cracks[4], collapse of walls, and sliding of roof with respect to the lower storey. Vertical reinforcement within the masonry wall will help to prevent such failures. It improves the bending strength of the wall to control the horizontal cracks, reducing the possibility of the walls going out of plumb or collapsing. It helps bond the roof to the walls, providing support to the wall and controlling its shaking in an earthquake. It helps to improve the bond between adjacent storeys, which also strengthens the walls.

There are three effective ways to retrofit the wall using vertical reinforcement in the masonry walls

i. Single vertical reinforcement

ii. Reinforcement with welded wire mesh, and

iii. Post-tensioning

Generally 10 -12 dia TOR bar and 13 gauge WWM are used in the first and second option of the retrofitting as specified in National Disaster Management Division, Govt. of India Guidelines for J&K. For third option 12 – 16 dia TOR bar are used.

Single vertical bars must be installed at the inside corner of a wall-to-wall ‘L’ type junction. In the case of a ‘T’ junction it may be installed on either side of the junction as shown in the following figures.

The shear connectors are installed 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.

[4] Horizontal cracks are reduced by increasing horizontal bands (reducing distance between horizontal bands), vertical rebars are considered for shear strength.

Figure 4-7 Vertical bar in corner (source: UNDP, UNECO & GOI, 2007)

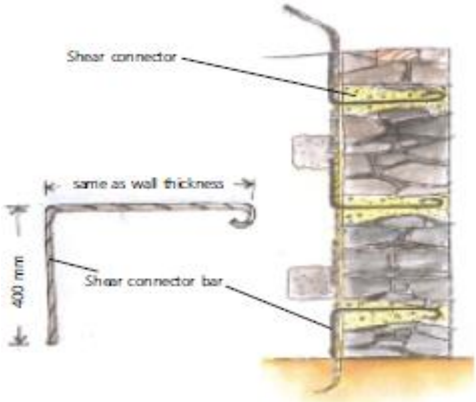


Figure 4-8 shear connector with vertical bar details (source: UNDP, UNECO & GOI, 2007)

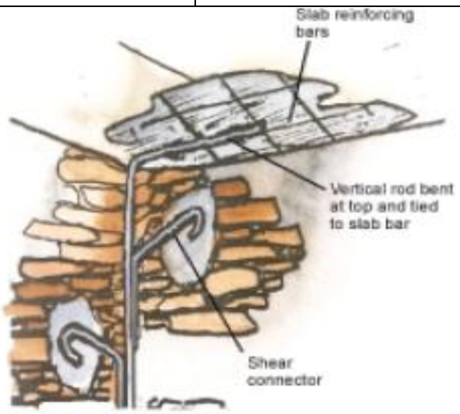


Figure 4-9 Connecting top bent end of vertical rod to slab (source: UNDP, UNECO & GOI, 2007)

The reinforcement with WWM is installed in an 'L' configuration on the outside of 'L' type wall-to-wall junction and in a flat configuration on the outside of a 'T' type junction as shown in the following figures. The belt will start from 300 mm below plinth level and continue up to the top of wall at roof level.

In case of rubble walls, cast in situ RC shear connectors are used with 'L' shaped dowel bar for greater reliability. Shear connectors are to be installed starting at 150 mm (6") above floor level with a spacing of 600 mm (2'). Successive connectors are to be placed on different walls in the corner.

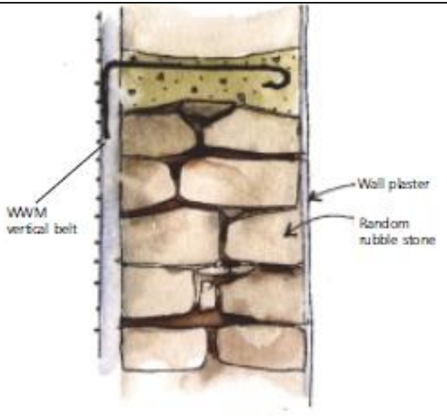


Figure 4-10 Anchoring WWM to random rubble wall with shear Connector (source: UNDP, UNECO & GOI, 2007)

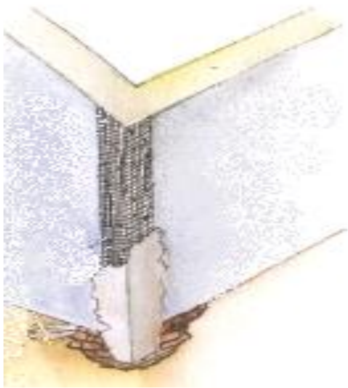


Figure 4-11 Plastering vertical WWM belt using cement plaster (source: UNDP, UNECO & GOI, 2007)

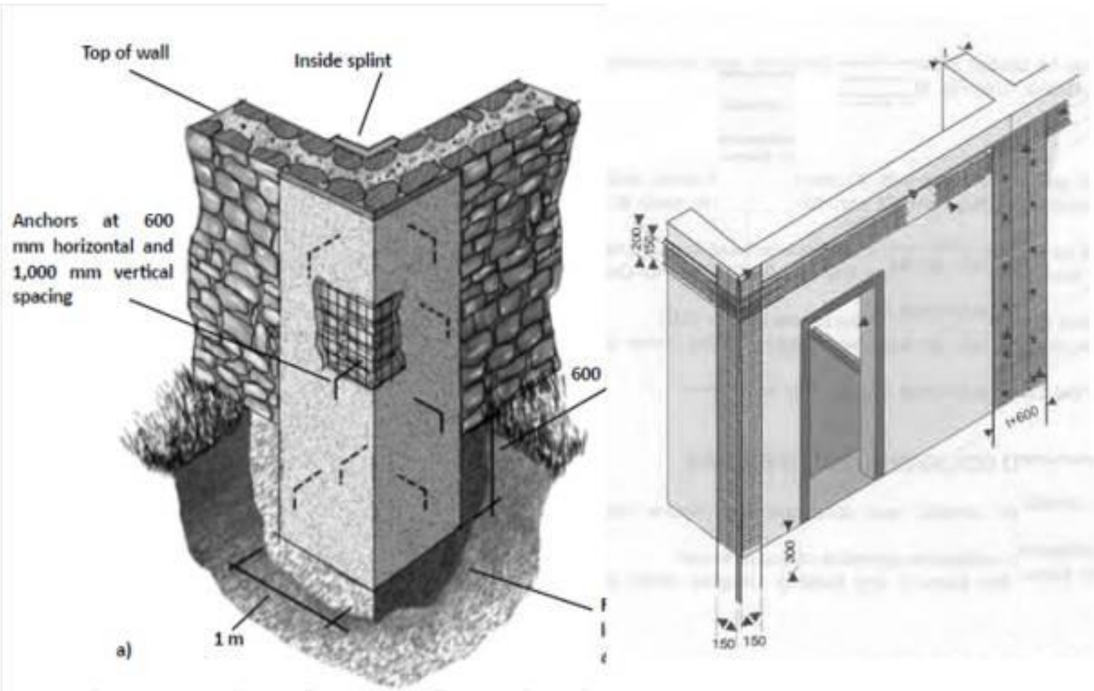


Figure 4-12 Vertical reinforcement with WWM (Source: GOM 1998)

Post-tensioning vertical reinforcement is another effective method for increasing strength of masonry walls. The post-tensioning may be applied externally or be installed internally by drilling vertical cores through the middle of a wall and then inserting steel rods into these cores. The rods may or may not be set in grout, and are then tensioned, which provides an additional compressive force in the wall. This loading modifies the stress behavior of the masonry in bending (i.e. the result of out-of-plane loading). It also increases the shear capacity of the wall.

4.2.1.3 Encasement belt around opening

A typical masonry wall consists of piers between openings, plus a portion below openings (sill masonry) and above openings (spandrel masonry). When subjected to in-plane earthquake shaking, masonry walls demonstrate either rocking or diagonal cracking started from the opening corners. Rocking is characterized by the rotation of an entire pier, which results in the crushing of pier end zones. Alternatively, masonry piers subjected to shear forces can experience diagonal shear cracking (also known as X-cracking. Diagonal cracks develop when tensile stresses in the pier exceed the masonry tensile strength, which is inherently very low.

To prevent such damages, it is necessary to strengthen the boundary around the opening, especially at the corners where concentration of tensile stresses occurs. Encasement helps resist the tearing action that occurs at opening corners. Likewise wrapping of the pier which has very weak resistance to shearing and bending is greatly strengthens it against these forces and prevents the cracks and crushing of piers.

Figure 4-13: Encasing around window and door openings (source: UNDP, UNECO & GOI, 2007)

Generally 280 to 300 mm wide encasement belts are used around the openings underneath of lintel bands, on the sides of the openings, and under the windows and ventilations. The construction procedure is same as that used for horizontal and the horizontal and vertical seismic belts.

4.2.2 STIFFENING WALL/ WALL JACKETING

Aim: Provide out-of-plane stability to unreinforced adobe walls resisting out-of-plane flexure; provide in-plane continuity limiting the relative displacement of cracked walls section preventing extensive wall deterioration.

Adobe walls are weak when subjected to forces other than compression. Even when fully secured to floors at each level, out-of-plane forces can cause significant wall bending that is governed by the ratio of the height between levels of support to the thickness of the wall. Some walls have sufficient thickness or have cross-walls or buttresses which enable them to withstand these out-of-plane forces without modification, however many walls will require seismic improvement. There are a number of approaches to combat this problem as described below:

4.2.2.1 Polypropylene (PP) bands

PP-band retrofitting is a simple and low-cost method that consists of confining all adobe walls with a mesh of PP-bands. PP-bands are an inexpensive, durable, strong, and widely available material, commonly used for packing. PP-band meshes increase the structure ductility and energy dissipation capacity through controlled cracking. It has had practical application in Nepal, Pakistan and Peru with positive reception from the communities.

Shake table tests were performed to verify the efficiency of this technique. Figure shows a full-scale adobe model reinforced with PP bands after a shake table test (Meguro 2008). The scheme was developed in Japan.

Static and dynamic testing by Macabuag (2009), shows that this method extended the collapse time of unreinforced masonry buildings and also provided confinement. The PP-bands are able to prevent brittle collapse, since loads can be maintained even after initial failure of walls.

		
Figure 4-14 Full scale adobe model reinforced with pp band after a shake table test	Figure 4-15PP band mesh (source: Megura and Mayorca)	Figure 4-16 PP band retrofitted house before mortar laying (source: Megura and Mayorca)

Design Methodology:[5]

1. Determine the original structure strength, Vc, and natural period, T.
2. Calculate the elastic base shear, V, according to the regional seismic code.
3. From the relation between V and Vc, estimate the strength reduction factor, Rd.
4. Choose a certain PP-band mesh density, D, and determine the ductility demand, μ_{dem}, from the μ_{dem} versus R_d graph and also the maximum displacement, $\Delta_{max} = \mu_{dem} \times$ first cracking displacement.
5. Assess Δ_{max}.

   If Δ_{max} is acceptable, proceed with out-of-plane verification.

