

SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL

2016

A D O B E



Government of Nepal
Ministry of Urban Development
Department of Urban Development and Building Construction
Babarmahal, Kathmandu

MINISTRY OF URBAN DEVELOPMENT, 2016

The Seismic Retrofitting Guidelines of Buildings in Nepal has been developed by Center of Resilient Development (CoRD) and MRB Associates with support from UNDP/Comprehensive Disaster Risk Management Programme.

The personnel involved:

Dr. Hari Darshan Shrestha (Team leader, CoRD)

Dr. Jishnu Subedi (CoRD)

Mr. Manohar Rajbhandari (MRB Associates)

Manual for Restoration and Retrofitting of Rural Structures in Kashmir prepared by UNDP/UNESCO and GOI has been heavily referred in preparation of this guideline.

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SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL, 2016
ADOBE AND LOW STRENGTH MASONRY STRUCTURES

Government of Nepal

Hon'ble Arjun Narasingha KC
Minister
Ministry of Urban Development
Singhadurbar, Kathmandu, Nepal



Phone: 977-1-4211792 (Off.)
Fax: 977-1-4211873 (Off.)
Phone: 977-1-4353270 (Res.)
Website: www.moud.gov.np

Ref.No.

MESSAGE



I am glad to know that the Ministry of Urban Development is publishing the "SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL (ADOBE)," SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL (MASONRY)" and " SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL (RCC)". The aim of these documents is to guide and facilitate the retrofitting works of buildings to make them earthquake resistant and thereby reducing the risk of life and injury during an earthquake.

Recent Gorkha Earthquake 2015 claimed 8,790 lives in total and more than 22,300 were injured. More than 500,000 residential buildings and 2656 official buildings were collapsed completely and almost 200,000 residential buildings and 3,622 official buildings were partially damaged. Likewise more than 19,000 rooms of school building were completely damaged and more than 11,000 rooms were partially damaged. More than 2,900 numbers of temples and social buildings have been severely affected including all seven world heritage sites in Kathmandu.

This retrofitting guideline will be a useful document for those existing building which are partially damaged and can be re-used through retrofitting and can also be used in controlling the extent of damage of an existing structure.

I would like to encourage the practitioner, technical persons, designers and Engineers to follow the guidelines who are involved in retrofitting and construction works of buildings and would like to request for the media persons too for highlighting the usefulness of this document for safer building construction in our nation.

I would like to acknowledge the efforts made by the staffs of Ministry of Urban Development, and Department of Urban Development and Building Construction who have given their valuable feedback and guidance. Likewise I would also like to extend my gratitude to United Nations Development Program (UNDP) and Center of Resilience Development (CoRD) and MRB Associates for their support to prepare these documents.

A handwritten signature in black ink, reading 'Arjun Narasingha KC'.

Arjun Narasingha K.C.
Honorable Minister
Ministry of Urban Development



Government of Nepal
MINISTRY OF URBAN DEVELOPMENT

4211693
Phone: 4211883
4211673
Fax: 977-1-4211873

Ref.No.

Singhadurbar,
Kathmandu, Nepal

FOREWORD



Nepal is located between two active tectonic plates, Indian and Tibetan, where the Indian plate is subducting at a rate of about 3 cm per year below the Tibetan plate. The existence of the young Himalayan range is an evidence of this continued uplift. As a result of this tectonic movement, Nepal lies in the most seismically active zone.

The past history of frequency and intensity of earthquake have exposed the vulnerability and coping capacity of the nation. The damage incurred during the earthquakes has been massive, for instance in 1988, the 6.7 magnitude earthquake killed 721 people and 7000 buildings were destroyed. Recent Gorkha Earthquake 2015 claimed 8,790 lives in total and more than 22,300 were injured. More than 500,000 residential buildings and 2,656 official buildings were collapsed completely and almost 200,000 residential buildings and 3,622 official buildings were partially damaged. These earthquakes highlighted a need for preparation of the National Building Code to ensure structural safety of the buildings, though it was formally enforced only in 2004.

While the implementation of building code has been a challenge, there is already a significant stock of non-engineered, semi-engineered structures, built before the code was implemented that need to be strengthened for withstanding the future earthquake. This document - "Retrofitting Guideline" has been developed to fill this gap.

The objective of this document is to reduce risk to life and injury during an earthquake damage or to control the extent of damage of existing structures. This will be a guiding document for the design professionals with the primary purpose of providing analysis and design methodology for use in the seismic evaluation and retrofitting of the existing buildings in Nepal. This manual is being prepared in three separate volumes providing retrofitting guidelines for adobe structure, masonry structure and RCC structure covering both theoretical and practical aspects of retrofitting.

I would like to acknowledge the efforts made by Mr. Shiva Hari Sharma, Joint Secretary and Mr. Pramod Krishna Karmacharya, undersecretary of Ministry of Urban Development. Likewise the staffs of Department of Urban Development and Building Construction who have given their valuable feedback and guidance also deserve recognition. I would also like to extend my gratitude to Mr. Vijaya Singh, Assistant Country Director of United Nations Development Program (UNDP) and Center of Resilience Development (CoRD) and MRB Associates for their support to prepare these documents. Last, but not the least, I would also like to extend my gratitude to all the professionals, who were engaged in the process of preparation of this document for giving it a final shape.

Deependra Nath Sharma
Secretary
Ministry of Urban Development



Government of Nepal
MINISTRY OF URBAN DEVELOPMENT
Department of Urban Development
and Building Construction

Phone: 4262945
4262365
Fax: 977-1- 4262439

Babar Mahal,
Kathmandu, Nepal

Ref.No.

ACKNOWLEDGEMENT



It gives me immense pleasure for the publication of Seismic Retrofitting Design Guidelines of Buildings in Nepal. This Guideline is the first attempt for the Government of Nepal to guide the respective practitioner and academicians for making the structure safer.

I expect that this guideline will be useful for the Designers as well as Engineers in general who are involved in retrofitting design and construction of buildings. This guideline will also be helpful in raising the safety awareness and making the community disaster resilient.

My sincere thanks go to the respected Secretary Mr. Deependra Nath Sharma, Joint Secretary Mr. Shiva Hari Sharma, Senior Divisional Engineer Mr. Pramod Krishna Karmacharya and all the personnel involved directly or indirectly for preparing of this design guideline.

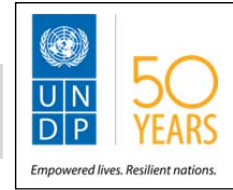
Also, my thanks go to United Nations Comprehensive Disaster Risk Management Program, CDRMP for the support during the preparation of the guideline and publication as well.

At last, but not the least, I would like to thank Center of Resilience Development (CoRD) and MRB Associates for their support and preparing this design guideline.

(Dr. Ramesh Prasad Singh)
Director General



FOREWORD



Nepal is home to the breathtaking Himalayas which as the world's youngest ranges are growing a few centimeters each year due to the uplift caused by the northward push of the Indian tectonic plate against the Eurasian plate. This manifests in large magnitude earthquakes recurring at a periodicity of 7 to 8 decades, and situates the entire country in a high seismic risk zone and its ranking as 11th in terms of its relative vulnerability to earthquake.

UNDP's Comprehensive Disaster Management Programme (CDRMP), taking cognizance of these risks and those posed by climate variability and change was formulated in 2011. It aims to strengthen the institutional and legislative sectors of Disaster Risk Management in Nepal by building the capacity of the key government ministries, its line agencies and local bodies. It also aims to enhance resilience in Nepal by strengthening partnership with national, institutional and the private sector, civil society and other development actors for Disaster Risk Management including Climate Change Adaptation.

Seismic Risk Reduction continues to be a key area of UNDP's collaboration with Government of Nepal from over two decades ago. UNDP contributed to preparation of National Building Code, the development of curricula and manuals for training of engineers and masons to implement provisions of the building code and implementation of code-compliant building permit systems in several municipalities in Nepal. Kathmandu Metropolitan City has recently fully operationalized the electronic Building Permit System with technical assistance from UNDP and funding support from UK Aid. Through Nepal Risk Reduction Consortium, UNDP co-led the formulation of a National Action Plan for Safer Building Construction.

Learning from our engagement on retrofitting of schools in Ilam and Taplejung districts post-2011 Sikkim earthquake, and realizing the need for strengthening existing vulnerable buildings, UNDP in collaboration with Department of Urban Development and Building Construction (DUDBC) of Ministry of Urban Development (MOUD) formulated the Retrofitting Guideline. This guideline was prepared to strengthen existing housing stock to cope with seismic shocks with technical support from Center of Resilient Development (CORD) and Manohar Rajbhandari Associates, and with active engagement of officials of DUDBC. The guideline encompasses three volumes addressing the three dominant construction typologies namely Adobe and low strength masonry, Masonry and RCC construction.

The April 2015 Gorkha Earthquake exposed the significant vulnerability of the existing buildings in both urban and rural areas and highlighted importance for such a guideline resulting in MOUD approving the guidelines in October this year. We hope this will help to undertake trainings and necessary repair, restoration and retrofitting of buildings damaged by April 2015 earthquake. It could also serve as a guiding document for practitioners, engineers and designers to undertake retrofitting measures to reduce risk to future disaster. This guideline is an initial yet important step as we embark on this journey to create safe and resilient buildings and settlements in Nepal.

I would like to extend my sincere acknowledgement to Mr. Shiva Hari Sharma, Joint Secretary, MOUD and Mr. Ramesh Prasad Singh, Director General DUDBC and their team for their valuable feedback and guidance throughout the process of formulation and approval of the guidelines. I would also like to thank Dr. Hari Darshan Shrestha, Dr. Jishnu Subedi and Mr. Manohar Rajbhandari for their technical support in preparation of this guideline, and acknowledge contributions of my colleagues at UNDP in this endeavor.

Valerie Julliaud
UNDP Resident Representative &
United Nations Resident Coordinator

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LIST OF ACRONYM

a_g	Peak ground acceleration
b	Thickness of brick units
CGI	Corrugated Galvanized Iron
CoRD	Center of Resilient Development
D	Displacement
D	PP- band mesh density
D_e	Effective displacement
	Development
D_{max}	Maximum displacement
D_u	Ultimate displacement
EMS	European Macroseismic scale
F_0	Force parameter
	Failure
	Failure mode
f_y	Characteristic strength of concrete
g_m	Unit weight of masonry wall
K_2	Secant stiffness
K_r	Overburden load
l	Collapse multiplier factor
LS	Limit state
MDOF	Multiple degree of freedom
MoHA	Ministry of Home affairs
r	Number of courses within the failing portion
R_d	Strength reduction factor
S	Soil factor
SDOF	Single degree of freedom
SL	Slenderness ratio
T	Natural Period
T_a	Period of vibration of the wall
UNDP	United Nations Development Programme
UNESCO	United Nations Educational, Scientific and Cultural Organization
USAID	United States Agency for International Development
V	Elastic base shear
V*	In plane shear force
V_c	Original structure strength
V_{dt}	Resistance corresponding to the diagonal tension
V_n	Nominal shear resistance
V_s	Resistance corresponding to the bed – joint sliding
V_{tc}	Resistance corresponding to the crushing failure mode
WWM	Welded Wire Mesh
x	Damping ratio
β	Number of edge and internal perpendicular walls
μ	Friction coefficient
μ_{dem}	Ductility demand
ω	Crack width

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1. INTRODUCTION

Nepal is one of the hotspots for disaster and is ranked as 11th most vulnerable country 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 for 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 and low strength masonry. 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 of 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, foundation 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 is 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 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 and on the adaptation of different techniques used in other countries through literature survey. Manual for Restoration and Retrofitting of Rural Structures in Kashmir prepared for UNDP/UNESCO and GoI by NCPDP, India has been referred to in preparation of this guideline. It includes the building typologies - adobe (earthen sun-dried bricks) with mud mortar, fired bricks in mud mortar and stone masonry buildings.

1.1 PURPOSE

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

1.2 OBJECTIVE AND SCOPE

The objective of this document is to reduce vulnerability of buildings thereby decreasing likelihood of 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.

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 and low strength masonry buildings.

1.3 CONCEPT OF REPAIR, RESTORATION AND RETROFITTING¹

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:

- 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 addresses 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.

¹ Adapted from LAEE Manual

- 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 and low strength masonry structures for sustaining design utilities.

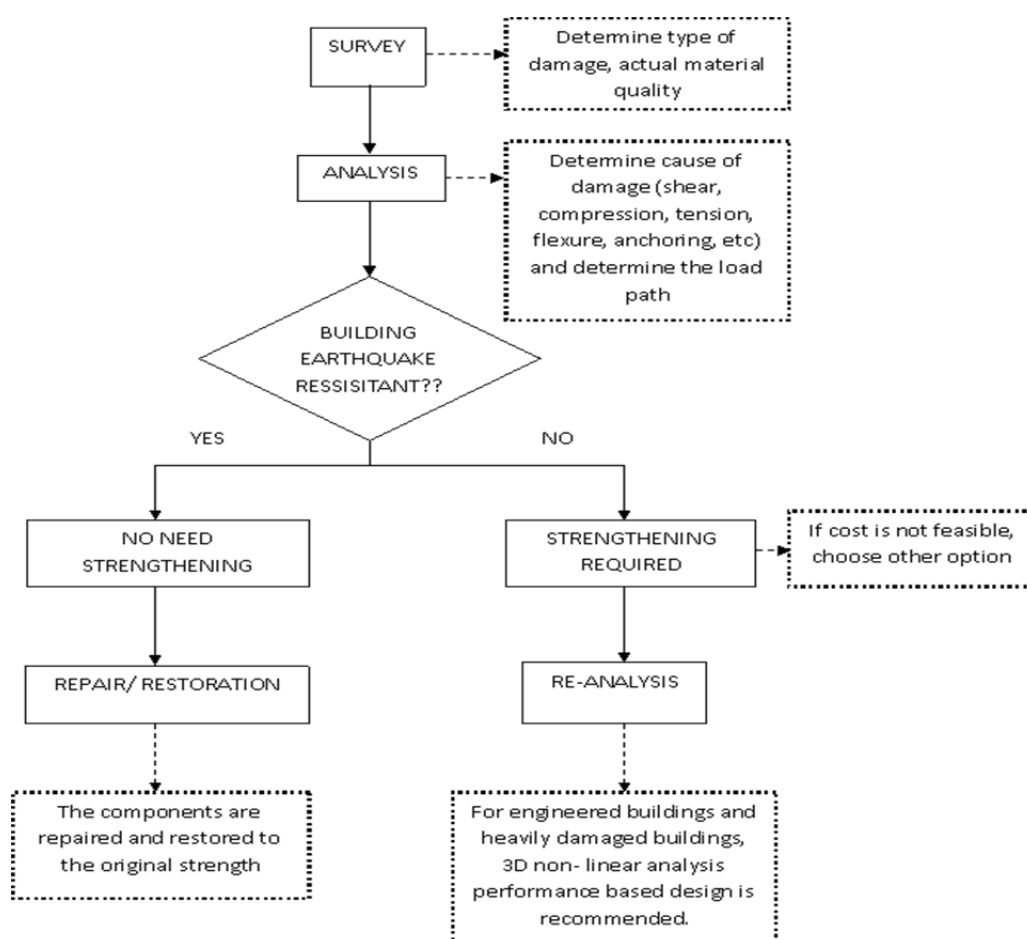


Figure 1.1 Stepwise Process of Seismic Retrofitting Of Building

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 confirm the considerable damage to 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.