   If Δ_{max} is unacceptable, reduce the μ_{dem}. Repeat the calculation.
6. Verify that out-of-plane deformations do not cause instability

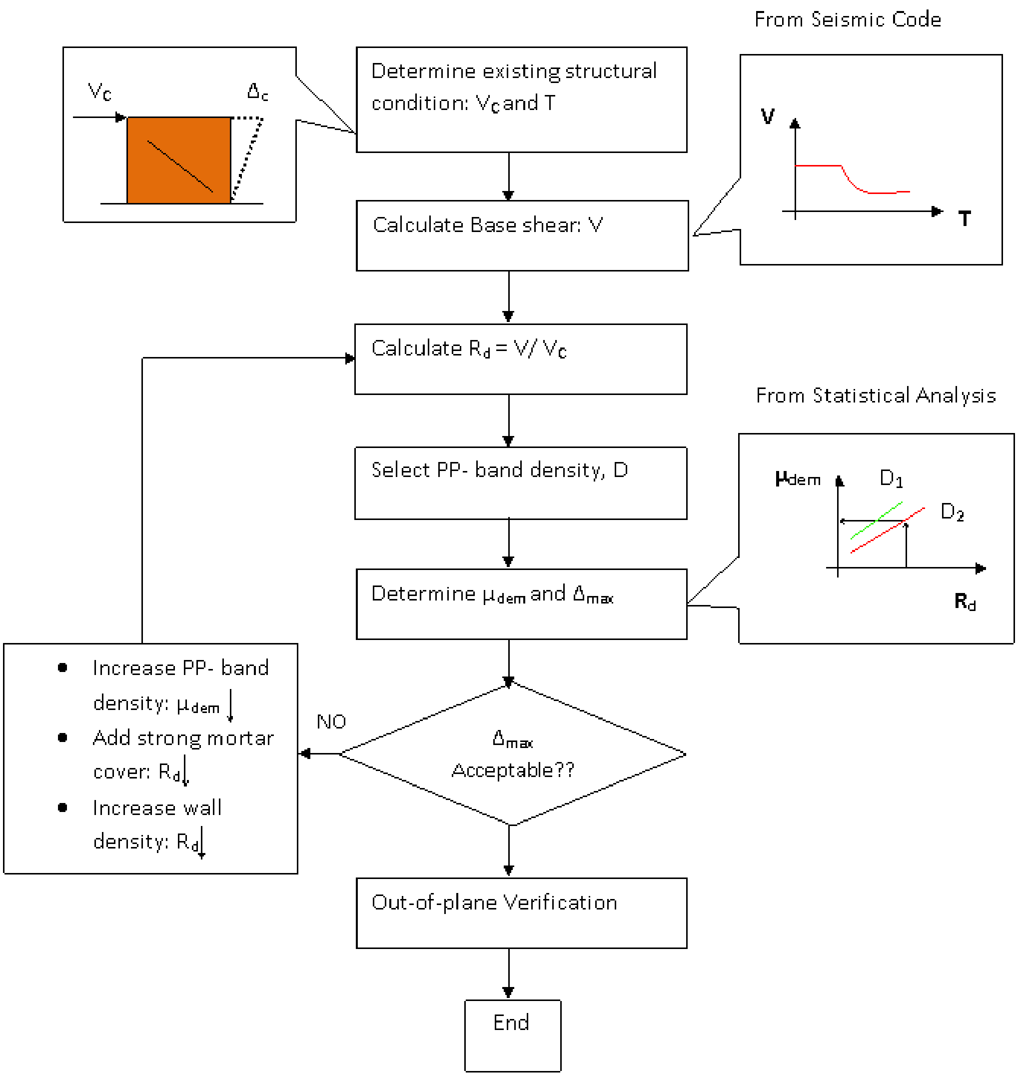


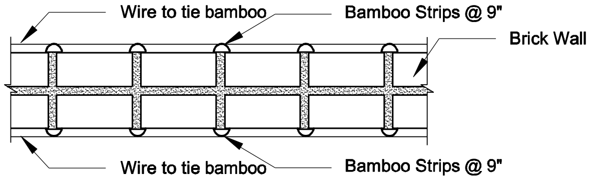
Figure 4-17 Flow chart of Design of PP band (wall Jacketing)

[5] A Step Towards The Formulation Of A Simple Method To Design Pp-Band Mesh Retrofitting For Adobe/Masonry Houses,P. Mayorca and K. Meguro

4.2.2.2 Bamboo Reinforcing:

In this system adobe wall is reinforced by bamboo straps with internal chicken wire mesh. The bamboo is placed horizontally and vertical on adjacent (inside and outside) to the main external wall to encase adobe walls which will prevent both collapse and the escape of debris during earthquake. The retrofitting techniques has been developed and tested at the University of Technology, Australia (Dowling et al. 2005). The test results (see: www.yubetube.com) shows that this method has significantly improved seismic resistance of the adobe structures. A timber ring beam is also included in this complete.[6] The vertical bamboos reinforcements are nailed to the ring beam, thus it ensures the complete support of the wall. Since the technique is fairly simple and less invasive in design, this retrofitting technique is simple and suitable for local builders and is an affordable option for buildings in developing countries.

A simple construction procedure of this technique is presented below:



4-19 Plan showing bamboo reinforcing

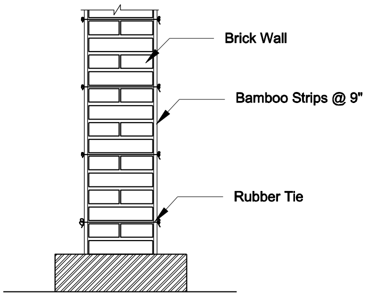


Figure 4-18Wall section showing bamboo reinforcing

6 Seismic Resistant Retrofitting For Buildings, Aimi Elias for Practical Action



Source: Photos by Hari. D. Shrestha Other than stated

Figure 4-20 Construction procedure of bamboo reinforcement (source: www.wuakesafeadobe.net)

4.2.2.3 External cane and rope mesh

An external reinforcement system consisting of vertical cane tied with horizontal ropes forming an approximately 450 mm square mesh can be used to wrap adobe walls, as shown in Figure 4-20. An adobe building model with this reinforcement system was tested on the PUCP shake table (Torrealva 2005) and even though severe cracking occurred, this reinforcement scheme successfully prevented collapse.

Figure 4-21cane reinforcement

4.2.2.4 External wire mesh reinforcement

This technique consists of nailing wire mesh bands against the adobe walls and then covering them with cement mortar. The mesh is placed in horizontal and vertical strips, following a layout similar to that of beams and columns.

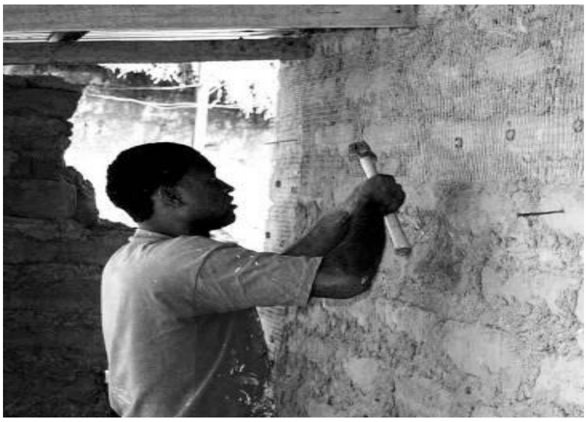


Figure 4-22 Placing the mesh on the wall (source: M. Blondet et al 2003, EERI)

Figure 4-23 Reinforced house (Pisco Earthquake 2007)

4.2.2.5 External polymer mesh reinforcement

This technique uses polymer mesh (geomesh) commonly used for geotechnical applications. The advantage of this material lies in the compatibility with the earthen wall deformation and its ability to provide an adequate transmission of tensile strength to the walls up to the final state.[7]The mesh is attached to adobe walls by plastic or nylon forming a confinement and consequently preventing the total collapse.

The researchers found that it is possible for the walls to disintegrate into large blocks during severe ground shaking, however the mesh prevents the walls from falling apart, and collapse can be avoided (Blondet et al. 2006).

Figure 4-24 Reinforced house with geomesh (Source: world housing tutorial)

Source: Photos by Hari. D. Shrestha Other than stated

[7] Earthquake Resistant Design Criteria and Testing of Adobe Buildings at Pontificia Universidad Católica del Perú, Daniel Torrealva, Julio Vargas Neumann, and Marcial Blondet

A polymeric mesh was selected due to its following characteristics:

- Commercial product with high availability in the market;
- Low-cost when compared with other available meshes;
- Non-corrodible;
- No polished exterior texture;
- Opening size of 15 x 20 mm2, which is an area considered to provide an adequate distribution of stresses and deformations without making the plaster difficult to apply;
- Easily flexible, with a small mesh thickness (0.8 x 0.6 mm2), which can provide a high malleability and good adjustment to all of the wall's irregularities.

4.2.2.6 Used car tire straps

This method uses circumferentially cut straps from the treads of used car tires for tension reinforcement to improve the seismic safety of earthen wall construction. Continuous straps pass through holes drilled in the adobe walls to wrap them horizontally every 600 mm and vertically every 1.2 m approximately. This reinforcement enhances the in-plane and out-of-plane resistance of adobe walls to seismic effects. Vertical straps pass underneath or through the foundations, then rise up the walls, wrap over them and are nailed to the timber wall top plate. The main purpose of this strengthening method is to improve life safety rather than preventing economic loss of property during an earthquake.

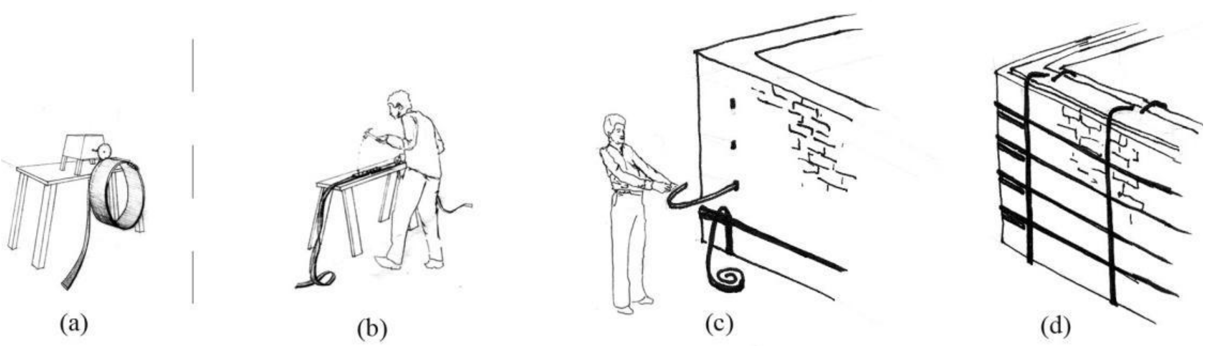


Figure 4-25 Steps in the process of reinforcing an earthen (adobe) house with tire straps. Step (a) is performed in a workshop or factory and (b) to (d) on site. (Source: Courtesy Matthew French)

This type of reinforcing pattern is designed so as at least one pair of straps, either vertical or horizontal, cross every large potential crack that will open during an earthquake (Figure 4-

25).8 The reinforcement provides structural strength and tying-action after the earthen wall material has failed.