Figure 2.1 Severe damage to adobe buildings in Chorrillos district in Peru earthquake 1974
(Source: www.earthquake.usgs.gov)



Figure 2.2 Damage of adobe houses in Guatemala City during Guatemala Earthquake 1976
(Source: irapl.altervista.org, figure 69-B, U.S. Geological Survey Professional paper 1002)



Figure 2.3 Collapsed adobe structures 2003 Bam Earthquake
(Source: www.worldhousing.net/whereport1view.php?ID=100130)



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 <ul style="list-style-type: none"> • 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 <ul style="list-style-type: none"> • 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 <ul style="list-style-type: none"> • 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 <ul style="list-style-type: none"> • Serious failure of walls (gaps in walls) • Partial structural failure of roof and floors
5	Grade 5: Destruction	Very heavy structural damage <ul style="list-style-type: none"> • Total or near total collapse of the building

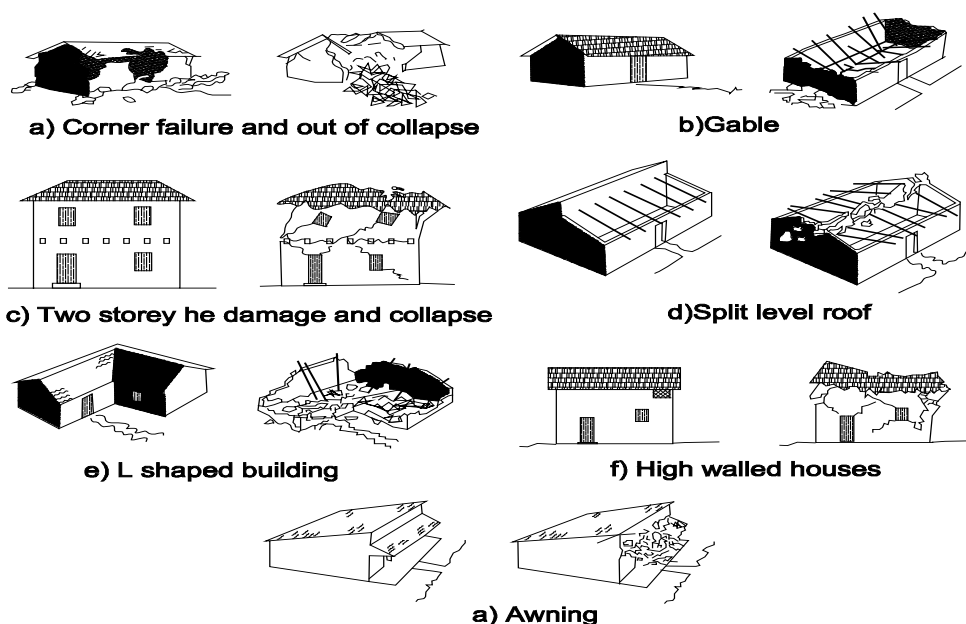


Figure 2.5 Damage Typology

(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.

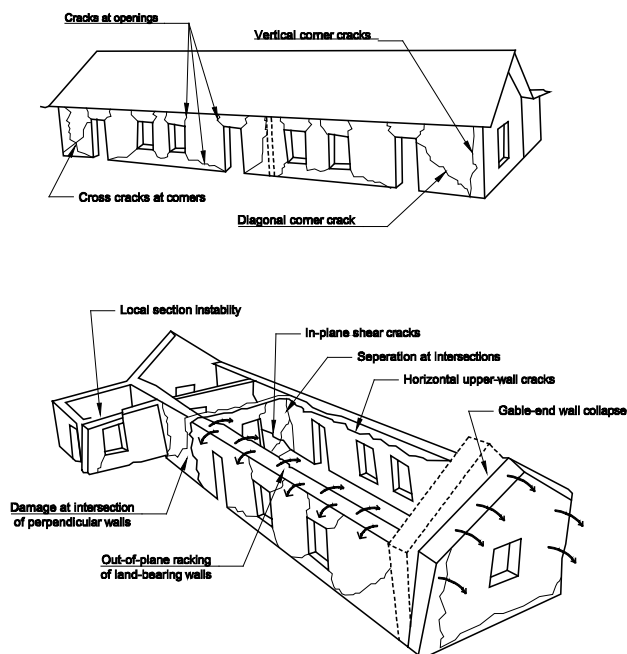


Figure 2.6 Typical damage modes observed in adobe buildings
(Source: Manual of The Getty Conservation Institute)

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

2.2.1. OUT OF PLANE WALL DAMAGE

Adobe walls are very susceptible to cracking from flexural stresses caused by out-of-plane ground motions. These cracks usually occur 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 0.2 g.

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



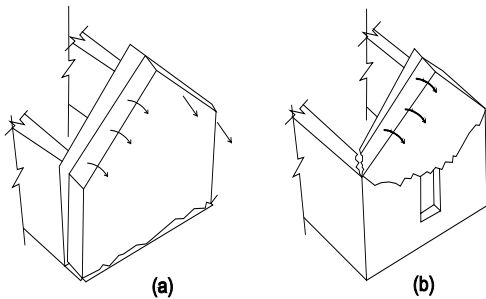
*Figure 2.7 Out-of-plane wall collapse – 1996
Nazca earthquake, Peru*
(Source: Report #52 in EERI/ IAEE World Housing Encyclopedia)



*Figure 2.8 Out-of-plane wall collapse after the 2007
Pisco, Peru earthquake*
(Photo: M. Blondet)

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.



*Figure 2.9 Gable-end wall collapse: (a) overturning at
base of wall, and (b) mid-height collapse*
(Source: Manual of The Getty Conservation Institute)



*Figure 2.10 Gable end wall
mid-height collapse*
(Sinam, Eastern Nepal Earthquake, September 1st, 2011)

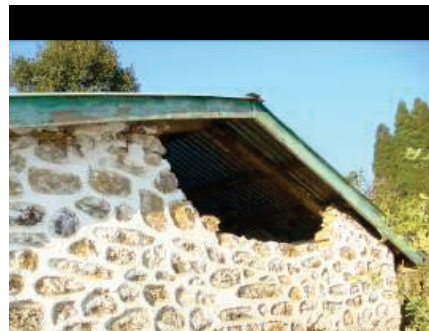


Figure 2.11 some other examples of Gable wall damage during Eastern Nepal Earthquake, 2011
(Photo: CoRD)

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.

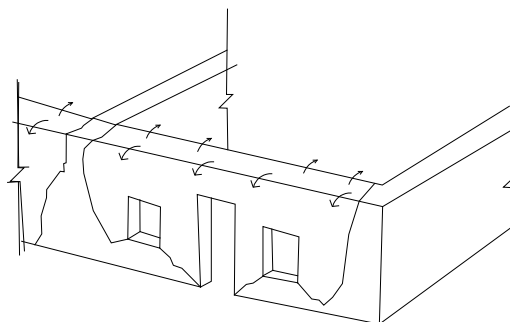


Figure 2.12 Out-of-plane flexure of load bearing wall
(Source: Manual of The Getty Conservation Institute)

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.

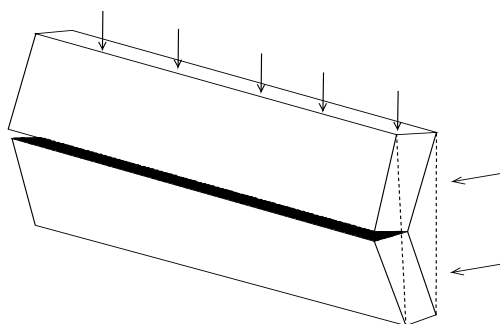


Figure 2.13 Sketch of mid-height out of plane failure
(Source: Manual of The Getty Conservation Institute)



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.

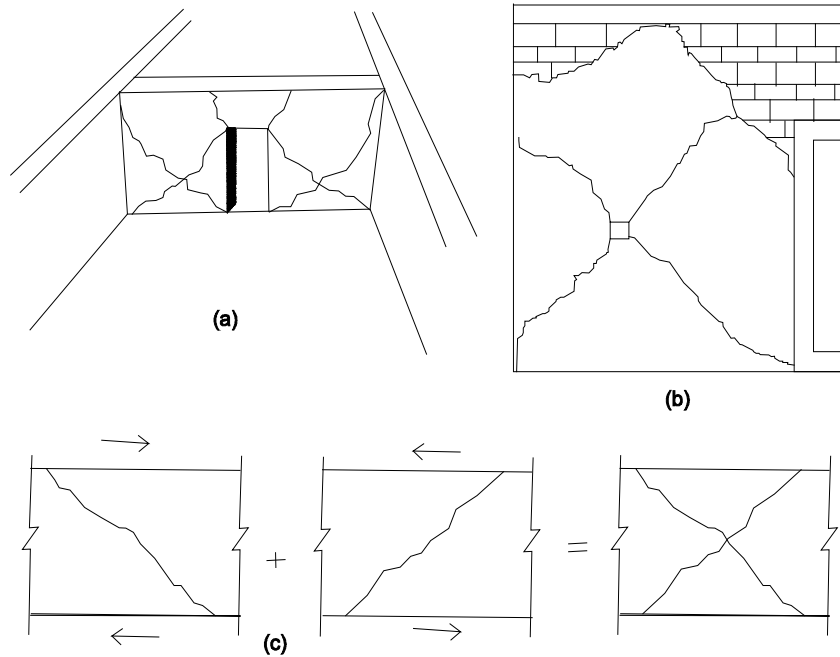


Figure 2.15 Illustrations show (a) drawing of X-shaped shear cracks in an interior wall; (b) typical X pattern (Leonis Adobe, Calabasas, Calif.); and (c) how X-shaped cracks result from a combination of shear cracks caused by alternate ground motions in opposite directions.

(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.

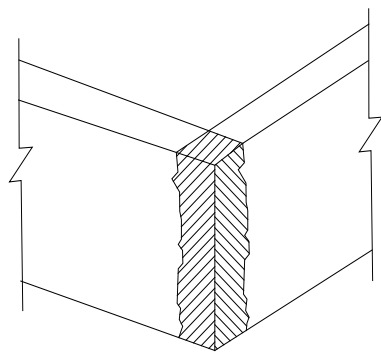


Figure 2.16 Illustrations showing (a) how vertical cracks at corner can lead to instability of intersection
(Source: Manual of The Getty Conservation Institute)



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).



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.

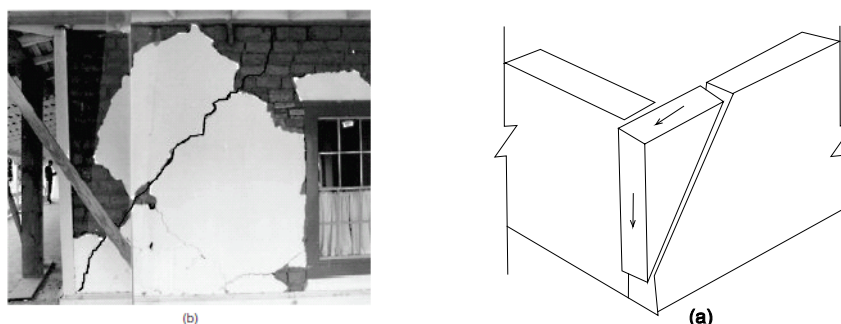


Figure 2.20 Corner cracks: (a) illustration of vertical downward and horizontal displacement of a corner wall section, and (b) example of displaced wall section (Leonis Adobe)
(Source: Manual of The Getty Conservation Institute)

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.

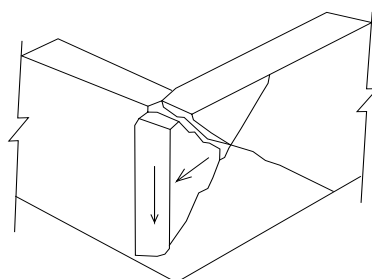


Figure 2.21 Illustration showing how combination of shear and flexural cracks can result in corner displacement or collapse
(Source: Manual of The Getty Conservation Institute)

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 2.23 (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.

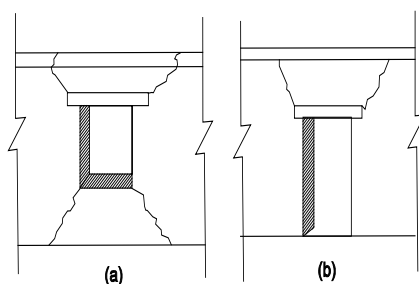


Figure 2.22 Illustration of cracks originating at stress concentration locations: (a) cracks appearing first at upper corners of window opening followed by lower corner cracks; and (b) cracks at upper corners of door opening.

(Source: Manual of The Getty Conservation Institute)

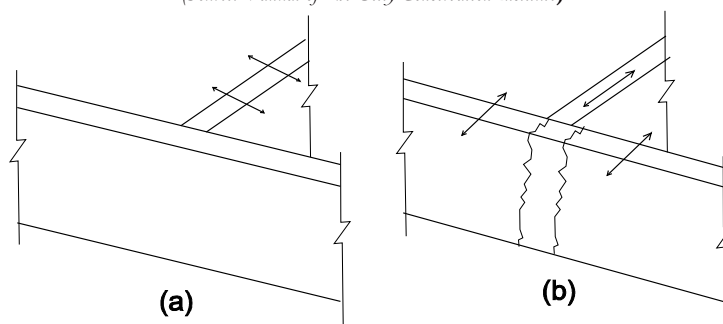


Figure 2.23 Illustrations showing (a) how separation can occur between in-plane and out-of-plane walls, and (b) how vertical cracks develop in out-of-plane walls at the intersection with perpendicular, in-plane walls.

(Source: Manual of The Getty Conservation Institute)



Figure 2.24 Some examples of Cracks at opening

(Source: CoRD)

3. VULNERABILITY ASSESSMENT OF EXISTING BUILDINGS

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 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.

² Adapted from LAEE Manual

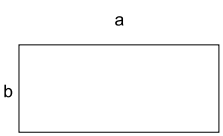
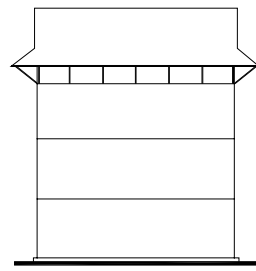
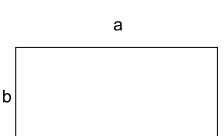
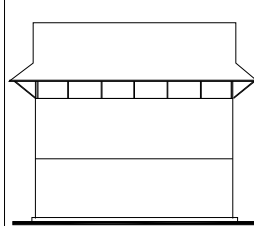
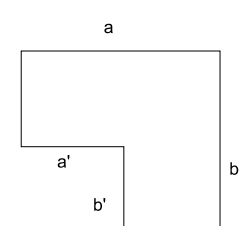
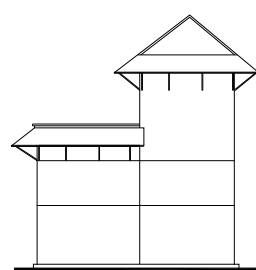
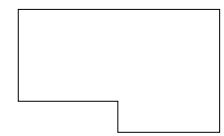
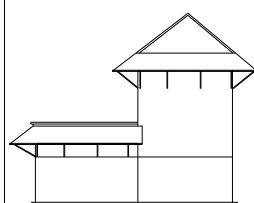
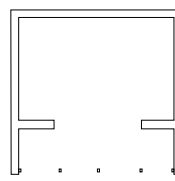
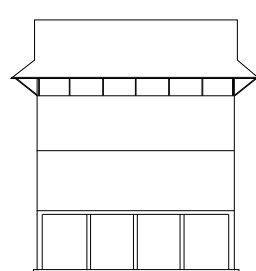
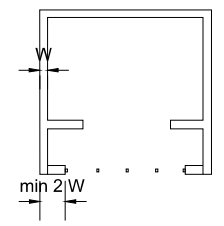
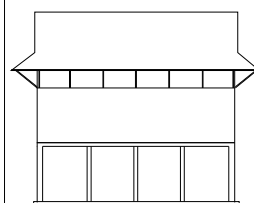
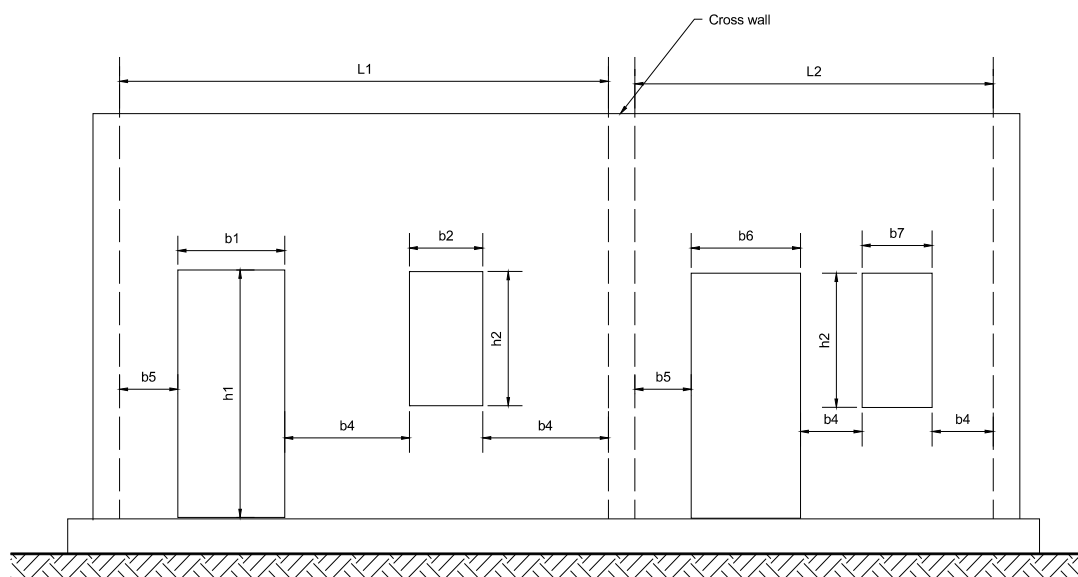
NO		YES	
PLAN	ELEVATION	PLAN	ELEVATION
 <p>$a \geq 3b$</p>		 <p>$a \leq 3b$</p>	
 <p>$a = b$ $a' = b'$</p>			
	 <p>MORE THAN 3 STOREYS</p>	 <p>min 2W</p>	 <p>MAXIMUM STOREYS 2 + ATTIC</p>

Figure 4.1 Recommended forms of buildings
(Adapted from NBC 203)



RECOMMENDATION REGARDING OPENINGS IN LOAD BEARING WALLS

NOTE:

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

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

$b4 \geq 0.5 h2$ but not less than 600 mm.

$b5 \geq 0.25 h1$ but not less than 450 mm.

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.

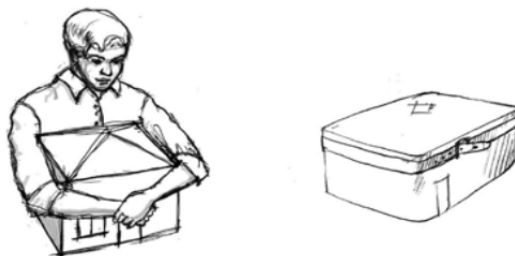


Figure 4.3 A seismic band acts like a belt
(Adopted from: GOM 1994)

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 building³ 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.

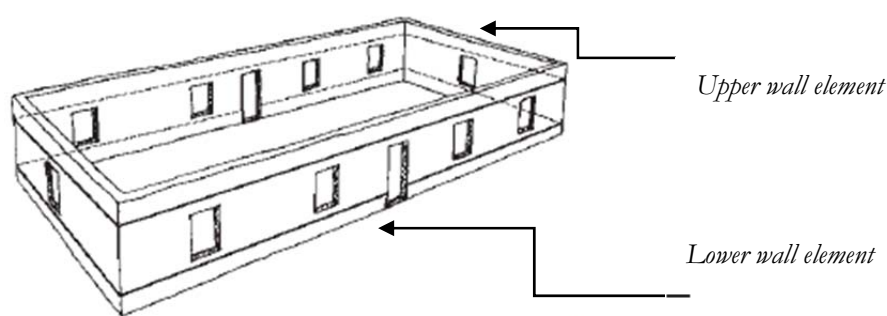


Figure 4.4 Seismic belt showing upper and lower wall elements
(Source: Manual of The Getty Conservation Institute)

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.

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

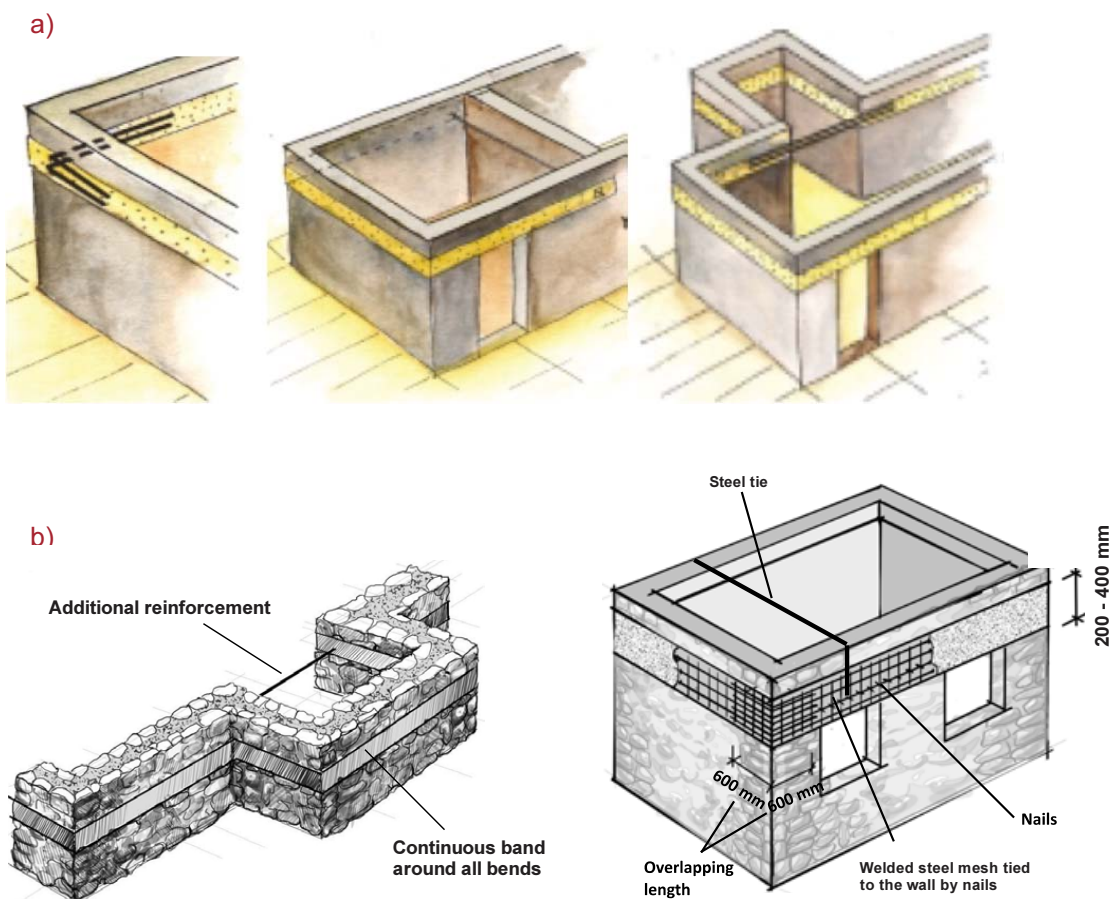


Figure 4.5 a) Seismic belts around various locations (Source: UNDP, UNESCO & GOI, 2007)
 b) Additional reinforcement (Source: R. Desai and GOM 1998)

Steps for construction of the belts:

Mark the location of belts and remove plaster in the marked 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.

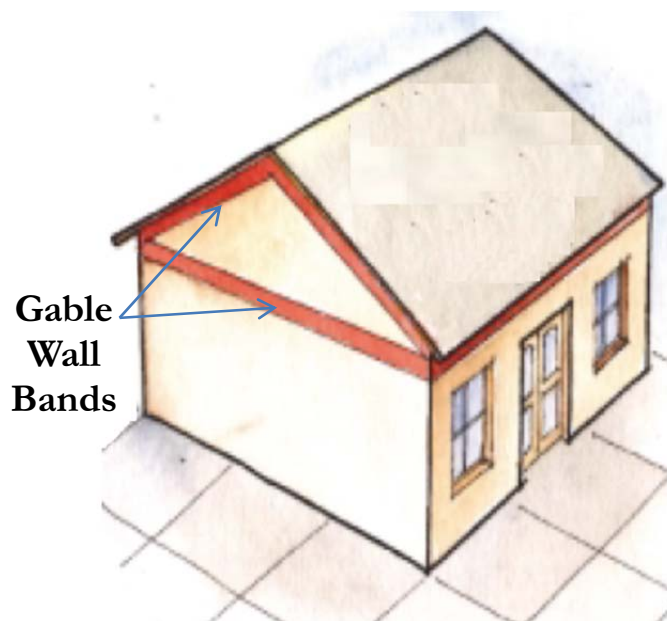


Figure 4.6 Gable band

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 cracks⁴, 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.

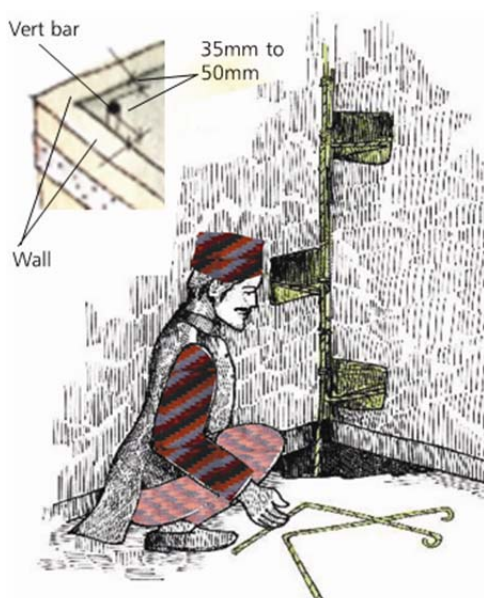


Figure 4.7 Vertical bar in corner
(Adapted from: UNDP, UNESCO & GOI, 2007)

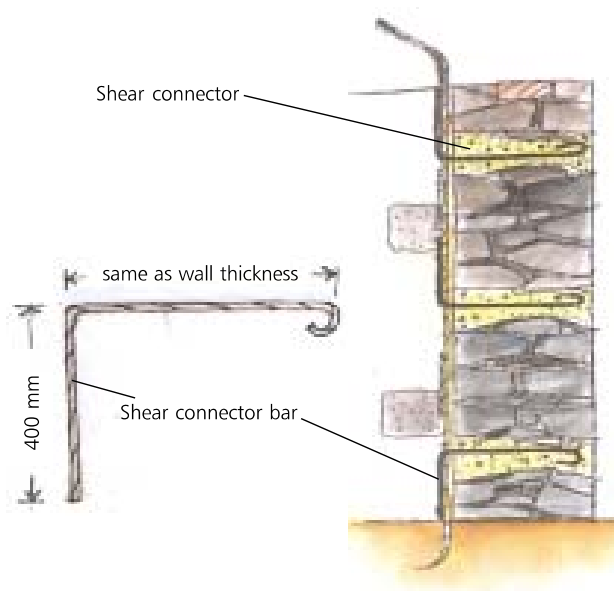


Figure 4.8 Shear connector with vertical bar details
(Source: UNDP, UNESCO & GOI, 2007)

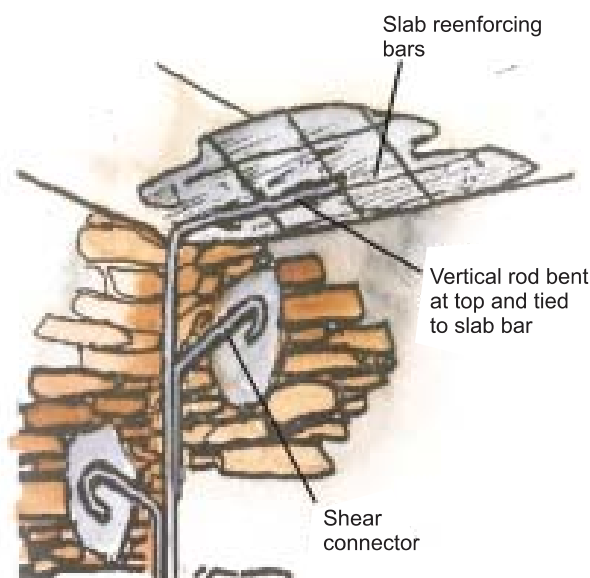


Figure 4.9 Connecting top bent end of vertical rod to slab
(Source: UNDP, UNESCO & GOI, 2007)

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.

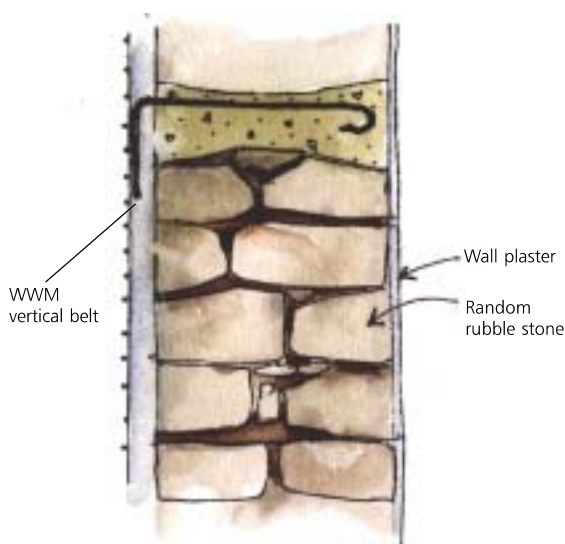


Figure 4.10 Anchoring WWM to random rubble wall with shear Connector
(Source: UNDP, UNESCO & GOI, 2007)



Figure 4.11 Plastering vertical WWM belt using cement plaster
(Adapted from: UNDP, UNESCO & GOI, 2007)

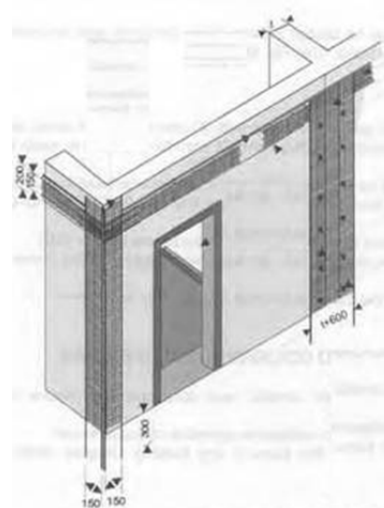
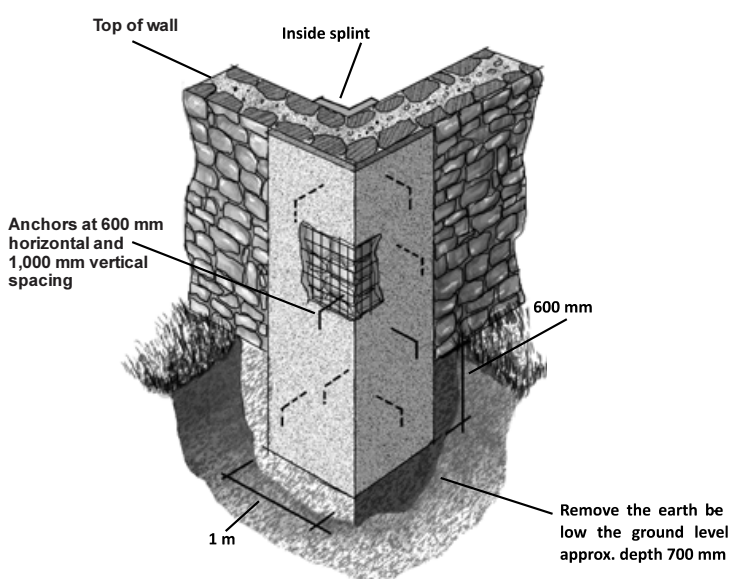