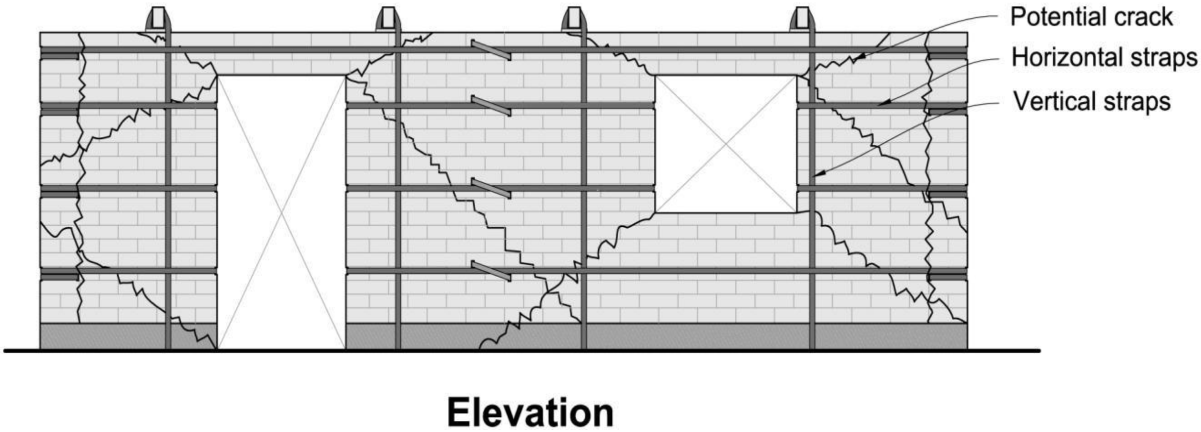


Figure 4-26 An elevation of a typical wall showing positions of expected cracks and strap (Source: Seismic Strengthening of Earthen Houses Using Straps Cut from Used Car Tires: A Construction Guide, Andrew Charleso

4.3 Strengthening of Floor/Diaphragm

4.3.1 STIFFENING FLOOR/ DIAPHRAGMS

Aim: Increase in-plane stiffness of horizontal diaphragms (floors and roof) so the seismic forces can be efficiently transferred to masonry shear walls

In Nepal most of the adobe buildings has the timber horizontal flooring, typically consisting of timber joists with covered with wooden planks, ballast fill, and tile flooring (see Figure 4-26), is termed a flexible diaphragm. A timber floor structure overlaid by planks and bamboo strips is also common. In most cases, timber joists are placed on top of walls without any positive connection; this has a negative effect on seismic performance. The flexible diaphragm amplifies and redistributes seismic forces to the load bearing walls. Inadequate diaphragms are often encountered in larger seismic force amplification. However, this problem can be solved by stiffening the existing the floor structure, and enhancing the connections between floor and walls for ensuring safe transfer of force to and from stiffened diaphragms. Some common techniques are as follows:9

a) Installing new steel straps:

[8] Seismic Strengthening Of Earthen Houses Using Straps Cut From Used Car Tires: A Construction Guide, Andrew Charleson

[9] A Tutorial: Improving The Seismic Performance Of Stone Masonry Buildings, Jtendra Bothara, Svetlana Brzev

New steel straps can be installed to connect the exterior walls to a timber floor, as shown in Figure 4-26 (UNIDO, 1983). This is convenient when the floor beams are perpendicular to the exterior wall, and the connection can be achieved using bolts rather than nails. However, when the floor beams are parallel to the exterior walls, V-shaped straps need to be attached to the floor and anchored to the wall, as shown in Figure 4-26. It is important that straps are sufficiently long and that the timber floor has an adequate tension capacity. The strap thickness should be 3 to 5 mm.

b) Casting a new RC topping atop the existing floor:

A thin RC topping (with a minimum thickness of 40 mm) reinforced with reinforcement mesh can be placed atop an existing floor or roof, as shown in Figure 4-27. The connection between the concrete topping and the existing timber floor should be adequately secured using a sufficient number of well-distributed nails. The RC topping has to be anchored to the walls (similar to Figure 4-27).

c) Installing new timber planks:

A layer of new timber planks can be laid perpendicular to the existing planks and nailed to the floor, as shown in Figure 4-27.

d) Diagonal bracing:

Floor structure can be stiffened by providing new diagonal braces made of timber or steel underneath the existing floor or roof. The braces must be anchored to the walls, as shown in Figure 4-28.

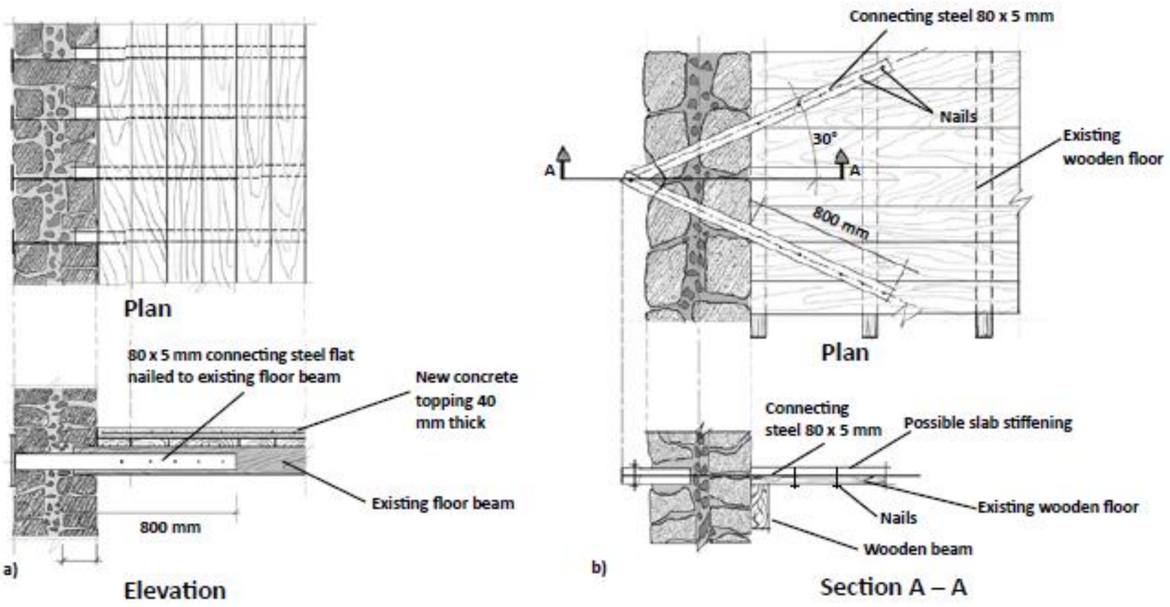


Figure 4-27 Steel straps for wall-to-floor anchorage: a) floor beams perpendicular to the wall, and b) floor beams parallel to the wall (source: UNIDO 1983)

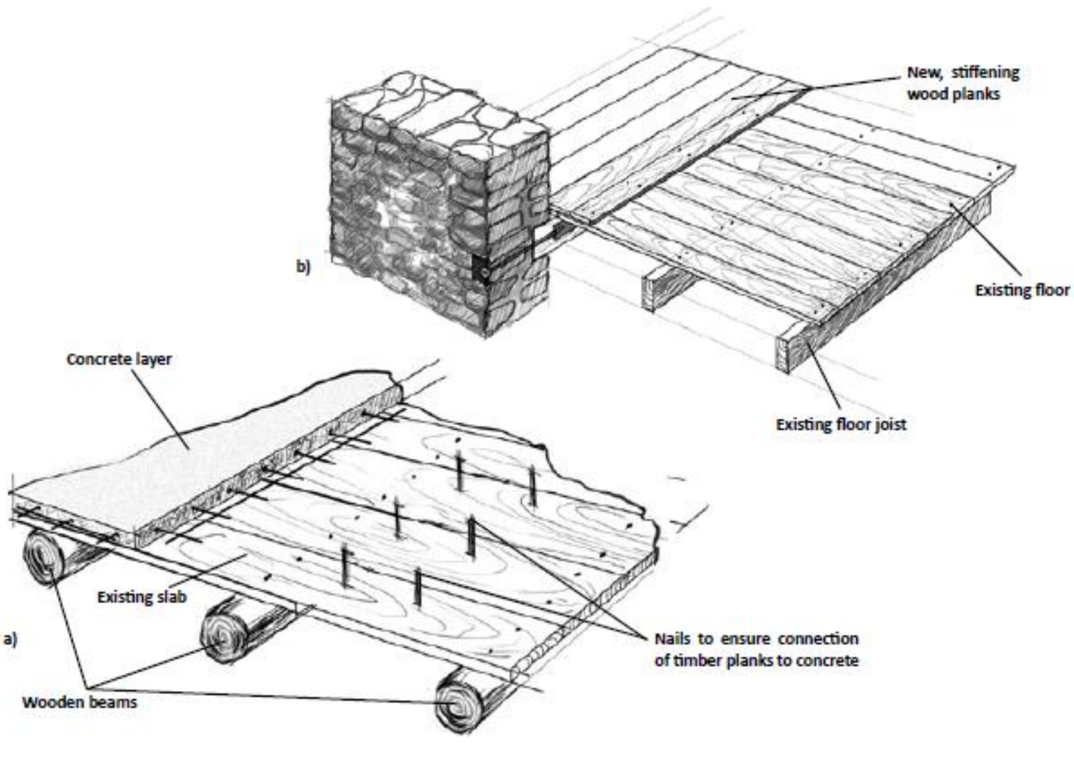


Figure 4-28 Stiffening the floor structures: a) RC topping, and b) new timber planks (source: UNIDO 1983)

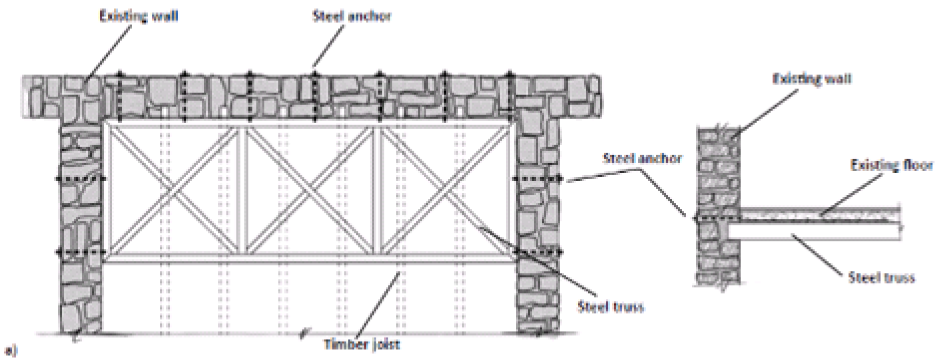


Figure 4-29 Retrofitting the floor and roof structures: a) diagonal braces (adapted from: Tomazevic 1999)

4.3.2 STIFFENING ROOF

Aim: Increasing in plane roof stiffness allows loads to be transferred more efficiently and evenly to the walls to which they are connected, enhancing wall to roof connection.

4.3.2.1 Stiffening the flat wooden roof

Many of the damaged houses have flat floor or roof made of wood logs or timber joists covered with wooden planks and earth. Very often, the framing is not actually attached at all and just rests on top of the wall. Thus, the roof framing can slide relative to the wall or can dislodge bricks at the top of the wall. It also makes the flat roofs a non rigid diaphragm. Thus 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. See figure 4-30.

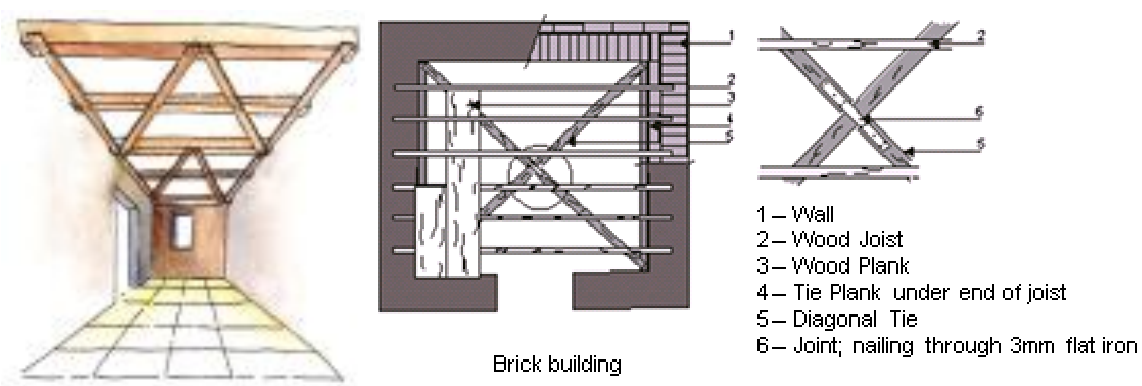


Figure 4-30 Stiffening flat wooden roof

Figure 4-31 Roof to wall connection

4.3.2.2 Stiffening the sloping roof surface

Most of the sloping roof are usually made of timber rafters, purlins with covering of burnt clay tiles or corrugated galvanized iron (CGI) sheets on top. Such roofs push the walls outward during earthquakes. Timber roofs must be braced in plane. The integrity of a timber roof can be improved by tying roof components with straps and nailing them. 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 (Figure 4-31). Also the collars should be provided to prevent roof spreading (Figure 4-31). The important point in retrofitting is the provision of seismic belts just below eave level and the gable level.