Figure 4.12 Vertical reinforcement with WWM
(Source: GOM 1998)

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, UNESCO & 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 4.15 PP band mesh (Source: Meguro and Mayorca)



Figure 4.16 PP band retrofitted house before mortar laying (Source: Meguro and Mayorca)

Design Methodology:⁵

1. Determine the original structure strength, V_c , and natural period, T .
2. Calculate the elastic base shear, V , according to the regional seismic code.
3. From the relation between V and V_c , estimate the strength reduction factor, R_d .
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

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

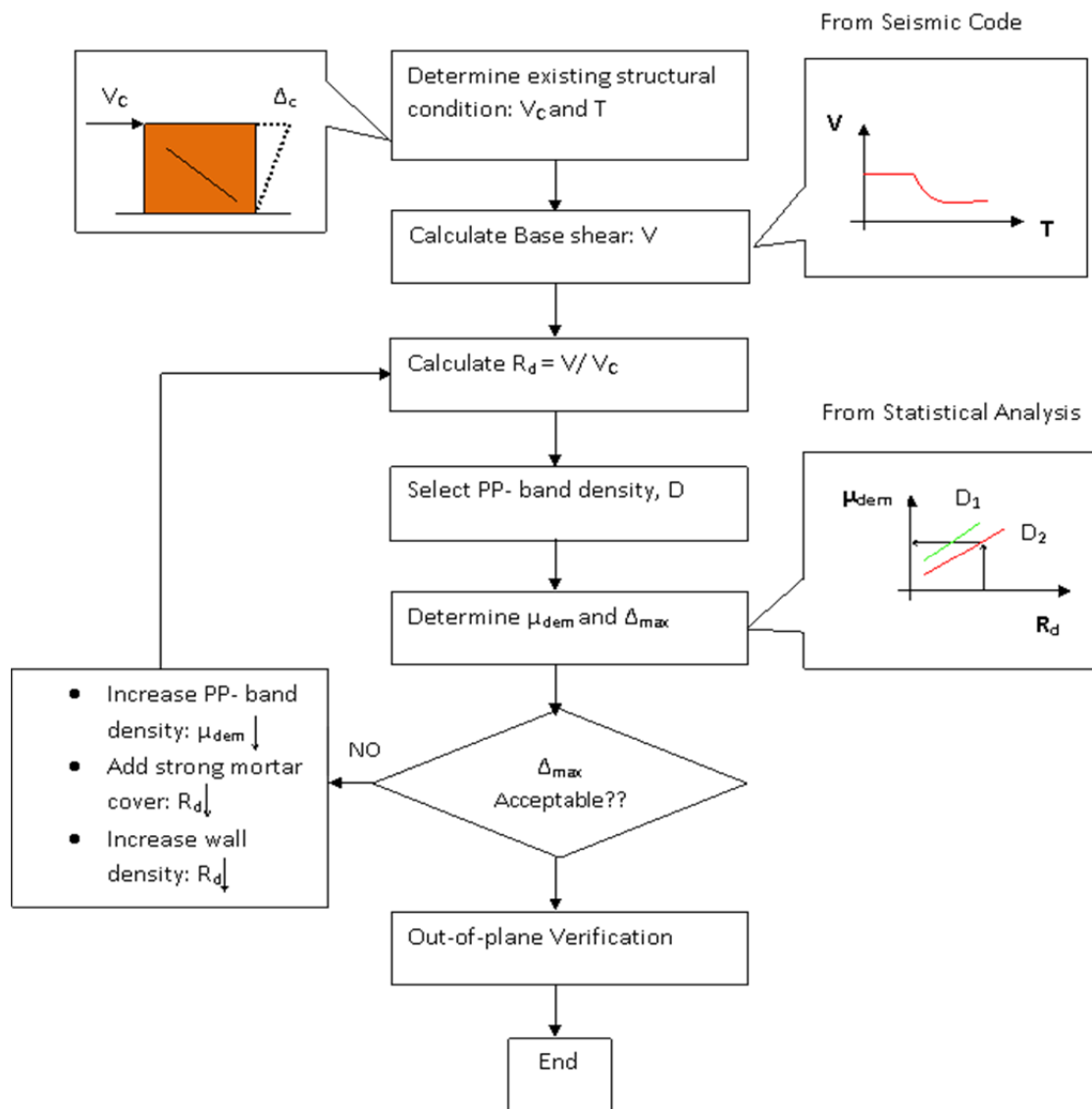


Figure 4.17 Flow chart of Design of PP band (Wall Jacketing)

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 shows that this method has significantly improved seismic resistance of the adobe structures. A timber ring beam is also included in this structure.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.

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

A simple construction procedure of this technique is presented below:

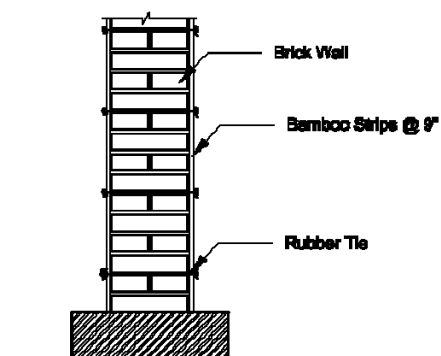


Figure 4.19 Plan showing bamboo reinforcing

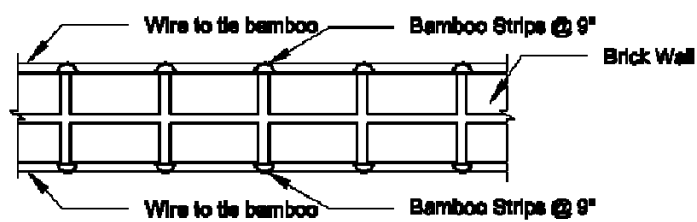


Figure 4.18 Wall section showing bamboo reinforcing





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 4.21 Cane reinforcement

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

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.

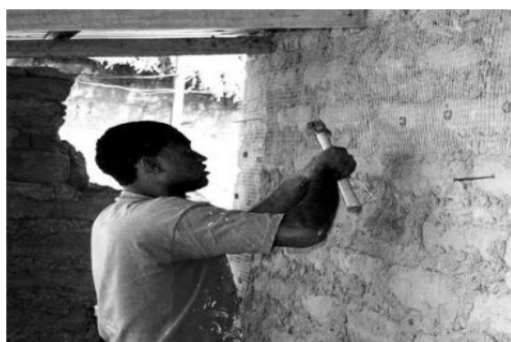


Figure 4.22 Placing the mesh on the wall

(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)

⁷Earthquake 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 $15 \times 20 \text{ mm}^2$, 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 ($0.8 \times 0.6 \text{ mm}^2$), 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.

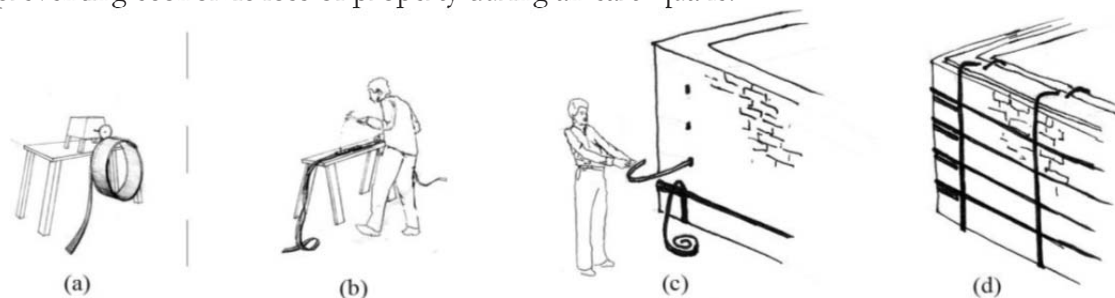


Figure 4.25 Steps in the process of reinforcing an earthen (adobe) house with tire straps.

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.

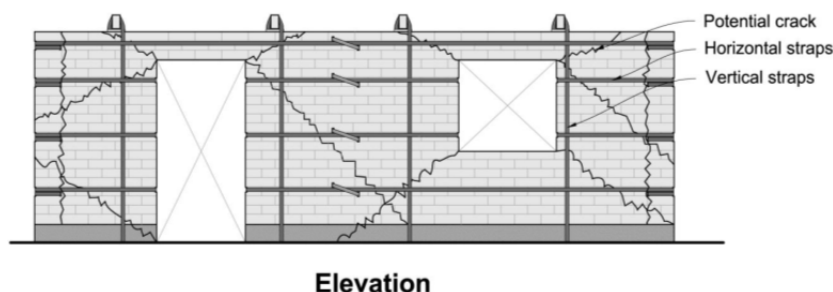


Figure 4.26 An elevation of a typical wall showing positions of expected cracks and strap

(Source: *Seismic Strengthening of Earthen Houses Using Straps Cut from Used Car Tires: A Construction Guide*, Andrew Charleso)

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

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 follows⁹

a) *Installing new steel straps:*

New steel straps can be installed to connect the exterior walls to a timber floor, as shown in Figure 4.27 (a) (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 4.27 (b). 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 4.28 b).

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 4.29.

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.

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

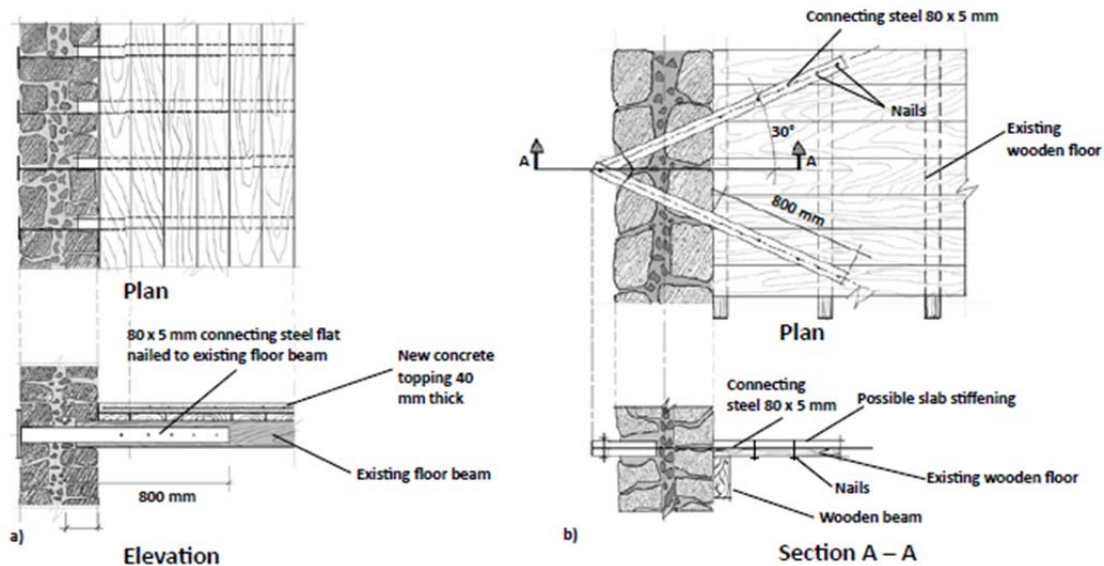


Figure 4.27 Steel straps for wall-to-floor anchorage: a) floor beams perpendicular to the wall, and b) floor beams parallel to the wall
(Source: UNIDO 1983)

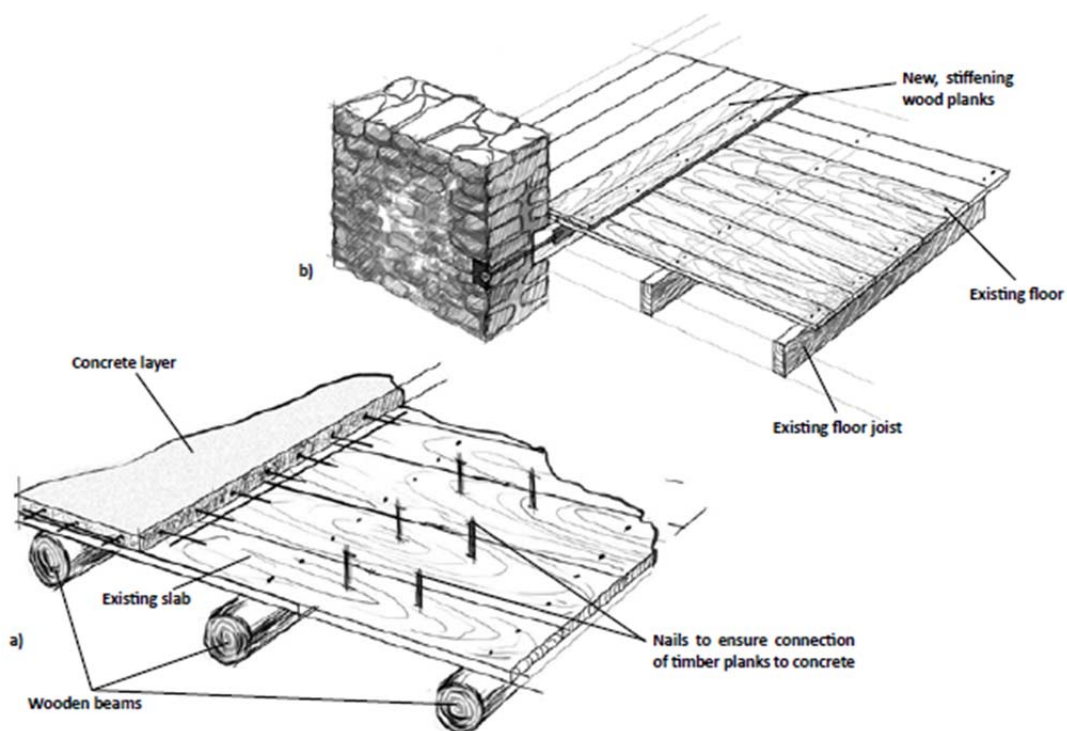


Figure 4.28 Stiffening the floor structures: a) RC topping, and b) new timber planks
(Source: UNIDO 1983)

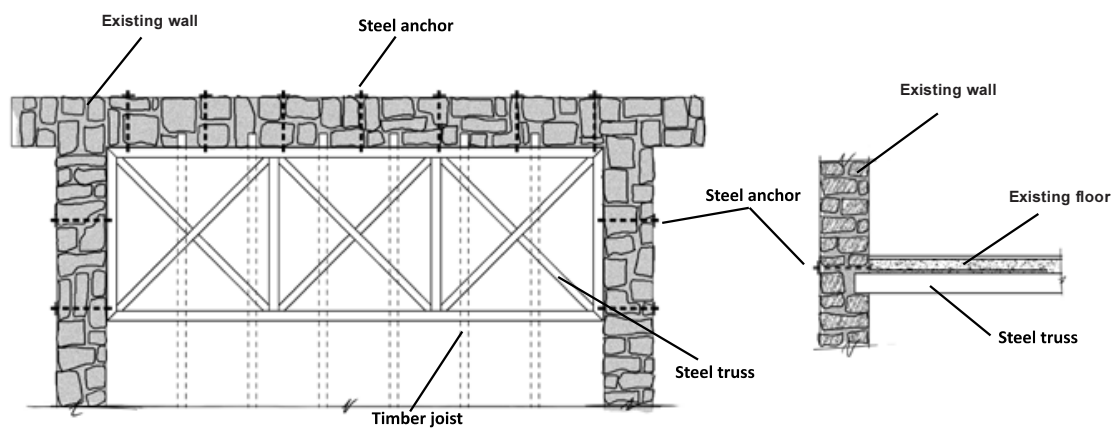


Figure 4.29 Retrofitting the floor and roof structures: a) diagonal brace
(Adapted from: Tomazevic 1999)

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.