Note1

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

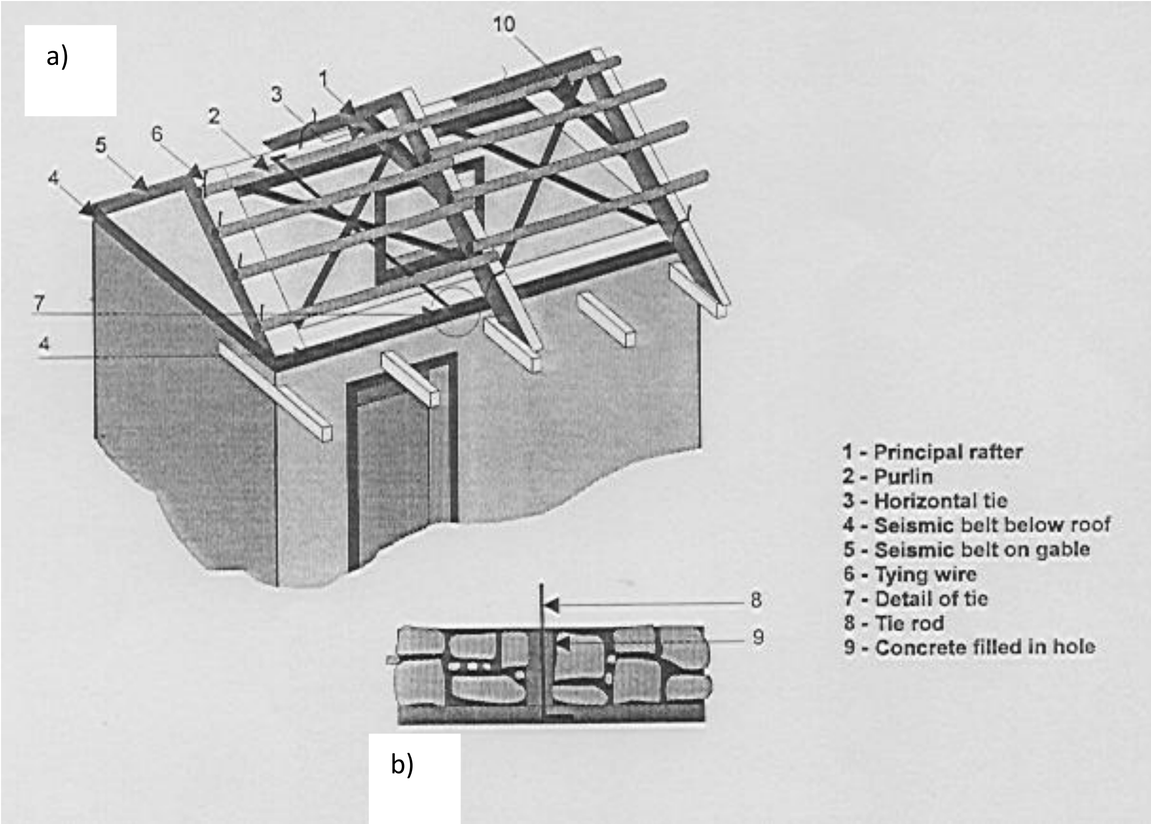


Figure 4-32 Stiffening of sloping roof structure

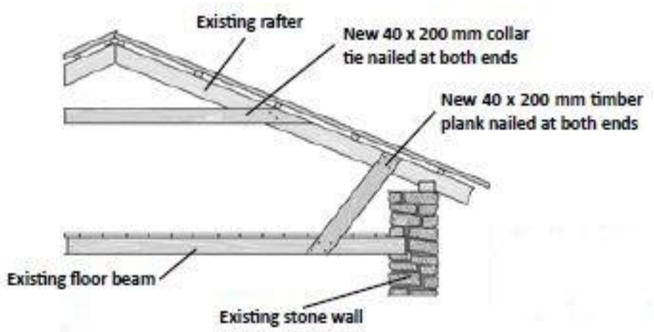


Figure 4-33 roof rafters tying to ceiling joist

Figure 4-34 example (source: Santosh Shrestha)

4.4 Strengthening of Foundation

4.4.1 STRENGTHENING FOUNDATION[10]

Strengthening existing foundations is a difficult and expensive task. A special investigation is recommended before any such intervention.

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[10] A Tutorial: Improving The Seismic Performance Of Stone Masonry Buildings, Jitendra Bothara, Svetlana Brzev

A foundation structure which has experienced differential settlement can be supported by underpinning. Underpinning can be carried out in phases by placing concrete blocks, as illustrated in Figure 4-34.

Sliding movement of a foundation structure can be prevented by constructing new RC supporting beams. This method is especially feasible in sloping ground areas. These beams are constructed deep in the soil, toward the downward sloping side of the foundation. In this way, the foundation is supported sideways and also underneath. Sliding movements can also be prevented by providing RC belts (tie beams) around the building at the foundation level, or by installing a tie beam along the inner side of the foundation (similar to an RC plinth band), as shown in Figure 4-34.

The continuity of longitudinal reinforcement bars should be ensured in all the above schemes. Foundation capacity can also be improved by providing a drainage apron around the building to avoid water seepage directly into the soil beneath the foundation.

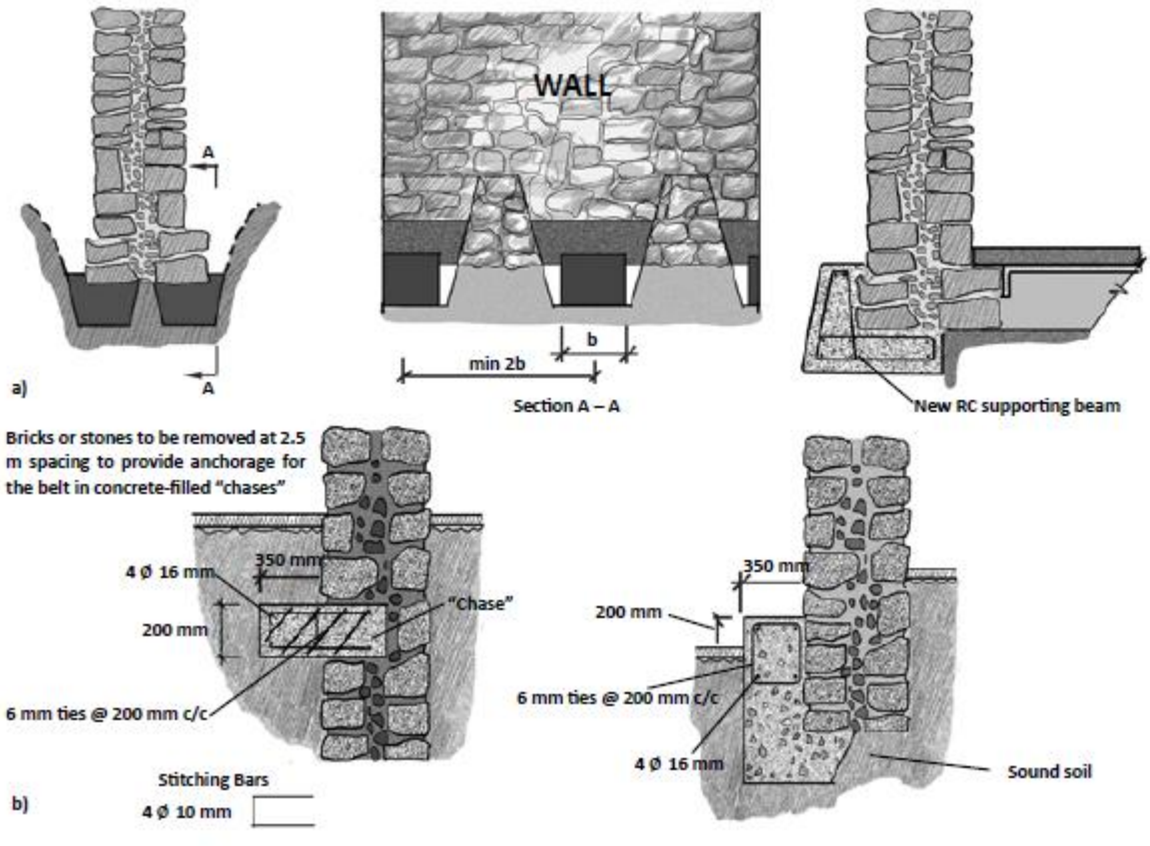


Figure 4-35 Strengthening existing foundations: a) underpinning the foundation, and b) external RC belt (adapted from: GOM 1998 and UNIDO 1983)

4.4.2 CONTROL ON DOOR AND WINDOW OPENINGS IN MASONRY WALLS

4.4.2.1 INFILL OPENINGS[11]

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.

4.4.2.1.1 Seismic belts around door / window opening[12]

The jambs and piers between window and door openings require vertical reinforcement as in table 6-3:

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 4-1 Mesh and reinforcement for covering the jamb area

No. of Storey	**Storey**	**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.

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

12 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

4.4.3 ENHANCING THE LATERAL LOAD RESISTANCE OF STONE MASONRY WALLS

4.4.3.1 Cast in situ Reinforced Concrete Bond Elements/Through-stones

During earthquakes, it shows that the wythes in stone masonry walls bulges outward and delaminate (separate) vertically down the middle due to the absence of through-stones, thereby causing disintegration of the interior and exterior wall wythes as shown in the photo. In an extreme case, collapse of the entire building may occur. Chances of bulging of wall and its delamination can considerably be reduced by stitching wall wythes together by means of through- stones.

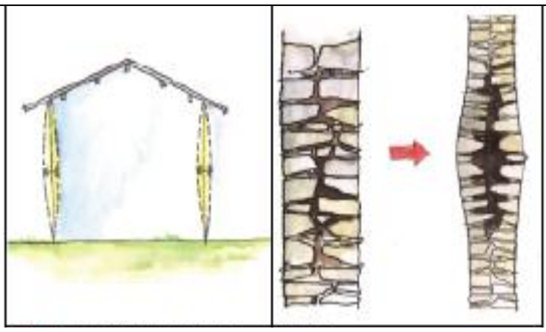


Figure 4-36 Bulging of wall wythes (source: UNDP, UNECO & GOI, 2007)

Figure 4-37 Delamination

Installing cast in-situ reinforced concrete bond element:

I)

Figure 4-38 Marking points for bond elements

II)

Figure 4-39 Removing stone by small rod

III)

Figure 4-40 Making dumb-bell shaped hole through wall

IV)

Figure 4-41 Placing steel bar and filling concrete

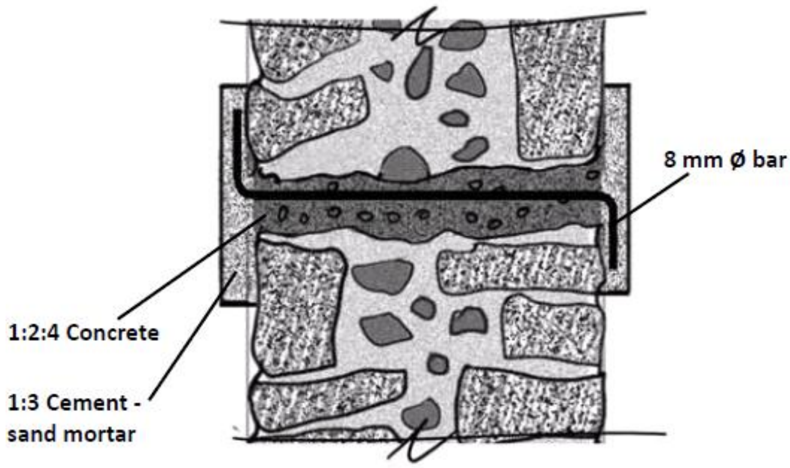


Figure 4-42 Cross section of through-stone

The installation of through-stones is labor-intensive, but it may be a feasible retrofit option for stone masonry walls provided that the wall thickness is not excessively large. First, points spaced horizontally and vertically 1m apart, with a horizontal stagger of 500 mm should be marked. A hole at each point needs to be created in the wall by removing stones. To create a hole, stones need to be loosened by yanking gently sideways, upward and downward using a small crowbar or rod, so that the other stones in the wall are not disturbed. The hole should be dumbbell-shaped, that is, it will be larger on the wall surfaces than in the interior. A hooked steel bar needs to be installed and the hole should be filled with concrete. Finally, the exposed surface should be covered with a rich cement and sand plaster coating and cured for at least 14 days. Through-stones should be carefully installed, otherwise surrounding portions of the wall may be damaged. Examples of through-stone applications are shown in Figures 4.42.

Figure 4-43 Examples of through-stone installation in Maharashtra, India: a) removing stone from the existing wall, and b) surface of a through-stone covered with a plaster (source: UNDP, UNECO & GOI, 2007)

4.5 RESTORATION OF DAMAGED STRUCTURES

4.5.1 GROUTING

Typically the degradation of earth structures results in the formation of cracks, loss of material, loss of cohesion, loss of strength or even collapse of the construction.13 Repairing those cracks is fundamental in order to obtain an improved structural behavior or to re-establish the structural integrity and monolithic behavior that the construction had before. Crack repair also prevents further decay caused by other agents, like water infiltration and plant growth. The traditional techniques for repairing cracks in earth constructions require the removal of parts of the original walls, in order to create a key pattern around the crack and in some cases it requires the enlargement of the crack, which may destabilize the construction. The removed material is then replaced by new materials, which have to assure the bond between the two faces of the crack.14 These techniques are very disturbing and intrusive, which makes the grout injection a more practical and less intrusive solution. Thus, grout injection seems to be a promising solution for repairing earth constructions. However, an

[13]Grouting as a repair/strengthening solution for earth constructions Rui A. Silva, Luc Schueremans, Daniel V. Oliveira

[14] L. Keefe: Earth Building: Methods and materials, repair and conservation, Taylor &Francis, 2005, London, UK.

overall design methodology for grout injection of earth constructions is not available yet. The methodology used for masonry can be adopted.

Methodology for Grouting of cracks[15]

a) Minor Cracks: (Hair cracks less than 5mm)

Procedure:

Step-1 Make a 'V' notch along the crack by chiseling out.
Step-2 Clean the crack with a wire brush.
Step-3 Fill the gap with 1:3 cement mortars (1-cement: 3-coarse sand). Finish the restored parts to match the surrounding wall surface.

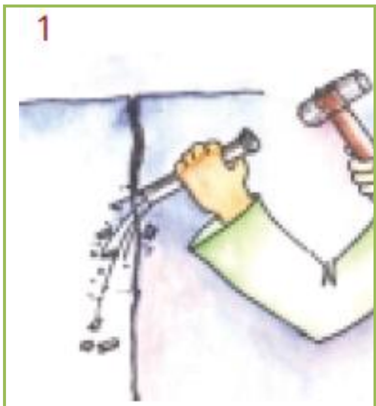


Figure 4-4-44 Making 'V' notch

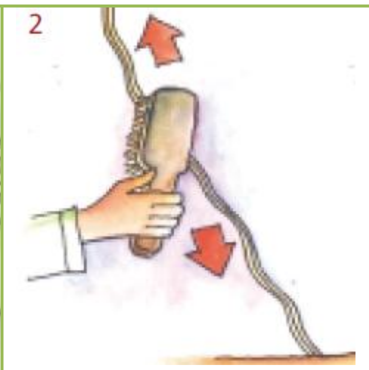


Figure 4-4-45 Cleaning crack with wire brush

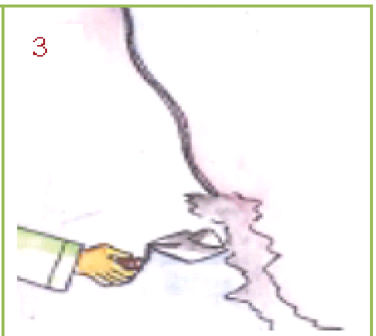


Figure 4-46 Sealing crack with cement mortar

b) Medium Cracks (crack width upto 5mm)

Procedure:

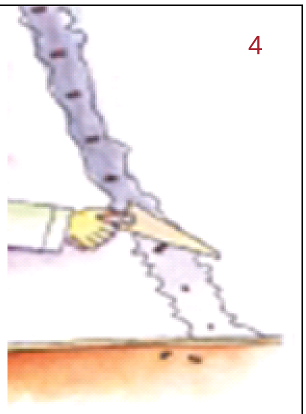
Step-1 Make a 'V' notch along the crack, clean it with a wire brush.

Step-2 Fix grouting nipples in the 'V' groove, projecting 50 mm from the crack on both faces of wall, at a spacing of 150 mm to 200 mm.

Step-3 Clean crack with compressed air through nipples to remove the fine, loose particles inside the crack. (if available).

Step-4 Seal the crack with 1:3 cement mortar, with nipples still projecting, and allow it to harden for some time.

[15] Adapted from Manual for Restoration and Retrofitting of Rural Structures in Kashmir, UNESCO New Delhi Office, UNDP India

			
Figure 4-47 Fixing nipples	*Figure 4-48 Sealing crack with cement mortar*	*Figure 4-49 injecting cement slurry*	*Figure 4-50 Injecting cement slurry*

Step-5 Inject water into crack through the topmost nipple, and then repeat with the lower nipples in succession.

Step-6 Make cement slurry with 1:1 (non-shrink cement: water) and begin injecting it into the nipple, starting with the lowest nipple until the slurry comes out of the next higher nipple. Next inject into the successively higher nipples, one after the other.

Step-7 Cut off the nipples, seal the holes with 1:3 cement mortar and finish the surface to match the adjacent surface.

c) Major Cracks (Crack width between 5mm and 10 mm)

Procedures:

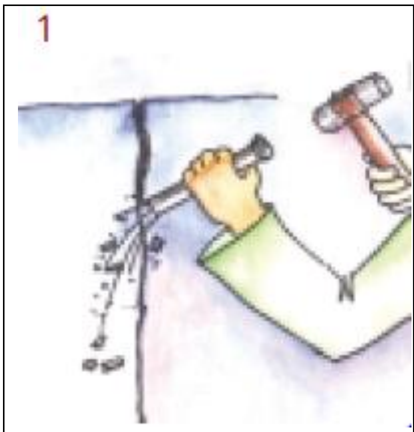
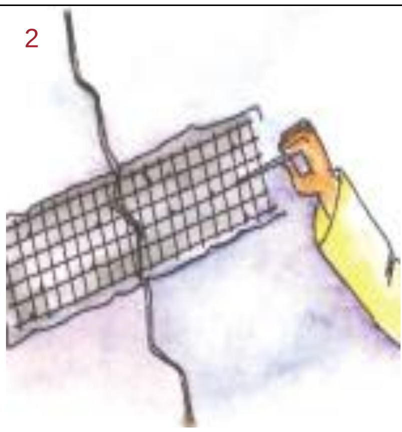
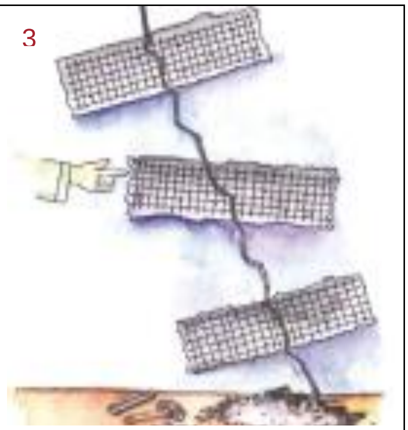
		

Figure 4-51 Making 'V' notch	*Figure 4-52Fixing WWM with wire nails*	*Figure 4-53Ferro cement splices across cracks*

Step-1 Make a 'V' notch along the crack, clean it with a wire brush.

Step-2 Clean crack with water to remove the fine, loose particles inside the crack.

Step-3 Prepare masonry surface on both faces of the wall for fixing 200 mm wide ferro-cement splices across the crack as shown in the diagram, by removing the plaster, raking the joints up to 12 mm depth, and cleaning it with water, extending on both sides of the crack to a minimum of 450 mm length.

Step-4 Fill the crack with 1:3 cement mortar (non-shrink cement: fine sand) with just enough water to permit pushing in of mortar as far in as possible, from both faces of the wall.

Step-5 Install the 150 mm wide 25x25 14 gauge galvanized welded wire mesh (WWM) (2.03 mm diameter) with 100 mm long wire nails inserted at spacing no greater than 300 mm in a staggered manner.

Step-6 A gap of 10 mm must be maintained between the mesh and un-plastered wall.

Step-7 Plaster over the mesh with two 12 mm coats of 1:3 cement plaster.

Figure 4-54 Applying cement plaster on splices

Figure 4-55 Curing

4.5.2 SEALING OF FINE CRACKS[16]

In adobe, cracks are generally quite visible, but their causes may be difficult to diagnose. Some cracking is normal, such as the short hairline cracks that are caused as the adobe shrinks and continues to dry out. More extensive cracking, however, usually indicates serious

[16] Adapted from Manual for Restoration and Retrofitting of Rural Structures in Kashmir, UNESCO New Delhi Office, UNDP India

structural problems. In any case, cracks, like all structural problems, should be examined and should be treated with timely concern so as to prevent them from further propagation.

Procedure:

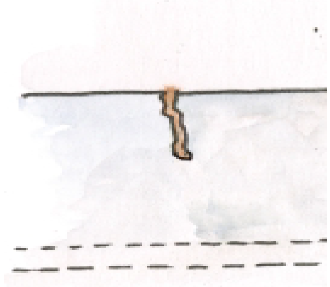


Figure 4-56 Cracks

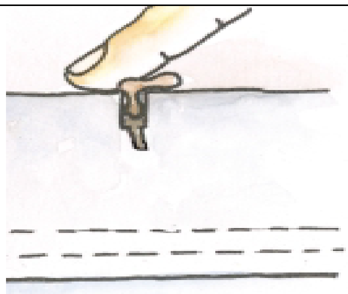


Figure 4-57 Sealing cracks with M-seal

I. Rake the crack with chisel and widen the crack

II. Clean the crack with a wire brush

III. Seal crack with M- seal. Before applying the M-seal, make sure the crack is absolutely dry.

IV. Apply M-seal with thumb pressure so that no space is left out. Remove excess sealant and let it harden.

ANNEX I

CAPACITY ASSESSMENT OF ADOBE BUILDING

1.1 STRUCTURAL BEHAVIORS OF ADOBE BUILDING

Unreinforced adobe has low material ductility coupled with low compressive strength; this is generally given as the reason for its poor seismic performance due to the properties of adobe masonry such as large mass, limited tensile strength, fragile behavior and softening and loss of strength upon saturation. According to the studies, it is seen that adobe buildings do not permit an equal movement of all adobe walls because of lack of proper confinement elements. The adobe's post-elastic behaviors are entirely different from those of ductile building materials because adobe is a brittle material. Due to this, it is possible that vertical cracks appear in the union of two walls during a ground shaking. It is in this case that the out-of-plane capacity of adobe walls can be more important than the in-plane capacity. In this chapter, the out-of-plane and in-plane capacity of adobe walls have been evaluated in the follows adopting from (Sabino N T R, 2008).