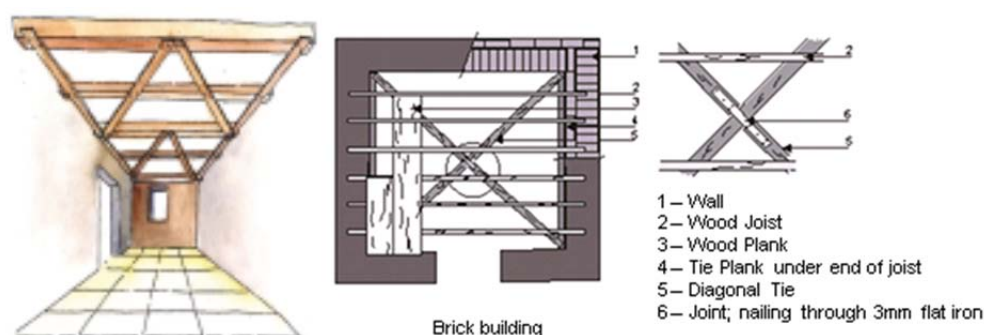


Figure 4.30 Stiffening flat wooden roof



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 4.32). Also the collars should be provided to prevent roof spreading (Figure 4.33). The important point in retrofitting is the provision of seismic belts just below eave level and the gable level.

Note-1

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

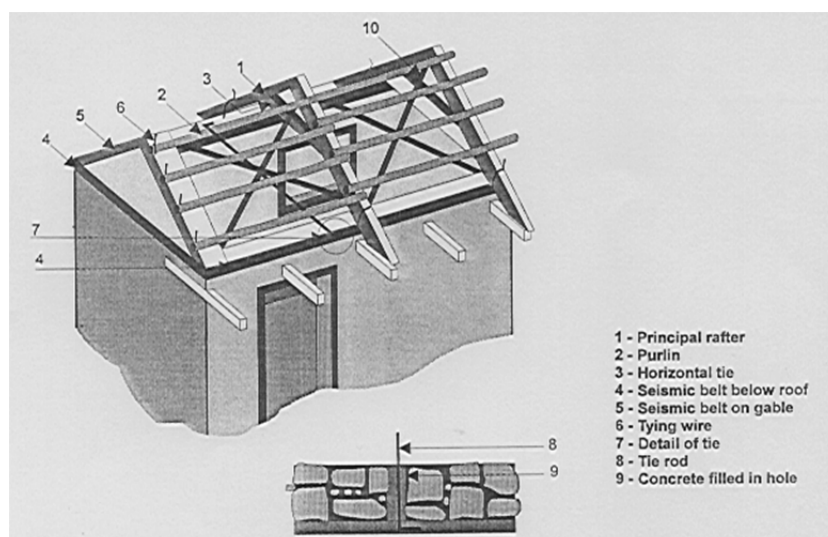


Figure 4.32 Stiffening of sloping roof structure

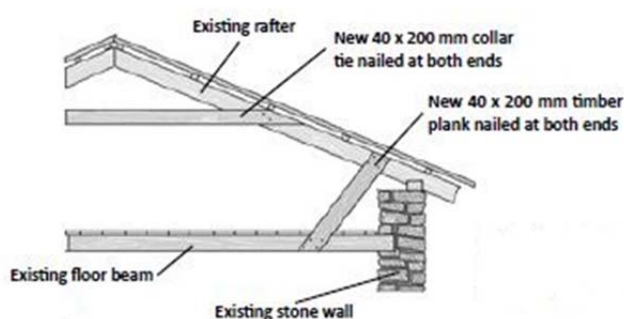


Figure 4.33 Roof rafters tying to ceiling joist



Figure 4.34 Example
(Source: Santosh Shrestha)

4.4 STRENGTHENING OF FOUNDATION

4.4.1 STRENGTHENING FOUNDATION¹⁰

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

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 4.35 a.

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 4-35 b.

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.

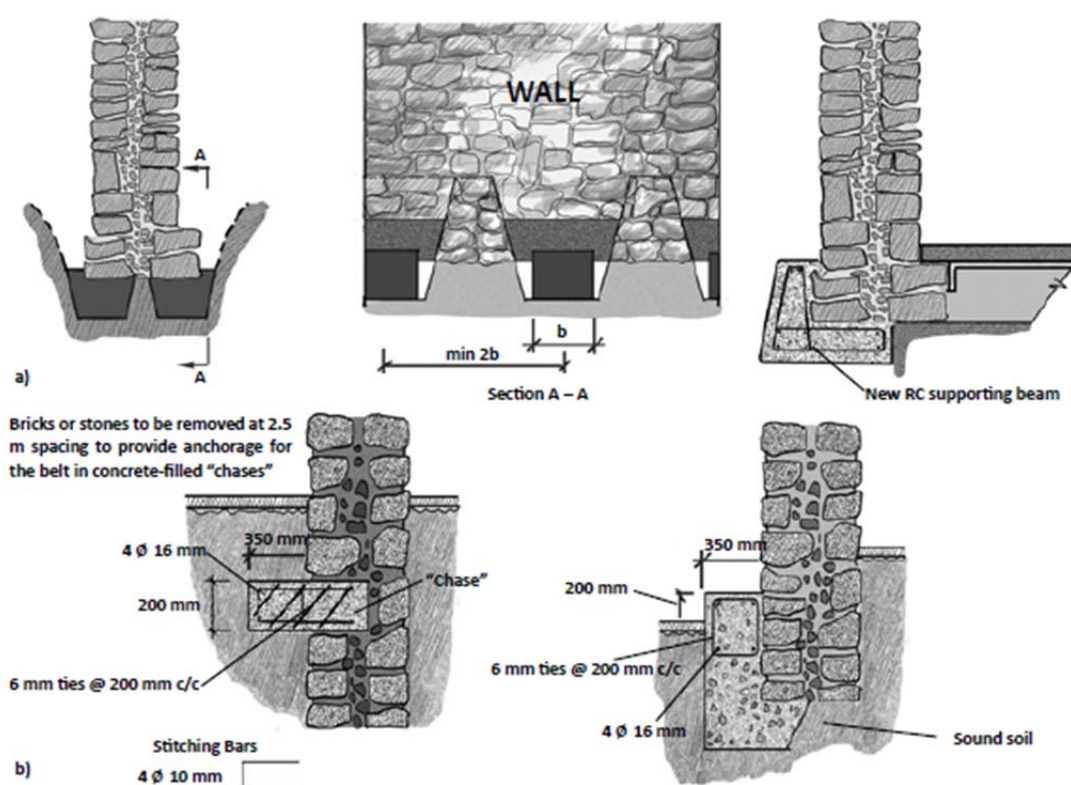


Figure 4.35 Strengthening existing foundations: a) underpinning the foundation, and b) external RC belt
(Adapted from: GOM 1998 and UNIDO 1983)

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

4.4.2 CONTROL ON DOOR AND WINDOW OPENINGS IN MASONRY WALLS

4.4.2.1 INFILL OPENINGS¹¹

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 opening¹²

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.

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.

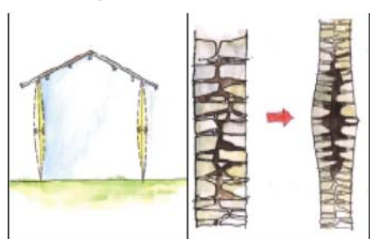


Figure 4.36 Bulging of wall wythes
(Source: UNDP, UNESCO & GOI, 2007)



Figure 4.37 Delamination

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

Installing cast in-situ reinforced concrete bond element:



Figure 4.38 Marking points for bond elements



Figure 4.39 Removing stone by small rod

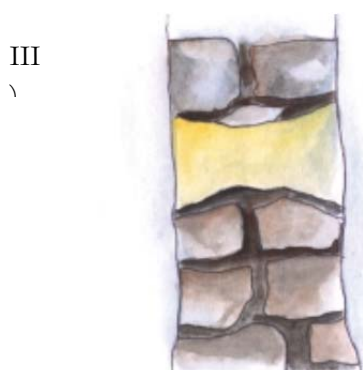


Figure 4.40 Making dumb-bell shaped hole through wall



Figure 4.41 Placing steel bar and filling concrete

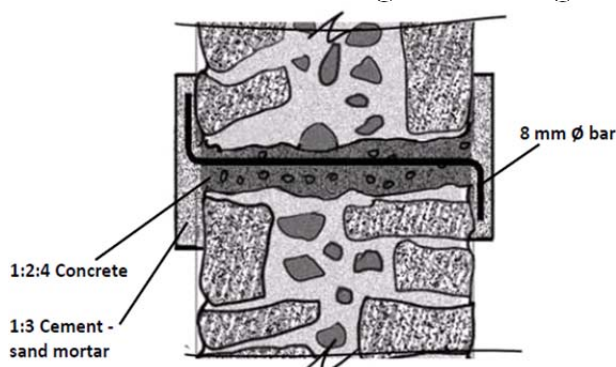


Figure 4.42 Cross section of through-stone
(Source: UNDP, UNESCO, GOI, 2007)

The installation of through-stones is labor-intensive, but it may be a feasible retrofit option for stonemasonry 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 4.38 through 4.43.



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, UNESCO & 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 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 cracks¹⁵

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.

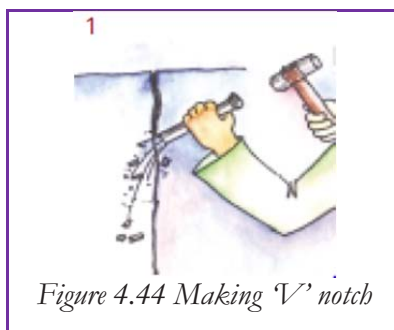


Figure 4.44 Making 'V' notch

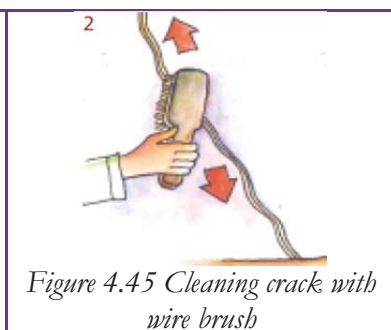


Figure 4.45 Cleaning crack with wire brush

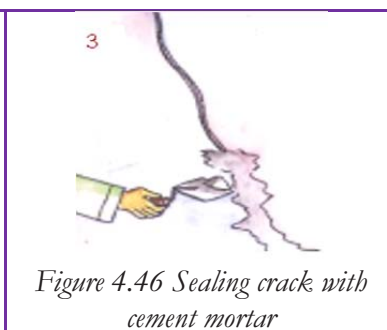


Figure 4.46 Sealing crack with cement mortar

(Source: UNDP, UNESCO & GOI, 2007)

¹³Grouting as a repair/strengthening solution for earthconstructions Rui A. Silva, Luc Schueremans, Daniel V. Oliveira

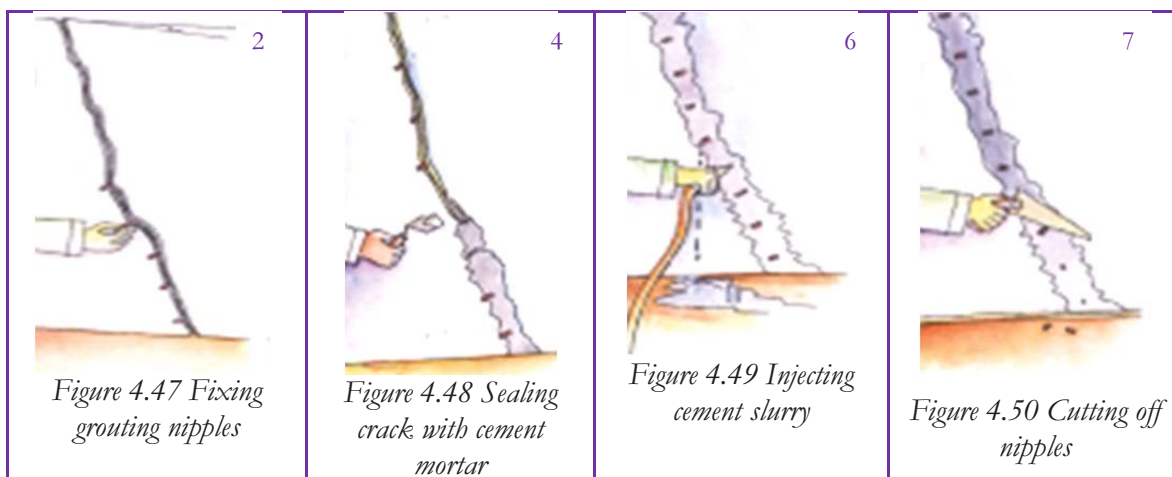
¹⁴L. Keefe: Earth Building Methods and materials, repair and conservation, Taylor & Francis, 2005, London, UK.

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

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.

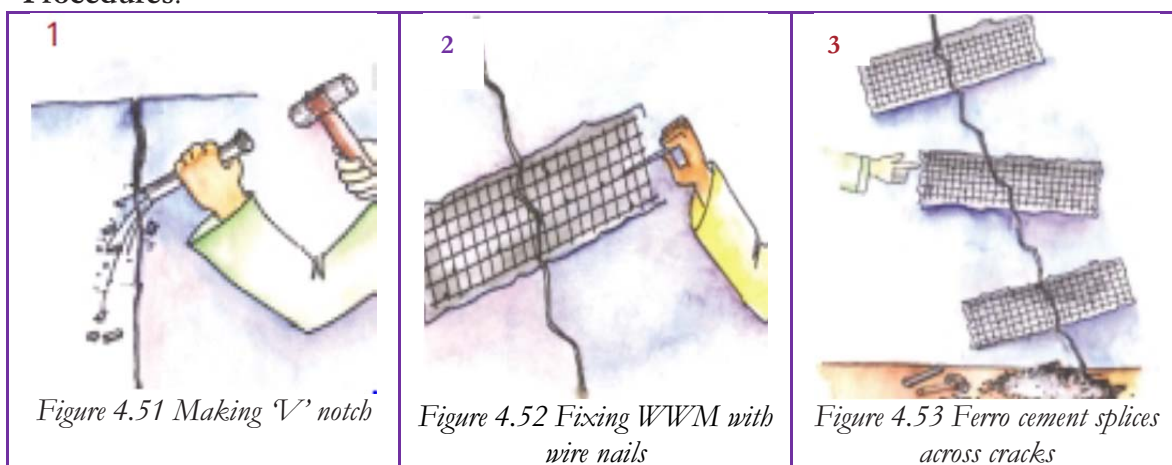


(Source: UNDP, UNESCO, GOI, 2007)

- 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:



(Source: UNDP, UNESCO, GOI, 2007)

- 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.