1.1.1 OUT OF PLANE BEHAVIOR

Due to the lack of good and proper connection between adobe walls, adobe buildings have mostly the out-of-plane failure. Since adobe is brittle material, the walls at the corner can separate from each other with vertical cracks even with just a short movement. Adobe buildings do not have vertical or horizontal confinement elements (such as beams or columns) that can be useful to form a rigid diaphragm with the roof. In this case adobe walls will try to behave independently of each other. The only stability condition for walls subjected to out-of-plane loads will be given by the rocking behavior, where the concept of slenderness plays an important.

According to the Tolles E. et al (2011), slenderness ratio of a wall less than 6 can result in stable walls (resistance to overturning) while, slenderness greater than 8 results in an unstable wall and the addition of vertical and horizontal reinforcement is compulsory.

In the past earthquake, adobe buildings suffered severe structural damages and collapsed causing innumerable human and materials losses. Majority of these damages and collapses are due to the lack of proper connection between adobe walls, quality of materials, thinner walls, and inadequate location of openings.

1.1.1.1 Procedure for seismic risk assessment

The displacement-based seismic analysis for out-of-plane bending of unreinforced masonry walls developed by Doherty et al. [2002] and Griffith et al. [2003] will be applied to adobe buildings. This procedure is straightforward and is based on a linearized displacement-based approach and has been adapted for a wide variety of URM wall boundary conditions (parapets and simple supported walls as shown in Figure 0-1). The main goal is to predict the response of URM walls when dynamically loading, taking into account their reserve capacity due to rocking.

It is important to remark that out-of-plane walls tend to behave as rigid bodies subjected to rocking and are more sensitive to displacement than acceleration [Restrepo 2004].

The capacity of the URM walls (cantilever or simple supported walls) for an ultimate limit state is evaluated taking into account the secant stiffness (K2) of the wall and the ultimate displacement ($\Delta u \approx t$), measured at the top or at the mid-height of walls, for parapets or simply supported walls, respectively. This capacity can be directly compared to the Displacement Response Spectrum (DRS) considering a 5% damping for maximum displacements greater than 0.5Δ u [Griffith et al. 2003]. For maximum displacements less than 0.5Δ u the stiffness can be represented as function of $\Delta 1$. The maximum displacement is referred to the ordinates of the DRS.

It can be assumed that displacement demand can be estimated via a simplified approach which makes use of elastic displacement response spectra [Doherty et al. 2002].

1.1.1.2 Demand

For the out-of-plane behavior, the ultimate displacement is measured at the top of the wall because we are considering cantilever walls without any collar ring-beam over the walls. If the wall is located above the first level, it is logical to think that the input demand at the ground floor should be amplified by the effect of the height (ground-floor acceleration). To evaluate this amplification some equations have been written in national codes, where not necessarily it is indicated that those are applied for walls above the first floor, even those can be applied to walls located on the ground floor.

For example the Euro-Code 8 gives the following expression, Eq 0-1:

$$S_a = \frac{a_g S}{g}\left[\frac{3(1+Z/H)}{1+(1-T_a/T_1)^2} - 0.5\right] \geq \frac{a_g S}{g} \qquad \text{..............................0-1}$$

where a_g is the peak ground acceleration, g is the gravity acceleration, S is a soil factor, Z is the height from the foundation to the centroid of the weight forces applied on rigid elements, H is the height of the structure, T a is the period of vibration of the wall and T1 is the period of vibration of the structure.

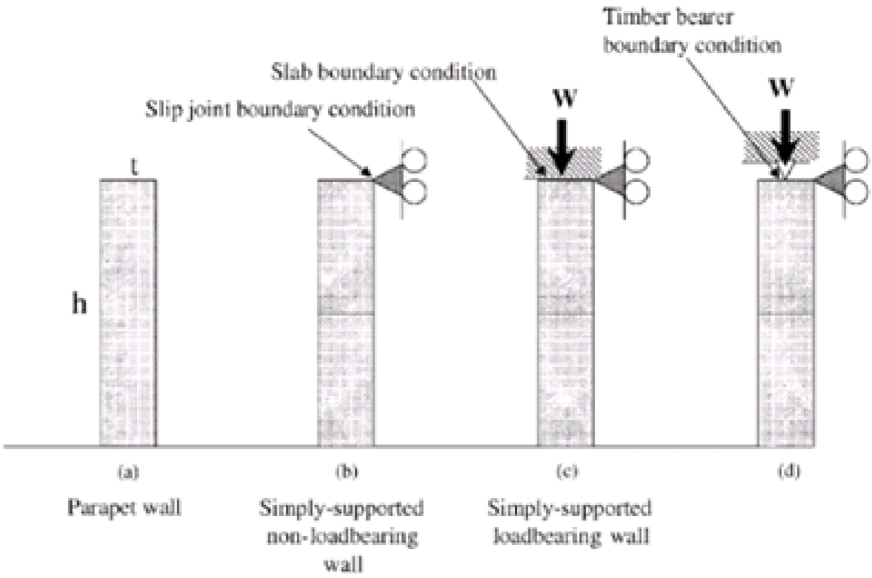


Figure 0-1 Unreinforced masonry wall support configurations (Doherty et al. [2002])

1.1.1.3 Limit States and displacement capacities

The nonlinear force-displacement (Figure 0-2) of a wall subjected to out-of-plane forces can be idealized by means of a suitable tri-linear curve defined by three displacement parameters, Δ_1 , Δ_2 , Δ_u and the force parameter F_o [Doherty et al. 2002]. This simplification will give a suitable relation between the ultimate displacement and the secant stiffness that is explained in the next section.

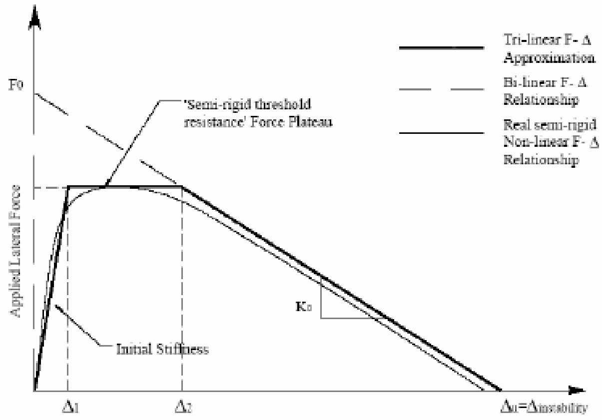


Figure 0-2 Trilinear idealization of the static force- displacement relationship (Griffith et al. 2003)

Δ_1 is related to the end of the initial stiffness and Δ_2 is related to the secant stiffness. Δu is the ultimate displacement, which means the point of static instability (ultimate limit state). From static equilibrium, $\Delta u \approx t$ for cantilever or simple supported walls.

Displacements greater than Δu mean that the wall will collapse. Fo = λW is the force at incipient rocking and is also called the "Rigid Threshold Resistance", λ is the collapse multiplier factor (see section 1.1.1.5).

From simple static equilibrium of the parapets and simple supported walls, the ultimate displacement at the top and at the mid-height of the walls can be obtained, respectively (Figure 0-3). In both cases the ultimate displacement is equal to the wall thickness, $\Delta_u = t$. At the equivalent height, the equivalent ultimate displacement is represented as (2/3) t.

The lateral static strength (F) and the ultimate displacement (Δu) are not affected by uncertainties in properties such as the elasticity module, whereas geometry, boundary conditions and applied vertical forces are the essential parameters [Griffith et al. 2003].

The Δ_1 and Δ_2 parameters can be related to the material properties and the state of degradation of the mortar at the pivot points as a proportion of Δ_u (Table 0-1).

Table 0-1 Displacement ratios for tri-linear model

State of degradation at cracked joint	Δ1 / Δu	Δ2 / Δu
New	6	28
Moderate	13	40
Severe	20	50

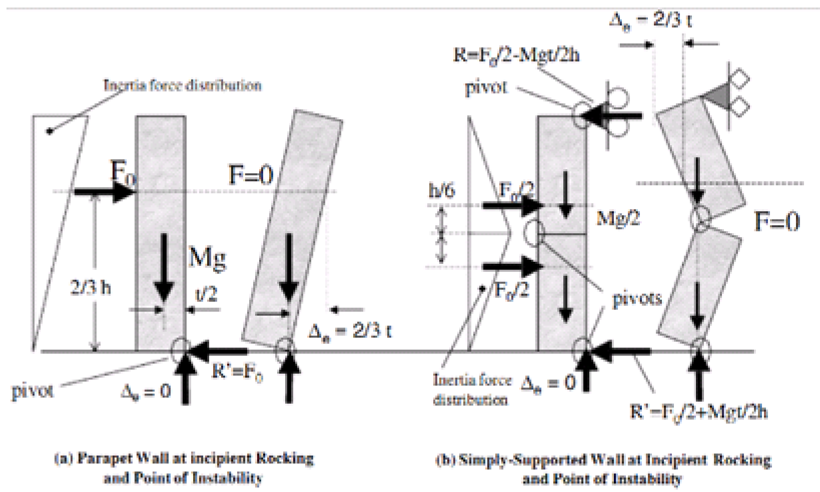


Figure 0-3inertia forces and reactions on rigid URM walls (Doherty et.al 2002*)*

The ultimate limit state is related to the complete stability or the collapse of adobe walls, which means displacement at the top of walls less or greater than the ultimate displacement. Since we are considering collapse mechanisms A, C and D, where walls are rotating at the base, a conservative value of Δu ≈0.8 t can be assumed, where some of the reasons for the reduction are the consideration of dynamic effects and degradation in walls. In this case the secant stiffness, K_2 is considered for the calculation of the period as suggested by [Griffith et al. 2003].

Knowing that adobe walls will have cracks at the base before they collapse, another intermediate limit state can be created. For this, the initial stiffness 1 K should be considered when we are dealing with maximum displacements less than 0.5Δ u [Griffith et al. 2003]. The following limit states described in Table 0-2 have been assumed for the out-of-plane behaviour. The top displacements and crack width have been calculated considering mean values of thickness and height of the adobe walls.

The relationship between top displacement and crack width is described further in the next section. The LS1, LS2 and LS3 indicate the beginning and increment of vertical cracks at the edges of perpendicular walls, which can lead to the separation of them. The ultimate limit state indicates the loss of static stability for the walls.