Figure 4.54 Applying cement plaster on splices



Figure 4.55 Curing

(Source: UNDP, UNESCO, GOI, 2007)

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.

4.5.2 SEALING OF FINE CRACKS¹⁶

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 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:

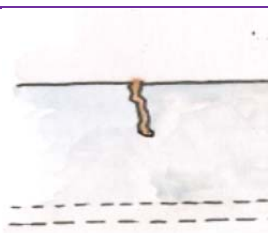


Figure 4.56 Cracks

(Source: UNDP, UNESCO, GOI, 2007)

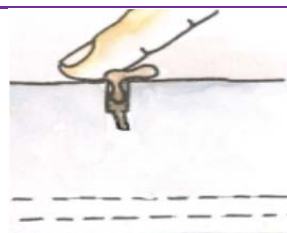


Figure 4.57 Sealing cracks with M-seal

(Source: UNDP, UNESCO, GOI, 2007)

- 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.

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

ANNEXES

SEISMIC VULNERABILITY EVALUATION GUIDELINE FOR PRIVATE AND PUBLIC BUILDING

A.I.1 INTRODUCTION

General

This guideline is for assisting professionals and the authorities in Nepal to implement qualitative and quantitative assessment of structural earthquake vulnerability of public and private buildings in Nepal. It is based on the experiences gained in Nepal in conducting visual qualitative as well as quantitative assessment of structural vulnerability of about 20 major hospitals and about 600 schools. This guideline is rather based on the adaptation of different available methodologies to the local conditions of Nepal, than on the fundamental research. Efforts have been made to simplify the procedures described in this guideline. It provides step by step suggestions on the procedure of carrying out the seismic vulnerability assessment.

Basis and Scope

This Guideline is targeted mainly for the civil-engineers and technicians who are involved in seismic vulnerability assessment of buildings. However, the government authorities, disaster risk managers and the policy makers, who are concerned with the safety of public and private buildings, may also use this guide line.

There are two phases of seismic vulnerability assessment. The *qualitative assessment* is for planning purpose and for identifying the priorities of intervention in the single building or the buildings complex. The *quantitative assessment* is for identification of retrofitting option and to examine the extent of intervention that would require in the building with consideration of technical, economic and practical feasibility. This guideline includes only some methods of detailed assessment and it is not meant to replace other method so detailed structural vulnerability assessment.

Guide line Dissemination

The guideline has the potential to improve the current situation of earthquake vulnerability of our community if appropriately implemented by concerned authorities. This guideline should reach to engineers and practitioners who are working in the construction field. They should use this document effectively and efficiently.

Guidelines are more likely to be effective if they are disseminated by an active tutoring. The distribution of printed guide lines alone is found to be ineffective in achieving expected change in practice. Hence, to ensure better understanding and best use of the guideline, training for the users is recommended.

Guidelines must obviously be made as widely available as possible in order to facilitate implementation. It is necessary to have wide circulations among engineers and practitioners working in the field of earthquake engineering. It thus requires an integrated effort by the concerned authorities like local government, municipalities, NGO's, INGO's and other related organizations towards dissemination of publication in wider range.

Further, dissemination and implementation of a guideline should be monitored and evaluated. The guideline also needs thorough review by experts in the field. This should undergo mandatory updating procedure to transform it in to pre-standard and then to building standard.

A.I.2 APPROACHES FOR DATA COLLECTION FOR VULNERABILITY ASSESSMENT

Physical Surveys

Acquisition of building data pertaining to the building is the first step in the evaluation process. The data shall be obtained preferably prior to the initial site visit and confirmed later during the visit. Construction documents like as-built drawing and Structural drawing shall be required for preliminary evaluation. Site condition and soil data shall also be collected if possible.

However, if these documents are not available prior to the visit, all necessary information shall be collected during the site visit. The general information required is about building dimensions, construction age, and description of structural system (framing, later all load resisting system, diaphragm system, basement and foundation system).

During the visits, the investigation of the interior of the structural members may require. In many buildings the structure is concealed by architectural finishes, and the inspector may need to get into attic, crawl over the spaces, and plenums to investigate. Some intrusive testing may require for determining the quality of material and allowable stresses. Even if structural drawings are available, some exposure of critical reinforcement may be necessary to verify conformity with the drawings. Photographs of building exterior and interiors may also be useful for the evaluation.

The evaluation should be based on facts, as opposed to assumptions, to the greatest extent possible. However, prudent engineering judgment may avoid huge efforts and cost of detailed investigation.

Interaction with Public Building Staff and Building Owners

Generally it is difficult to obtain as built or design drawings for most of the public buildings. For the private buildings also, the structural drawings are generally not prepared or are not available. Therefore, it is necessary to interact with the public buildings authorities and other staff for the public buildings and to the house owner for private buildings. It is also necessary to involve them in the process to get their buy-in on the outcome of the assessment and, more importantly, on the proposed mitigation actions, in case of public buildings. This approach will also help in sensitizing authorities and raising awareness of staff on the seismic safety issue. This is very important, as there is general lack of awareness and commitment on the issue. The approach with following considerations is, thus, suggested for effective evaluation, which induces the development and implementation of doable mitigation actions.

- The assessment shall not solely rely on secondary information and shall involve primary data collection and confirmation of available information with the active participation of the authority and owners. The authority shall also be involved in the process of identification of mitigation options.
- The assessment work shall be taken as an awareness raising and educative tool to promote overall earthquake safety of buildings as well as collective safety of personnel.

A.I.3 QUALITATIVE STRUCTURAL ASSESSMENT

Introduction

This chapter describes the preliminary evaluation processing general terms. Seismic Evaluation of an existing building shall be conducted in accordance with the process outlined below. This evaluation process is performed to determine whether the building, in

its existing condition, has the desired seismic performance capability. A method basically involves review of available drawings and visual evaluation of the building for potential damage it could suffer in the even to fan earthquake. It checks the code compliance for seismic design and detailing. The process is basically a qualitative measure to identify the areas of seismic deficiencies in a building before a detailed evaluation. This will help in deciding the retrofitting requirements for the building.

Assessment of the Building

Qualitative structural assessment of the building shall be done based on review of all available documents and drawings pertaining to the design and construction, design details and visual observation during site visit. If no documents are available, an as built set of drawing shall be prepared indicating the existing lateral force resisting system. If there cords are not available, an attempt can be made to obtain some in formation based on interviews with those who were involved in the design and construction of the building or familiar with the contemporary method so construction, and the owners/residents. Different seismic vulnerability factors are checked and expected and performance of the building is estimated for different earthquake intensities. Different steps of the assessment process and their outcomes are described in this section.

Identification of Seismicity of the Region

The region of seismicity of the building shall be identified. This is done locating the building in seismic hazard map of the region in which the building stands. The zone map of Nepal is provided in Nepal National Building Code NBC105:1994.

Establish Seismic Target Performance Level

Desired performance level of protection is established prior to conducting seismic evaluation and strengthening. These are classified as:

- Operational
- Immediate occupancy
- Life safety
- Collapse Prevention

A wide range of structural performance level could be desired by individual building owners. The basic objective should be to increase Life Safety Performance Level-reducing the risk of life loss in the largest expected earthquake. Buildings meeting the Life Safety performance level are expected to experience little damage from relatively frequent, moderate earthquakes. But significantly more damage and potential economic loss from the most severe and infrequent earthquakes could affect them. Only the buildings classified as essential facilities (such as hospitals or other medical facilities, fire or rescue and police stations, communication centers, emergency preparedness centers, etc.) should be evaluated for Immediate Occupancy Performance Level.

Obtain As-Built Information

Available as-built information for the building shall be obtained and site visit shall be conducted. Information of the building such as age of building, use, soil type and geological condition, structural system, architectural and structural characteristic, presence of earthquake resistant elements and other relevant construction data are to be collected from the archives. Standard checklists shall be prepared for this purpose.

If architectural and structural drawings are not available, evaluation may become difficult as the building structure is usually concealed by architectural finishes. Even if the drawings and structural details are available, it is necessary to verify conformance to the details at site. The structural design engineer, the contractor and the house owner should be consulted, if possible. Building information can be obtained by any of the following processes.

Site Visit: A site visit shall be conducted by the evaluating design professional to verify existing data or collect additional data, determine the general condition of the building and verify or assess the site condition.

Interview: Interviews should be conducted with knowledge able people residing in or nearby the buildings, with those who were involved in the design and construction of the building or with older engineers who have knowledge of contemporary method so construction in the community or region to understand the building history, used construction materials, construction technologies, and alterations in the buildings as well as general aspects of the building.

Material exploration: For a proper evaluation, the actual condition of the building is to be assessed. The lateral force resisting system should be established. This can be done by implementing non-destructive test such as the use of bar scanner, test hammers and Ultrasonic testing instruments or by destructive tests as drilling in walls, scrapping of plasters and making inspection holes, if necessary, to determine the structural system and the expected strength of structural elements.

Building Typology Identification

The building being evaluate dis-identified by type of structural system listed in tabular form below. This is based on the lateral force resisting system and the type of diaphragm. A building with more than one type of lateral-force-resisting system shall be classified as a mixed system. A fundamental to this analysis is the grouping of buildings in to sets that have similar behavioral characteristic.

Table A.I.1: Common Building Types in Nepal

No.	Building Types in Kathmandu Valley	Description
1	Adobe, stone in mud, brick-in-mud (Low Strength Masonry).	<p><i>Adobe Buildings:</i> These are buildings constructed in sun-dried bricks (earthen) with mud mortar for the construction of structural walls. The wall thickness is usually more than 350 mm.</p> <p><i>Stone in Mud:</i> These are stone-masonry buildings constructed using dressed or undressed stones with mud mortar. These types of buildings have generally flexible floors and roof.</p> <p><i>Brick in Mud:</i> These are the brick masonry buildings with fired bricks in mud mortar</p>
2	Brick in Cement, Stone in Cement	These are the brick masonry buildings with fired bricks in cement or lime mortar and stone-masonry buildings using dressed or undressed stones with cement mortar.
3	Non-engineered Reinforced Concrete Moment-Resisting-	These are the buildings with reinforced concrete frames and unreinforced brick masonry in fill in cement mortar. The thickness of in fill walls is

No.	Building Types in Kathmandu Valley	Description
	Frame Buildings	230mm (9") or 115mm (4 1/2") and column size is pre dominantly 9"x9". The prevalent practice in most urban area of Nepal for the construction of residential and commercial complexes generally falls under this category. These building These Buildings are not structurally designed and supervised by engineers during construction. This category also includes the buildings that have architectural drawings prepared by engineers.
4.	Engineered Reinforced Concrete Moment-Resisting-Frame Buildings	These buildings consist of a frame assembly of cast-in-situ concrete beams and columns. Floor and roof framings consist of cast-in-situ concrete slabs. Lateral forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. These are engineered buildings with structural design and construction supervision is made by engineers. Some of the newly constructed reinforced concrete buildings are of this type.
5	Others	Wooden buildings, Mixed buildings like Stone and Adobe, Stone and Brick in Mud, Brick in Mud and Brick in cement etc. are other building type in Kathmandu valley and other part of the country.

Determining Fragility of the Identified Building Typology

The probable damage to the building structures, that are available in Nepal and the region, at different intensities are derived based on “*The Development of Alternative Building Materials and Technologies for Nepal, Appendix-C: Vulnerability Assessment, UNDP/UNCHS1994*” and “*European Macro-seismic Scale (EMS98)*” http://www.gfz-potsdam.de/pb5/pb53/projekt/ems/core/emsa_cor.htm is given in Table A.I.2.

Table A.I.2(a) Building Fragility: Adobe+Field Stone Masonry Building

Shaking Intensity (MMI)		VI	VII	VIII	IX
PGA(%g)		5-10	10-20	20-35	>35
Damage Grade for different classes of buildings	Weak	DG4	DG5	DG5	DG5
	Average	DG3	DG4	DG5	DG5
	Good	DG2	DG3	DG4	DG4

Table A.I.2 (b) Building Fragility: Brick in Mud (General) Building

Shaking Intensity (MMI)		VI	VII	VIII	IX
PGA(%g)		5-10	10-20	20-35	>35
Damage Grade for different classes of buildings	Weak	DG3	DG4	DG5	DG5
	Average	DG2	DG3	DG4	DG5
	Good	DG1	DG2	DG3	DG4

Table A.I.2(c) Building Fragility: Brick in Mud (Well Built) + Brick in Cement (Ordinary)

Shaking Intensity (MMI)		VI	VII	VIII	IX
PGA(%g)		5-10	10-20	20-35	>35
Damage Grade for different classes of buildings	Weak	DG2	DG3	DG4	DG5
	Average	DG1	DG2	DG3	DG4
	Good	-	DG1	DG2	DG3

Table A.I. 2(d) Non-Engineered Reinforced Concrete Frame Buildings (≥ 4 storey)

Shaking Intensity (MMI)		VI	VII	VIII	IX
PGA(%g)		5-10	10-20	20-35	>35
Damage Grade for different classes of buildings	Weak	DG1	DG2	DG4	DG5
	Average	-	DG1	DG3	DG4
	Good	-	DG1	DG2	DG3

Table A.I. 2(e) Non-Engineered Reinforced Concrete Frame Buildings (≤ 3 storey) + Engineered Reinforced Concrete Buildings + Reinforced Masonry Buildings

Shaking Intensity (MMI)		VI	VII	VIII	IX
PGA(%g)		5-10	10-20	20-35	>35
Damage Grade for different classes of buildings	Weak	DG1	DG2	DG3	DG4
	Average	-	DG1	DG2	DG3
	Good	-	-	DG1	DG2

Identification of Vulnerability Factors

Different Vulnerability factors associated with the particular type of building are checked with a set of appropriate checklists from FEMA310, "Hand book for the Seismic Evaluation of Buildings" and "IS Guidelines for Seismic Evaluation and Strengthening of Existing Buildings". Separate check list is used for each of the common building types.

The basic vulnerability factors related to Building system, Lateral force resisting system, Connections and Diaphragms are evaluated based on visual inspection and review of drawings. A list of deficiencies identified by evaluation statements for which the building is found to be compliant and non-compliant shall be compiled up on completion of the checklist. If non-compliant, further investigation is required.

The evaluation statements are based on observed earthquake structural damage during actual earthquakes. Based on past performance of these types of buildings in earthquakes, the behavior of the structure must be examined and understood. However, the checklists will provide insight and information about the structure prior to quantitative evaluation. By quickly identifying the potential deficiencies in the structure, the design professional has a better idea of what to examine and analyze in quantitative evaluation.

Analysis performed as part of this evaluation is limited to quick checks. The evaluation involves a set of initial calculations and identifies areas of potential weaknesses in the building. The checks to be investigated are classified into two groups: *configuration related* and *strength related*. The preliminary evaluation also checks the compliance with the provisions of the seismic design and detailing codes. Quick checks shall be performed in accordance with

evaluation statement to verify compliance non-compliance situation of the statement. Seismic shear force for use in the quick checks shall be computed as per National building seismic code of the region.

The factors that pose less vulnerability to the building during earth quake shaking are listed below:

- Building should be regular in plan elevation and structural system
- Building should have sufficient redundancy
- Demand Capacity Ratio (DCR) of each structural elements as well as the whole structure should be less than 1
- The building shall contain one complete load path
- Building shall have no damage and deterioration of structural elements and materials it self
- There shall be no hammering between adjacent buildings
- There shall be no diaphragm discontinuity
- Structural elements and the building shall not be slender
- There shall be proper connection between each structural elements and between structural and non-structural elements
- Building should have sufficient ductility
- Building should not be situated on liquefaction susceptible soil, steep and rock fall areas, fault rupture surfaces and soil filled areas
- Non-structural elements should be restrained properly

Reverse of the criteria as mentioned above pose vulnerability to the building.