Table 0-2 Limit states for adobe walls subjected to out of plane forces

Limit State	Top displacement	Crack width at the base	ζ (%)
LS1	≈ 17mm	≈ 3mm	5
LS2	≈ 40mm	≈ 7mm	5
LS3	$\Delta 1 \approx 0.12\Delta u+\sigma_{SD} \approx 45$mm, σ_{SD} =0.01	≈ 8mm	5
Ultimate LS	$\Delta = \varphi\Delta u$, $\Delta u = 0.8\, t$, $\varphi \approx 0.8 \sim 1.0$	≈ 50mm	5

1.1.1.4 Capacity

The scope in this step will be the definition of the period of vibration for a given limit state. Then, with the displacement known as described in the previous section and with the period of vibration, it can be possible to compare the capacity with the demand for each limit state. In this case is not necessary to go from a MDOF system to a SDOF one because we are going to analyze the displacement at the top of the wall. The tri-linear representation of the nonlinear response of the wall can be given in terms of ultimate displacement at the top and $Fo = \lambda W$, where λ is the collapse multiplier factor (see section 1.1.1.5). Following the work of Griffith et al. [2003], the lateral static strength F can be evaluated using following equation and the secant stiffness K_2 by Eq.(0-3), where $Fo = \lambda W$ is the force necessary to trigger overturning.

$$F = F_o\left(1 - \frac{\Delta_2}{\Delta_u}\right) \quad \text{..0-2}$$

$$K_{\Delta_2} = \frac{F}{\Delta_2} \quad \text{..0-3}$$

The lateral static strength F and the ultimate displacement Δu of a wall subjected to out-of-plane action are not affected significantly by uncertainties in the material properties as the elasticity module or the masonry compressive strength, whereas geometry, boundary conditions and applied vertical forces (including self weight) are the essential parameters [Griffith et al. 2003].

For the ultimate displacement is used the secant stiffness K_2 because it is a valid parameter in order to determine if the wall will collapse or not [Griffith et al. 2003]: "…*the use of the effective stiffness K_2 and of the effective period T2 combined with an elastic, 5% damped* *displacement response spectrum seems to work rather well in the prediction of the displacement demand in the large amplitude displacement region (Δu > 0.7Δmax), and can be regarded as suitable for predicting whether a wall will collapse or not*" . Even Doherty et al.[2002] says that the peak response of the tri-linear oscillator can be estimated via an equivalent linear system with secant stiffness K_2 .

a) Period of vibration

The period of vibration for the ultimate limit state can be obtained from:$T = 2\pi(M / K)^{1/2}$. So,using Eq.(4-2) and (4-3) it is obtained Eq.(0-4):

$$T_2 = \left(\frac{4\pi^2 \cdot \Delta_u \cdot \Delta_2}{\lambda g(\Delta_u - \Delta_2)} \right)^{1/2} \qquad \text{......................................0-4}$$

Rewritten Eq.(0-4) for the ultimate limit state it is obtained Eq.(0-5):

$$T_{LSu} = \left(\frac{4\pi^2 \cdot \Delta_{LSu} \cdot \rho_2}{\lambda \phi g(1 - \rho_2)} \right)^{1/2} \qquad \text{...............................0-5}$$

where $\Delta LSu = \varphi \Delta u$, with φis a factor that can be assumed from 0.8 to 1 just to reduce theultimate limit state, and $\rho_2 = \Delta_2 / \Delta u$ (Table 0-2).

For intermediate limit states (where displacement limits are less or equal to Δ_1) a value of

0.12 for $\Delta_1 / \Delta u$ is assumed with a standard deviation of 0.01 (Table 0-1). From static equilibrium a relation between the crack width (ω) and the displacement at the top can be obtained, Eq.(0-6). According to this it is seen that the greater the crack width, the greater the displacement.

$$\omega = \frac{t \cdot \Delta_i}{h} \quad \text{or} \quad \Delta_i = \frac{\omega \cdot h}{t} \qquad \text{...........................0-6}$$

In this case the period of vibration for all the intermediate limit states will be related to the given Δ_1 (initial stiffness) as follows, Eq.(0-7):

$$K_1 = \frac{F}{\Delta_1} = \frac{F_o}{\Delta_1}\left(1 - \frac{\Delta_2}{\Delta_u}\right) \qquad \text{0-7}$$

Replacing Eq.(0-7) into

$$T_1 = \left(4\pi^2 \frac{M}{K_1}\right)^{\frac{1}{2}} \qquad \text{0-8}$$

Eq. (0-8) is obtained which is a fixed period of vibration for all the intermediate limit states:

$$T_{LSi} = \left(\frac{4\pi^2 \Delta_1}{\lambda g(1-\rho_2)}\right)^{\frac{1}{2}} \qquad \text{0-9}$$

1.1.1.5 Collapse mechanisms

In the work done by D'Ayala and Speranza [2003] some typical and feasible collapse mechanisms for historical masonry building have been defined. These mechanisms have been previously identified by post earthquake damage inspections. D'Ayala and Speranza [2003] developed some equations in order to get their associated failure load factor (collapse multiplier,λ= F /W) that is the ratio between the maximum lateral force for static stability over the total weight of the wall.

When buildings do not have a horizontal restriction such as a collar ring-beam, the following mechanism can be seen: Mechanism A assumes that no connection is present at the edges of the wall, or this is insufficient to generate restraint by the party wall. Mechanism B1 and B2 will occur instead of mechanism A when the level of connection is sufficient to involve, beyond the façade wall, respectively, one or both party walls into overturning, due to sufficient length of overlapping between elements common to both walls. Mechanism C refers to the overturning of the corner and it will occur when at least one of the corners of the building is free, which means without adjacent structures. Mechanism D occurs when only a portion of the façade is subjected to overturning and the party walls are not involved directly in the mechanism. Mechanism E is considered when due to the window layout there might be solution of integrity within the façade plane leading to partial failures (Figure 0-4), [D'Ayala and Speranza 2003].

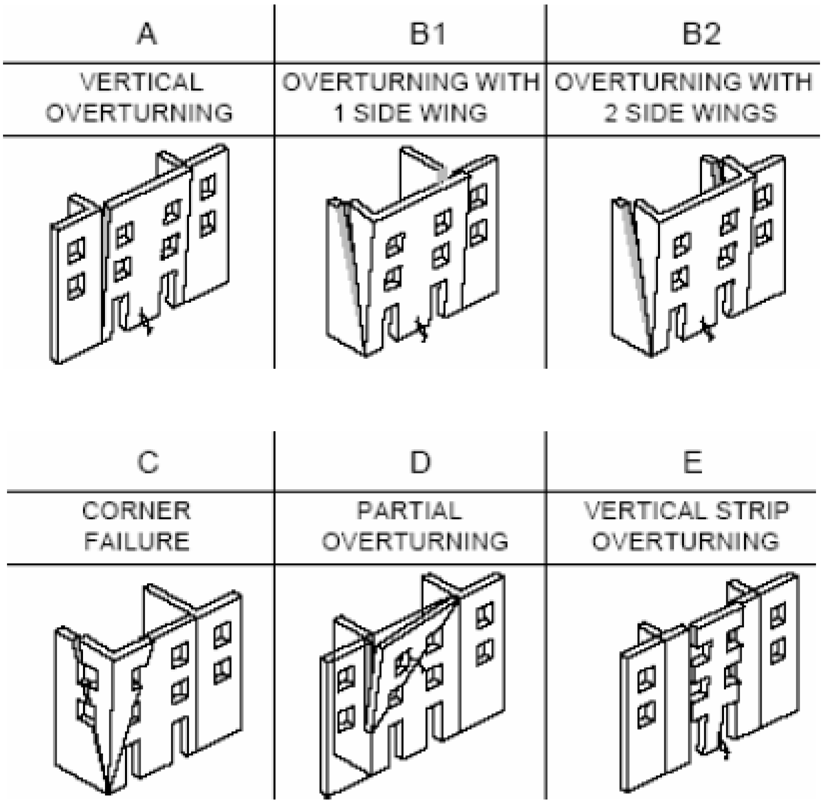


Figure 0-4 Collapse mechanism (D' analysis and Speranza 2003)

Restrepo [2004] has modified the equations for the aforementioned mechanisms in order to fit experimental data and added a new model of collapse. The base of the new equations is the consideration of a pure rigid body motion plus a friction term (just in those cases where friction has been identified as an important source of lateral strength).

These new equations seem to have more accuracy than other previous expressions, and for that reason those are going to be applied to 1-storey buildings in this report.

A description of each of the equations (modified by Restrepo 2004) for the collapse mechanisms is described below -see Eq.(0-10) to (0-16).

Mechanism A

$$\lambda = \frac{\left(T^{2}L/_{2}+\beta\Omega_{pef}\frac{n_s}{3}\mu s b\frac{(\Upsilon+1)}{2}+\frac{K_{\Upsilon}LT}{2}\right)}{h_s\left(\frac{LT}{2}+K_{\Upsilon}L\right)} \quad \text{...0-10}$$

$$K_r = \frac{Q_r}{\gamma_m h_s} \quad \text{……………………………………………………0-11}$$

where T and L are the thickness and length of the front walls, β is the number of edge and internal perpendicular walls, pef Ω is a partial efficiency factor to account for the limited effect of the friction, s h is the height of the failing portion of the wall, μ is the friction coefficient, s is the staggering length, b is the thickness of the brick units, r is the number of courses within the failing portion (assuming courses in the rocking portion). K r is the overburden load, Q r is the load per unit length on top of the front wall and γm is the unit weight of the masonry (18 N/m3).

The partial efficiency factor can be evaluated with Eq. (0-2).

$$\Omega_{pef} = 1.0 - 0.185\frac{L}{h_s} \geq 0 \quad \text{……………………………………..0-12}$$

Even Eq.(0-1) results in a collapse multiplier that represents a collapse mechanism between A and B2 . The friction coefficient for adobe blocks varies from tan 30° ≈0.6 [Corazao and Blondet 1974] to μ= 1.09 [Tejada 2001]. In this report a value of 0.8 will be assumed.

Mechanism C

$$\lambda = \frac{1}{\cos\frac{\pi}{4}}\left[\frac{3T}{2min\left(rh,\frac{Lh}{s}\right)}.\frac{L-L_2}{L}+\frac{T}{rh}.\frac{L_2}{L}\right] \quad \text{……………………0-13}$$

$$L_1 = \min\left(r, nr_{h_s}\right).s \quad \text{……………………………………….…………0-14}$$

$$L_2 = \begin{cases} 0; r < nr_{h_s} \\ L - L_1 \end{cases}; otherwise \quad \text{……………………………………0-15}$$

It is important to remark that when the height of the mechanism is less than the total height of the façade wall, then L2 is equal to zero. r_{hs} is the number of courses within the storey height and n is the number of storeys.

Mechanism D

$$\lambda = \left[\frac{3T}{2min\left(rh,\frac{Lh}{s}\right)}.\frac{L-L_2}{L}+\frac{T}{rh}.\frac{L_2}{L}\right] \quad \text{…………………………0-16}$$

1.1.2 IN-PLANE BEHAVIOR

When adobe walls are well connected or have some buttresses in-plane failure can be expected. That means that the walls can resist forces in its plane until diagonal cracks start to appear. According to the experience from the Pisco earthquake, it has been noticed that the first collapse mechanism of adobe structures is principally due to out-of-plane failure; however, the in-plane failure can be the second one.

1.1.2.1 Procedure for seismic risk assessment

The seismic capacity of the walls represented by the displacement capacity and the corresponding period will be compared with the seismic demand expressed by the Displacement Response Spectrum obtained from a scenario earthquake and developed for many return periods.

1.1.2.2 Demand

From a probabilistic analysis the acceleration response spectrum (ARS) is obtained and this can be transformed to have the displacement response spectrum (DRS). Since those spectra are usually evaluated for a 5% damping, it is necessary to multiply them by a coefficient that takes into account different values of damping for different limit states, Eq.(0-17), [Priestley2007].

$$\eta = \sqrt{\frac{7}{2+\varepsilon}} \qquad \text{...0-17}$$

where the damping ξ is given in %.

1.1.2.3 Limit States and displacement capacities

As it specified in above, the limit states for adobe walls shown in Table 0-3 have been derived from some experimental tests.