Reinterpretation of the Building Fragility Based on Observed Vulnerability Factors

After thorough analysis and interpretation of vulnerability factors, the building is categorized into weak, average or good type of that particular building typology. This facilitates in assessing the probable performance of the building at different intensities of earthquake in terms of damage grades viz. negligible, slight, moderate, heavy and very heavy damage or destruction.

Conclusions and Recommendation

The probable performance of the building at large expected earthquakes is identified based on the available information about the building, the architectural and structural information from field visit, and implementation of limited number of destructive and non-destructive field tests.

The evaluation helps in deciding whether the safety provided by the building is adequate. A decision is taken whether the building needs to be repaired, retro fitted or demolished based on the importance, target life, extent of deficiency of the building, the economic viability, the availability of the materials and technical resources and the expected life after retrofit. The stakeholders such as house owners, design engineers, occupants, municipality etc. are responsible in making the decision. The action can be either of the following.

- a) The safety of the building is adequate. The building needs some repair and regular maintenance, ensuring adequate performance during a future earthquake.
- b) The safety of the building is in adequate and hence, retrofit is necessary. The proposed retrofit scheme should be technically feasible and economically viable (Usually retrofitting is considered suitable if the cost of retrofitting is within 30% of the cost of new construction).

- c) The safety of the building is in adequate and the building is in imminent danger of collapse in the even to fan earthquake. The retrofit scheme is not economically viable or feasible. Unless the building has historical importance and is of traditional nature, it is recommended to demolish and reconstruct the building rather than retrofitting for better seismic performance.

The seismic life safety provided by a building is judged adequate if the requirements are met and many authorities accept this level of performance for their community. Any non-structural elements that pose life threatening risk to the occupants may either be removed or restrained.

A.I.4 QUANTITATIVE ASSESSMENT

Introduction

This Chapter describes the second phase study of seismic vulnerability assessment which is a quantitative approach and follows qualitative analysis. Before embarking on seismic retrofitting, seismic deficiencies shall have to be identified through a seismic evaluation process using a methodology described ANNEX I. The first phase assessment is general seismic vulnerability assessment method based on qualitative approach to identify the seismic deficiencies in the building. If the first phase study finds seismic deficiencies in the building and possible seismic performance is not up to the acceptable level/criteria, it recommends either second phase assessment or concludes the evaluation and state that potential deficiencies are identified. The second phase assessment involves a more detailed seismic evaluation with complete analysis of the building for seismic strengthening measures as modifications to correct/reduce seismic deficiencies identified during the evaluation procedure in first phase. Detail information about the building is required for this step of evaluation. Seismic retrofit becomes necessary if the building does not meet minimum requirements of the current Building Code, and may suffer severe damage or even collapse during a seismic event.

The most important issue when beginning to evaluate the seismic capabilities of an existing building is the availability and reliability of structural drawings. Detailed evaluation is impossible without framing and foundation plans, layout of preliminary lateral force elements, reinforcing for concrete structures, and connection detailing. This chapter assumes that sufficient information is available to perform a seismic evaluation that will identify all significant deficiencies.

Review Initial Considerations

The design professional shall review initial considerations which include structural characteristic of the building, seismic hazard including geological site hazards, results of prior seismic evaluations, areas of structural deficiencies, building use and occupancy requirements, historical status, economic considerations, societal issues, and local jurisdictional requirements. This step of evaluation should focus on the potential deficiencies as identified.

Seismic hazards other than ground shaking may also exist at the building site. The risk and possible extent of damage from such geologic site hazards should be considered before undertaking a seismic strengthening measure. In some cases it may be feasible to mitigate the site hazard or strengthen the building and still meet the performance level. In other cases, the risk due to site hazard may be so extreme and difficult to control that, seismic strengthening is neither cost-effective nor feasible.

Decide Performance Objective

The performance objective needs to be defined before analyzing the building for retrofit. The performance objective depends on various factors such as the use of building, cost and feasibility of any strengthening project, benefit to be obtained in terms of improved safety, reduction in property damage, interruption of use in the event of future earthquakes and the limiting damage states. The minimum objective is Life Safety i.e. any part of the building should not collapse threatening safety of occupants during a severe earthquake.

Design Basis Earthquake

Seismic hazard due to ground shaking shall be based on the location of the building with respect to causative faults, the regional and site-specific geologic characteristics, and a selected earthquake hazard level. Seismic hazard due to ground shaking shall be defined as acceleration response spectra acceleration time histories on either a probabilistic or deterministic basis. Seismic strengthening of buildings shall comply with the design criteria and procedures as specified in national building codes and standards of earthquake engineering.

A building must have been designed and constructed or evaluated in accordance with the current seismicity of the region.

Detailed Investigation

This includes the following steps:

- a) Obtaining the attributes of the structural materials used in the building.
- b) Determining the type and disposition of reinforcement in structural members.
- c) Locating deteriorated material and other defects, and identifying their causes.

For evaluation of member capacities, precise values of the material strength and the dimensions are desirable. Forth is, non-destructive and intrusive techniques are employed for determining the strength of the material.

Non-Destructive Tests

The following are the most common types of tests that are used for seismic evaluation of the building.

SOUNDING TEST

Description

Tapping on a wall with a dense object, such as a hammer, and listening to the vibrations emitted from the wall can be useful for identifying voids or *delaminations* in concrete and masonry walls. The sound produced from a solid wall will be different from that from a wall with voids or *delaminations* close to the surface. In concrete block masonry walls, sounding can be used to verify that the cells in the blocks have been grouted.

Equipment

The typical equipment required for sounding is a hammer. However, any hard, dense object can be used.

Conducting Test

In areas where the visual observations indicate that the wall may have *delaminations*, the wall can be sounded by tapping with a hammer. *Delaminations* and spalls will generally produce a hollow sound when compared with solid material. The wall should be tapped several times in the suspect area and away from the suspect area, and the sounds compared. It is important to test an area that is undamaged, and is of the same material and thickness to use as a baseline comparison. For a valid comparison, the force exerted by the tapping

should be similar for both the suspect and baseline areas. In reinforced masonry construction, sounding can be used to assess whether the cells in the wall have been grouted. Near the ends of a block, the unit is solid for the full thickness of the wall. For most of the length of the block, it is relatively thin at the faces. If the sound near the end of the block is substantially different than at the middle of the cell, the cell is probably not grouted.

Personal Qualification

Sounding of concrete and masonry walls should be performed by an engineer or trained technician. Engineers and technicians should have previous experience in identifying damage to concrete and masonry structures. Engineers and technicians should also be able to distinguish between sounds emitted from a hammer strike. Prior experience is necessary for proper interpretation of results.

Reporting Requirements

The personnel conducting the tests should provide sketches of the wall indicating the location of the tests and the findings. The sketch should include the following information:

- Mark the location of the test on either a floor plan or wall elevation.
- Report the results of the test, indicating the extent of *delamination*.
- Report the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

Limitations

The properties of the wall can influence the usefulness of sounding. The geometry of the wall and the thickness of the wall will affect the results. Sounding is best used away from the perimeter of the wall and on a wall of uniform thickness. The accuracy of information from sounding with a hammer also depends on the skill of the engineer or technician performing the test and on the depth of damage within the thickness of the wall. *Delaminations* up to the depth of the cover for the reinforcing bars (usually about 1 to 2 inches) can usually be detected. Detection of deeper spalls or *delamination* requires the use of other NDE techniques. Sounding cannot determine the depth of the spall or *delamination*.

Tapping on a loose section of material can cause the piece to become dislodged and fall. Avoid sounding overhead. A ladder, scaffold, or other lift device should be used to reach higher elevations of a wall.

REBOUND HAMMER TEST

Description

Rebound hammer provides a method for assessing the in-situ compressive strength of concrete. In this test, a calibrated hammer impact is applied to the surface of the concrete. The amount of rebound of the hammer is measured and correlated with the manufacturer's data to estimate the strength of the concrete. The method has also been used to evaluate the strength of masonry.

Equipment

A calibrated rebound hammer is a single piece of equipment that is hand operated

Execution

The person operating the equipment places the impact plunger of the hammer against the concrete and then presses the hammer until the hammer releases. The operator then records the value on the scale of the hammer. Typically three or more tests are conducted at

a location. If the values from the tests are consistent, record the average value. If the values vary significantly, additional readings should be taken until a consistent pattern of results is obtained.

Since the test is relatively rapid, a number of test locations can be chosen for each wall. The values from the tests are converted in to compressive strength using tables prepared by the manufacturer of the rebound hammer.



Fig A.I. 1.(a)Use of Rebound Hammer



Fig A.I. 1.(b)Rebound Hammer

Personal Qualification

A technician with minimal training can operate the rebound hammer. An engineer experienced with rebound hammer data should be available to supervise and verify that any anomalous values can be explained.

Reporting Requirements

The personnel conducting the tests should provide sketches of the wall, indicating the location of the tests and the findings. The sketch should include the following information:

- Mark the location of the test marked on either a floor plan or wall elevation.
- Record the number of tests conducted at a given location.
- Report either the average of actual readings or the average values converted in to compressive strength along with the method used to convert the values in to compressive strength.
- Report the type of rebound hammer used along with the date of last calibration.
- Record the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

Limitations

The rebound hammer does not give a precise value of compressive strength, but rather an estimate of strength that can be used for comparison. Frequent calibration of the unit is required (ACI,1994). Although manufacturers' tables can be used to estimate the concrete strength, better estimates can be obtained by removing core samples at selected locations where the rebound testing has been performed. The core samples are then subjected to compression tests. The rebound values from other areas can be compared with the rebound values that correspond to the measured core compressive strength.

The results of the rebound hammer tests are sensitive to the quality of the concrete on the outer several inches of the wall. More reproducible results can be obtained from formed

surfaces rather than from finished surfaces. Surface moisture and roughness can also affect the readings. The impact from the rebound hammer can produce a slight dimple in the surface of the wall. Do not take more than one reading at the same spot, since the first impact can affect the surface, and thus affect the results of a subsequent test.

When using the rebound hammer on masonry, the hammer should be placed at the center of the masonry unit. The values of the tests on masonry reflect the strength of the masonry unit and the mortar. This method is only useful in assessing the strength of the outer wythe of a multi-wythe wall.

REBAR DETECTION TEST

Description

Cover-meter is the general term for a rebar detector used to determine the location and size of reinforcing steel in a concrete or masonry wall. The basic principle of most rebar detectors is the interaction between the reinforcing bar and a low frequency magnetic field. If used properly, many types of rebar detectors can also identify the amount of cover for the bar and/or the size of the bar. Rebar detection is useful for verifying the construction of the wall, if drawings are available, and in preparing as built data if no previous construction information is available.

Equipment

Several types and brands of rebar detectors are commercially available. The two general classes are those based on the principle of magnetic reluctance and those based on the principle of eddy. The various models can have a variety of features including analogue or digital readout, audible signal, one handed operation, and readings for reinforcing bars and *pre-stressing* tendons. Some models can store the data on floppy disks to be imported into computer programs for plotting results.

Conducting Test

The unit is held away from metallic objects and calibrated to zero reading. After calibration, the unit is placed against the surface of the wall. The orientation of the probe should be in the direction of the rebar that is being detected. The probe is slid slowly along the wall, perpendicular to the orientation of the probe, until an audible or visual spike in the read out is encountered.

The probe is passed back and forth over the region of the spike to find the location of the maximum reading, which should correspond to the location of the rebar. This location is then marked on the wall. The procedure is repeated for the perpendicular direction of reinforcing.

If size of the bar is known, the cover-meter readout can be used to determine the depth of the reinforcing bar. If the depth of the bar is known, the readout can be used to determine the size of the bar. If neither quantity is known, most rebar detectors can be used to determine both the size and the depth using a pacer technique.

The process involves recording the peak reading at a bar and then introducing a pacer of known thickness between the probe and the surface of the wall. A second reading is then taken. The two readings are compared to estimate the bar size and depth. Intrusive testing can be used to help interpret the data from the detector readings. Selective removal of portions of the wall can be performed to expose the reinforcing bars. The rebar detector can be used adjacent to the area of removal to verify the accuracy of the readings.



Fig A.I.2.(a) Use of Rebar Detector for Verification of Reinforcement Details



Fig A.I.2.(b) Ferro-scan Detector

Personnel Qualifications

The personnel operating the equipment should be trained and experienced with the use of the particular model of cover-meter being used and should understand the limitations of the unit.

Reporting Requirements

The personnel conducting the tests should provide a sketch of the wall indicating the location of the testing and the findings. The sketch should include the following information:

- Mark the locations of the test on either a floor plan or wall elevation.
- Report the results of the test, including bar size and spacing and whether the size was verified.
- List the type of rebar detector used.
- Report the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

Limitations

Pulse-velocity measurements require access to both sides of the wall. The wall surfaces need to be relatively smooth. Rough areas can be ground smooth to improve the acoustic coupling. *Couplant* must be used to fill the air space between the transducer and the surface of the wall. If air voids exist between the transducer and the surface, the travel time of the pulse will increase, causing incorrect readings.

Some *couplant* materials can stain the wall surface. Non-staining gels are available, but should be checked in an inconspicuous area to verify that it will not disturb the appearance.

Embedded reinforcing bars, oriented in the direction of travel of the pulse, can affect the results, since the ultrasonic pulses travel through steel at a faster rate that will significantly affect the results. The moisture content of the concrete also has a slight effect (up to about 2 percent) on the pulse velocity.

Pulse-velocity measurements can detect the presence of voids or discontinuities within a wall; however, these measurements cannot determine the depth of the voids.

IN-SITU TESTING IN-PLACE SHEAR

Description

The shear strength of unreinforced masonry construction depends largely on the strength of the mortar used in the wall. An in-place shear test is the preferred method for determining the strength of existing mortar. The results of these tests are used to determine the shear strength of the wall.

Equipment

- Chisels and grinders are needed to remove the bricks and mortar adjacent to the test area.
- A hydraulic ram, calibrated and capable of displaying the applied load.
- A dial gauge, calibrated to 0.001 inch.

Execution

Prepare the test location by removing the brick, including the mortar, on one side of the brick to be tested. The head joint on the opposite side of the brick to be tested is also removed. Care must be exercised so that the mortar joint above or below the brick to be tested is not damaged.

The hydraulic ram is inserted in the space where the brick was removed. A steel loading block is placed between the ram and the brick to be tested so that the ram will distribute its load over the end face of the brick. The dial gauge can also be inserted in the space.

The brick is then loaded with the ram until the first indication of cracking or movement of the brick. The ram force and associated deflection on the dial gauge are recorded to develop a forced reflection plot on which the first cracking or movement should be indicated. A dial gauge can be used to calculate a rough estimate of shear stiffness.

Inspect the collar joint and estimate the percentage of the collar joint that was effective in resisting the force from the ram. The brick that was removed should then be replaced and the joints re-pointed.



Fig A. I.3. Test Setup for In-Situ Shear Test

Personnel Qualifications

The technician conducting this test should have previous experience with the technique and should be familiar with the operation of the equipment. Having a second technician at the site is useful for recording the data and watching for the first indication of cracking or movement. The structural engineer or designer should choose test locations that provide are presentative sampling of conditions.