Table 0-3 Limit state for adobe walls subjected to in plane forces

Limit state	Description	Drift(%)	ξ (%)	Ductility
LS-1	Operational	0.052	10	1
LS-2	Functional	0.1	10	2
LS-3	Life safety	0.26	12	5

LS-4	Near or collapsed	0.5	16	10

These drift values of the limit states are quite closed to those obtained by Calvi [1999] for brick masonry buildings (Table 0-4).

Table 0-4 Limit stats for brick masonry buildings (Calvi 199*)*

Limit state	Median drift (%)	Coefficient of variation (%)	ξ (%)	Ductility
LS-1 & LS-2	0.1	..	2	1
LS-3	0.3	..	5	1+3/n
LS-4	0.5	1.9	10	1+6/n

1.1.2.4 Capacity

As in the previous section, the scope in this step will be to recall the expression for the period of vibration at a given limit state and to produce an expression to calculate the displacement for a given limit state.

A multi degree of freedom system (MDOF) can be represented as a SDOF system having as principal parameters the effective mass (m_{eff}), the effective stiffness (k_{eff}) and the effective height (h_{eff}).

The maximum displacement for a given limit state (Δ_{LS}) can be represented as the summation of the yield displacement Δ_y and the plastic displacement p_{Δ} (Eq. 0-18, 0-19 and 0-20), The coefficients, k_1 and k_2 takes into account the conversion from MDOF to SDOF system.

$$\Delta_y = k_1 . h_r . \delta_y \quad \text{.......................................0-18}$$

$\Delta_p = k_2.(\delta_{LS} - \delta_r)h_{sp}$ ……………………………………0-19

$\Delta_p = \Delta_y + \Delta_p$ ………………………………………………..0-20

Where k_1 and k_2 are coefficients that depends on the mass distribution and on the h sp (effective height of the piers going to the inelastic range). The effective displacement is computed then with Eq.(0-21), (0-22) and (0-23), which assumes lumped masses f m at each floor and the masonry is assumed to have a distributed mass m_m per unit length.(Figure 0-5)

$$\Delta_e = \frac{\delta_y^2 \sum_{i=1}^{n} h^2 m_{f_i} + 2\delta_y \delta_p h_{sp} \sum_{i=1}^{n} h_i m_{f_i} + \delta_p^2 h_{sp}^2 \sum_{i=1}^{n} m_{f_i} + M_m}{\delta_y \sum_{i=1}^{n} h_i m_{f_i} + \delta_p h_{sp} \sum_{i=1}^{n} m_{f_i} + N_m}$$ ***…………0-21***

$$M_m = \int_0^h m_m (x\delta_y)^2 dx + \int_{h_1}^{h_f} m_m (x\delta_y + \delta_p h_{sp})^2 dx$$ …………………………..……0-22

$$N_m = \int_0^h m_m (x\delta_y) dx + \int_{h_1}^{h_f} m_m (x\delta_y + \delta_p h_{sp}) dx$$ ………………………………………0-23

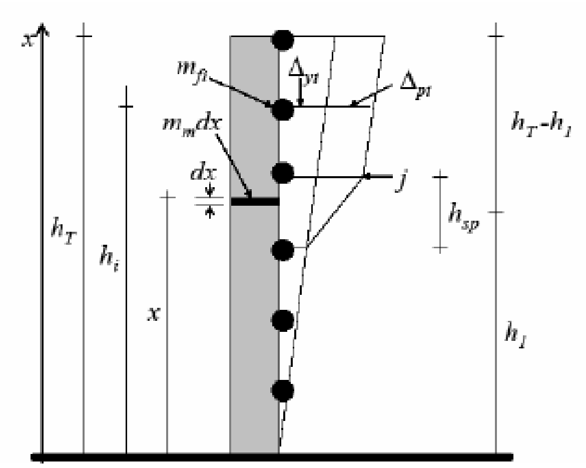


Figure 0-5 Simplified **model for the definition** of k2 (Restrepo 2004)

a) Evaluation of k1

The coefficients k1 can be evaluated in an explicit way equalling the effective displacement

Δe (having$\mu = 1$) to ΔLS. For example, assuming that for 1-storey building h1 is measured at the mid-height and μ= 1 ($\delta p = 0$), then Eq.(0-24), (0-25) and (0-26) are found, where m mT is the total mass of the wall $(m_{m_T} = m_m . h_T)$.

$$\Delta_e = \frac{\delta_y^2 (h_T^2 . m_{f_1} + M_m)}{\delta_y (h_T . m_{f_1}) + N_m} \quad \text{……0-24}$$

$$M_m = \frac{m_{mT} \delta_y^2 h_T^2}{3} \quad \text{……0-25}$$

$$N_m = \frac{m_{mT} \delta_y h_T}{2} \quad \text{……0-26}$$

Doing $\Delta e = \Delta LS$ and solving for 1 k it is obtained Eq.(0-27):

$$K_1 = \frac{m_f + \frac{m_{mT}}{3}}{m_f + \frac{m_{mT}}{2}} \quad \text{……0-27}$$

b) Evaluation of k2

Considering μ= 2 , $k_1 = 0.80$ and the effective height of the piers sp T $_h$ = h , the value of k_2 can be evaluated analyzing again with Eq. (0-21), (0-22) and (0-23).

$$\Delta_e = \frac{4\delta_y^2 h_T^2 m_f + M_m}{2\delta_y h_T m_f + N_m} \quad \text{……0-28}$$

$$M_m = \frac{19 m_{m_T} \delta_y^2 h_T^2}{12} \quad \text{……0-29}$$

$$N_m = \delta_y h_T m_{m_T} \quad \text{……0-30}$$

Replacing the last expressions into Eq(0-21) where $\Delta e = \Delta_{LS}$ it is obtained the expression for k_2 , Eq.(0-31):

$$k_2 = \frac{4m_f + \frac{19}{12} m_{m_T}}{2m_f + m_{m_T}} - k_1 \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 0\text{-}31$$

Evaluating Eq.(0-31) for the mass values explained before, it is obtained $k_2 = 0.95$. It is important to mention that variation in ductility does not affect greatly, k_2 values

c) Period of vibration

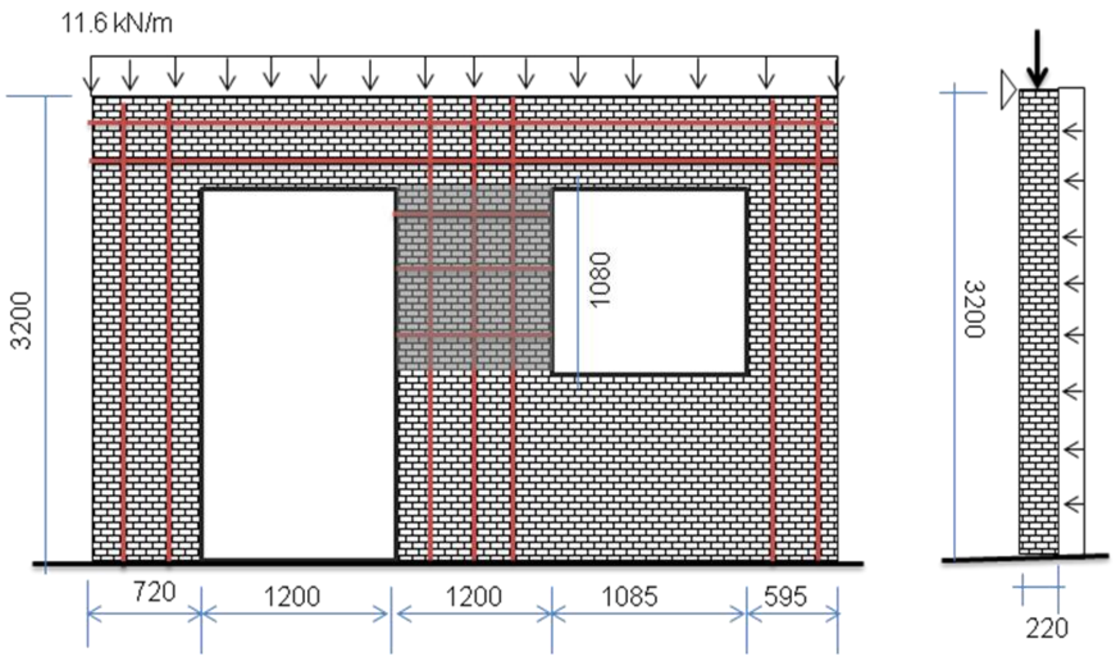
The limit state period of vibration of adobe walls is rewritten for convenience:

$$T_{LS} = T_y \sqrt{\mu_{LS}}\text{, where } T_y = 0.090 \cdot H^{3/4} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 0\text{-}32$$

This period is assumed equal to the period of the SDOF system. This is because the fundamental period of a MDOF is related more or less to 80% of the total mass, which can be a similar value to the effective mass m_{eff} in a SDOF system.

SAMPLE DESIGN CALCULATION

Design a TS bar jacketing seismic retrofit for a perforated adobe wall that is 220 mm thick and have geometric dimension shown in Figure below. Consider in-plane shear force of V* = 44.3 kN and an out-of-plane uniform pressure of vo* = 3.8 kN/m2 were calculated for a maximum credible earthquake. The overburden axial stress due to supported roof was calculated to be 11.6 kN on per meter of the walls. The walls are known to have adequate wall-diaphragm anchorage and masonry is in stable condition without any visible deterioration.



lw	=	1.2	m	q	=	11.6	kN/m
he	=	3.2	m	tw	=	0.22	m

1 Establishing seismic demands

V*	=	44.3	kN	In-plane seismic force
v*	=	3.8	kN/m2	Out-of-plane uniformly distributed seismic force

2 In-plane seismic demands

M* = 141.76 kN-m

3 Out-of-plane seismic demands

M* = 5.8368 kN-m

4 TS bar stress at nominal out-of-plane strength

The out-of-plane strength of wall is more critical than in-plane strength

5 Nominal out-of-plane strength

Masonry density	=	18	kN/m3
W_w	=	15.2064	kN
Nt +0.5 Ww	=	21.5232	

Using a permissible maximum TS bar stress at nominal strength

.= fy, the nominal out-of-plane flexural strength of the wall is established as,

Assume 10 mm dia twisted bars

fy	=	1104	Mpa	
Ah	=	14.8	mm2	10 mm dia
fyAs	=	32678.4	N	
	=	32.6784	KN	
d	=	225	mm	
	=	0.225	m	
a	=	0.004966	m	

Substituting the values of a and d,

nominal strength yields,

8.148 KN-m > 5.84 KN

Thus, assumed 10 mm dia is sufficient to provide the wall required out-of-plane strength

6 Nominal in-plane strength

Checking for shaded wall section

bw	=	220	mm
lw	=	1200	mm
he	=	1080	mm
Ww	=	5.13	kN
Nt + Ww	=	20.76	kN

7 Check for diagonal shear strength

V_{dt} = 51.18 kN

Nominal shear resistance, Vn, is determined as the minimum of resistance corresponding to the bed-joint sliding failure mode, Vs, resistance corresponding to the diagonal tension failure mode, Vdt, and resistance corresponding to the toe crushing failure mode, Vtc

$\Phi_{fv} V_{dt}$ = 38.39 KN < V* = 44.3 KN

A s Φfv Vdt < V*, the assumed Av is not sufficient to provide required in-plane shear strength to the wall section and additional shear reinforcement is required

Assume three 6 mm bars are provided for additional shear strength.

V_{dt} = 63.79 kN

$\Phi_{fv} V_{dt}$ = 47.84 KN > V* = 44.3 KN

Thus combination of Av and Ah is sufficient to provide required in-plane shear strength to the wall section

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