Reporting Results

The personnel conducting the tests should provide a written report of the findings to the evaluating engineer. The results for the in-places hear tests should contain, at a minimum, the following information for each test location:

- Describe test location or give the identification number provided by the engineer.
- Specify the length and width of the brick that was tested, and its cross-sectional area.
- Give the maximum mortar strength value measured during the test, in terms of force and stress.
- Estimate the effective area of the bond between the brick and the grout at the collar joint.
- Record the deflection of the brick at the point of peak applied force.
- Record the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

Limitations

This test procedure is only capable of measuring the shear strength of the mortar in the outer *wythe* of a multi-*wythe* wall. The engineer should verify that the exterior *wythe* being tested is a part of the structural wall, by checking for the presence of header courses. This test should not be conducted on veneer *wythes*.

Test values from exterior *wythes* may produce lower values when compared with tests conducted on inner *wythes*. The difference can be due to weathering of the mortar on the exterior *wythes*. The exterior brick may also have a reduced depth of mortar for aesthetic purposes.

The test results can only be qualitatively adjusted to account for the presence of mortar in the collar joints. If mortar is present in the collar joint, the engineer or technician

conducting the test is not able to discern how much of that mortar actually resisted the force from the ram.

The personnel conducting the tests must carefully watch the brick during the test to accurately determine the ram force at which first cracking or movement occurs. First cracking or movement indicates the maximum force, and thus the maximum shear strength. If this peak is missed, the values obtained will be based only on the sliding friction contribution of the mortar, which will be less than the bond strength contribution.

Seismic Analysis and Design

The detail seismic evaluation refers to the structural analysis of the building. Structural analysis is a part of the detailed evaluation of an existing building. The method of analysis is to be finalized at this stage based on building data. The evaluation procedure includes an analysis using the methods of Linear/ Non Linear Static procedure or Linear/Non Linear Dynamic procedure or special procedure for unreinforced masonry bearing wall buildings with flexible diaphragm being evaluated to the life safety Performance Level. The steps include developing a computational model of the building, applying the external forces, calculating the internal forces in the members of the building, calculating the deformation so the members and building, and finally interpreting the results. The structural analysis is performed using a suitable computer analysis program. The relevant seismic code is referred for lateral load calculation. The model is analyzed for the individual load cases after the computational model is developed and the loads areas signed.

Intervention Options for Better Seismic Performance

General

Retrofit strategy refers to any option of increasing the strength, stiffness and ductility of the members or of the whole building. The possible intervention options need to be selected based on the building typology and the expected performance of the building after retrofitting. Following considerations ought to be additionally made while selecting probable intervention options:

- a) Requirements to comply to the Building Code for design, materials and construction (Compatibility of the solution with the functional requirements of the structure)
- b) Possible cost implication
- c) In direct cost of retrofitting such as relocation cost
- d) Availability of construction technique (materials, equipment and workmanship) in construction industry
- e) Enhancement of the safety of the building after intervention of the selected option
- f) Aesthetic view of the building
- g) Once these considerations are made, different options of modifying the building to reduce the risk of damage should be studied. The corrective measures include stiffening or strengthening the structure, adding local elements to eliminate irregularities or tie the structure together, reducing the demand on the structure through the use of seismic isolation or energy dissipation devices, and reducing the height or mass of the structure.

Retrofitting Methods

General Improvement

Plan Shape

If the building is found irregular and unsymmetrical in plan shape, the plan shape of the building can be improved from earthquake point of view by separating wings and dividing into more regular, uniform and symmetrical shapes.

Elevation Improvement

Buildings may have unbalanced stiffness in plan and elevation. In many buildings, the rooms are added horizontally when and where required without seismic consideration. It makes one part of same house one storey while the rest is two-storied. Separating the two parts or demolition/ addition part of the building eliminating upper storey set back from base can solve this problem.

Load Path

Buildings may suffer from the problem of discontinuous load path. It needs more intelligent solutions, re-planning of space to create new and more direct load paths. A complete load path is a basic requirement for all buildings. If there is discontinuity in load path, the building is unable to resist seismic forces regardless of the strength of the existing elements.

Inserting New Walls

To improve effectiveness of existing walls to mitigate *torsional* problem due to non-symmetry in walls in plan, and to improve shear resistance of the buildings, or to provide return walls to existing walls, new walls are added at appropriate locations. It may require closing of some existing openings. Exact location of these walls is determined during detailed study.

Modification of Roofs or Floors

Heavy and brittle roof tiles that can easily dislodge should be replaced with light and corrugated iron and asbestos sheeting. Undesired heavy floor mass, that only in duce increased seismic force, need to be removed. False ceiling and heavy ceiling plasters that create a condition of potential hazard of falling during a shaking should either be anchored properly or replaced with light material. Roof truss should be braced by welding or clamping suitable diagonal bracing members in vertical as well as in horizontal planes. Anchors of roof trusses to supporting walls should be improved and the roof thrust on walls should be eliminated.

Strengthening the Arches

Jack arch roofs are common in old masonry buildings for spanning larger distance between walls.

To prevent spreading of arches, it is proposed to install tie rods across the arch spring level so slightly above it by drilling holes on both sides and grouting steel rods in them. However, where it is not possible a lintel consisting of steel channels or I-section, could be inserted just above the arch to take the load and relieve the arch.

REDUCTION IN BUILDING MASS

A reduction in mass of the building results in reduction in lateral forces. This can be achieved by removing unaccountable upper stories, replacing heavy cladding, floor and ceiling, removing heavy storage or change in occupancy use.

Seismic Retrofitting Strategies of Masonry Buildings

Major Weaknesses Revealed During Earthquakes in Similar Building

Typology

The following are the major types of problems and basic damage patterns observed during earthquakes in this type of buildings:

- *Torsional* effect to the building due to Irregular shape of the building
- Non-integrity of wall, floor and roof structures and their components
- Out-of-plane collapse due to lack of anchoring elements on upper parts of the wall of the flexible roof buildings
- Separate orthogonal walls at junctions due to developing cracks
- Collapse of gable wall since it behaves as a free cantilever
- Reduce wall stiffness or storey stiffness due to large opening
- Out-of plane failure of walls due to lack of cross walls
- Collapse of the building due to rapid cracking and disintegrating of various parts due to brittle nature

Common Retrofitting Methods for the Masonry Buildings

The concept of retrofitting masonry buildings start from enhancing integrity to the structure by providing proper connections 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 ability to resist. Typical important aspects are the connection a) between components of floors and roof; b) between roof or floor and walls; c) between intersecting walls; and d) walls and foundation.

Commonly used improvements methods include eliminating features that are: a) source of weakness or that produce concentrations of stresses in some members, b) abrupt change of stiffness from floor to floor, c) concentration of large masses, and d) large openings in walls without proper peripheral enforcement. Increasing the lateral strength in one or both directions, by reinforcing or by increasing wall plan areas or the number of walls may be required in some cases.

Avoiding the possibility of brittle mode of failure by providing proper reinforcement and connection of load resisting members is the overall objective in seismic strengthening.

Selected retrofitting options for the masonry buildings, considering the basic principles of retrofitting mentioned above, are described below. These methods are being implemented worldwide and are considered economically and technically viable though other expensive methods are also available.

Jacketing

This method is adopted on buildings constructed with a material that is heavy in weight, weak in strength, and brittle. It helps to basket the wall, hence improve its shear strength and ductility. This method also improves integrity and deform ability. Main improvements in different structural elements of the building by this method are as follows:

Walls:

To improve strength, deformability and to reduce risk of disintegration, *delamination* of walls resulting in total collapse of the building, thin reinforcement concrete jacketing of all the walls is done. In this alternative, two steel meshes should be placed on either two sides or one side of the wall and both the meshes should be connected by some steel bar connectors passing through the wall. The thickness of the added concrete should be about 40 to

50 mm thick. The concrete used ought to be a micro-concrete i.e. concrete with small aggregates. Selection of one-side jacketing or two-side jacketing depends on the analysis result.

Floors:

If the floor is flexible, bracing of the floor elements with steel or timber sections and tying up the floor elements with walls should be done to improve stiffness of the floor system and to obtain integrity between walls and floor.

Roof:

If the roof is flexible, similar to floor, bracing of the roof elements with steel or timber sections and tying up the roof elements with walls should be done to improve stiffness of the roof system and to obtain integrity between walls and roof.

False Ceiling:

Ceiling may need replacement with a light ceiling system and better anchorage system.

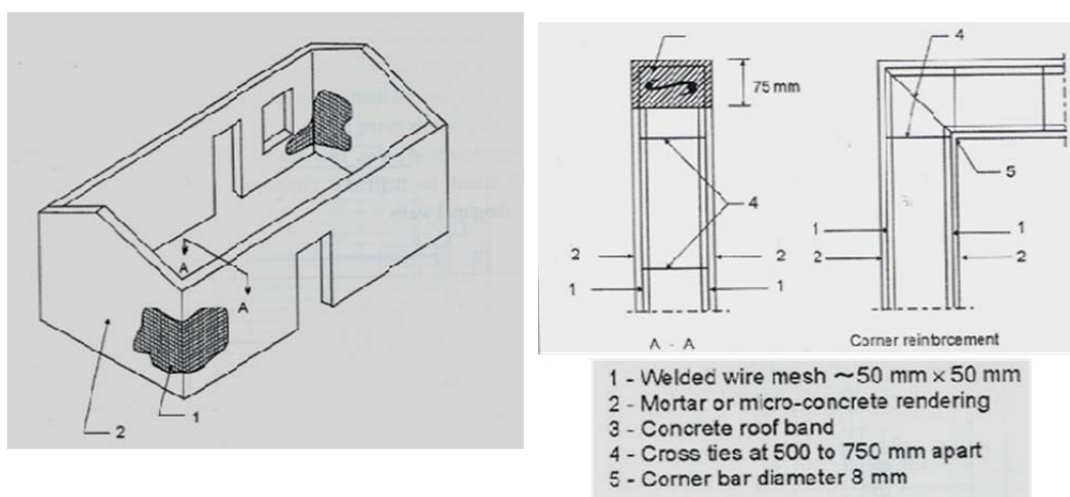


Fig A.I.4.(a) General Scheme of Jacketing



Fig A.I.4(b) Erection of Reinforcement for wall



Fig A. I. 4(c) Wall Jacketing in process

PROCESS OF WALL JACKETING

Splint and Bandage

The Splint and Bandage system is considered as an economic version of jacketing where reinforcing bars are provided at most critical locations (Figure A.I.5), where ever stress concentrations can develop. Splints are vertical elements provided at corners, wall junctions and jambs of openings in the external faces of the building. The objective is to provide integrity in vertical direction.

The bandages are horizontal elements running around all the walls and building to integrate various walls together there by preventing potential out of plane collapse of walls. In addition, openings are also surrounded by splints and bandages to prevent initiation and widening of cracks from their corners. Splints are provided in the external face only. The bandages could be provided on both the faces of the walls just at the lintel, eaves and sill level. This method is inferior to jacketing but better than bolting as discussed below in terms of safety enhancement. In splint and bandage system, the strengthening and stiffening of the floor and roof is made in the same way as discussed above.



Before



During



After

Fig A.I. 5 Process of Retro fitting by Splint and Bandage Method Bolting/ Pre-stressing

Confinement with Reinforced Concrete Elements

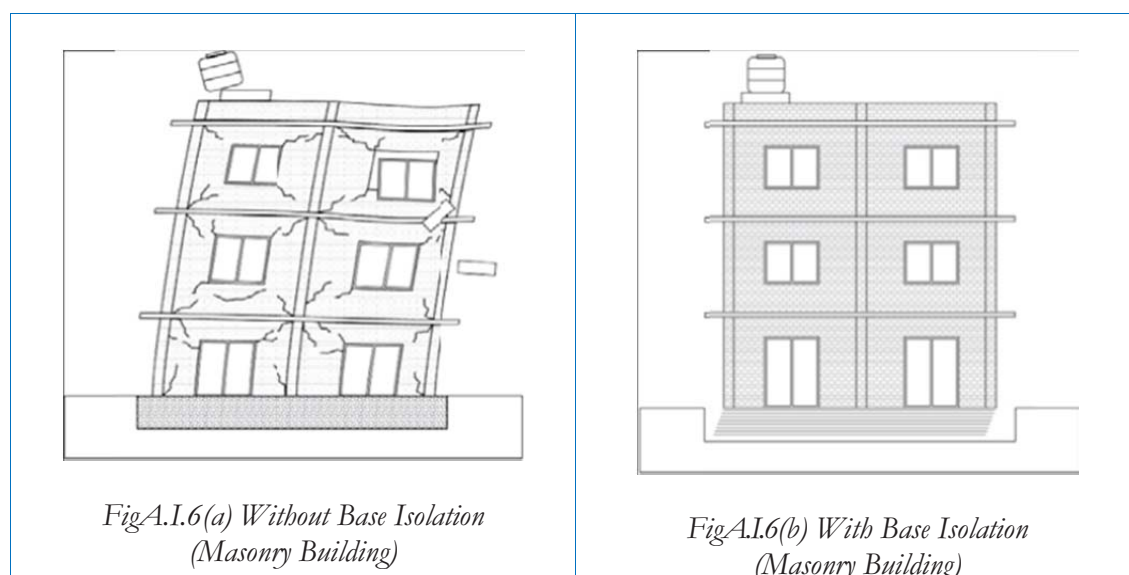
Confinement with reinforced concrete elements (beam and columns) make the existing masonry act as “confined masonry” in the sense that reinforced concrete elements are reinserted surrounding the wall panel or middle of the long wall, allowing the entire wall, or its portion, to act as a truss element, where the struts are inclined strips of unreinforced masonry. In this way, brittle and non-ductile wall becomes more ductile and its load carrying capacity increased several times with added confinement of there in forced

concrete elements. It is more suitable for buildings of one to three storey heights with monolithic reinforced concrete slab and horizontal bands over the load bearing walls at the lintel level. However, implementation of this method of retrofitting is more complex and needs special improvements for foundation also.

Base Isolation

What effectively is done in this scheme is that the super structure is strengthened nominally and is isolated from ground motion by introducing a flexible layer between the structure and the ground. The various types of base isolation devices are i) Laminated rubber bearing ii) Laminate drubber bearing with lead core iii) Sliding bearing and iv) Friction pendulum devices. Base isolation modifies the response characteristics so that the maximum earthquake forces on the building are much lower. The seismic isolation eliminates or significantly reduces not only the structural damage but also non-structural damage and enhances the safety of the building content and architectural components

(Figure A.I.6 below) This technique is usually employed for buildings with historic importance and critical facilities and is quite expensive as compared to other methods.



Use of FRP (Fiber Reinforced Polymer)

Seismic resistance of masonry buildings improve significantly by using glass or carbon FRP strips on walls. Strengthening with FRP is a new approach. Both flexural and shear capacity of masonry walls can be enhanced by applying thin films of glass or carbon FRP to the exterior surface of the wall.

Main advantages of Fiber Reinforced Polymer (FRP) retrofitting are:

- Increases out-of-plane flexural strength
- Increases in-plane shear strength
- Increases stiffness at service loads
- Results in monolithic action of all units
- Strengthening of entire wall can be accomplished by treating only a fraction of wall surface area
- Adds very little weight to the wall
- Minimum changes in the member size after repair

- Limited access requirements
- Lower installation cost
- Improved corrosion resistant
- On-site flexibility of use

Even though the materials used in FRP are relatively expensive as compared to the traditional strengthening materials such as steel and concrete, the labor, equipment and construction costs are often lower. It is a promising technique since its application is more easy and rapid with minimum disturbance to the occupants. Application of FRP, with care, provides significant increase in lateral strength but it does not provide as much ductility as the RC wall would provide, because of the brittleness of the material. For effective use, a firm anchorage should be provided between FRP and the wall panel. The possible schemes of layout of FRP wraps are shown in the figure below.

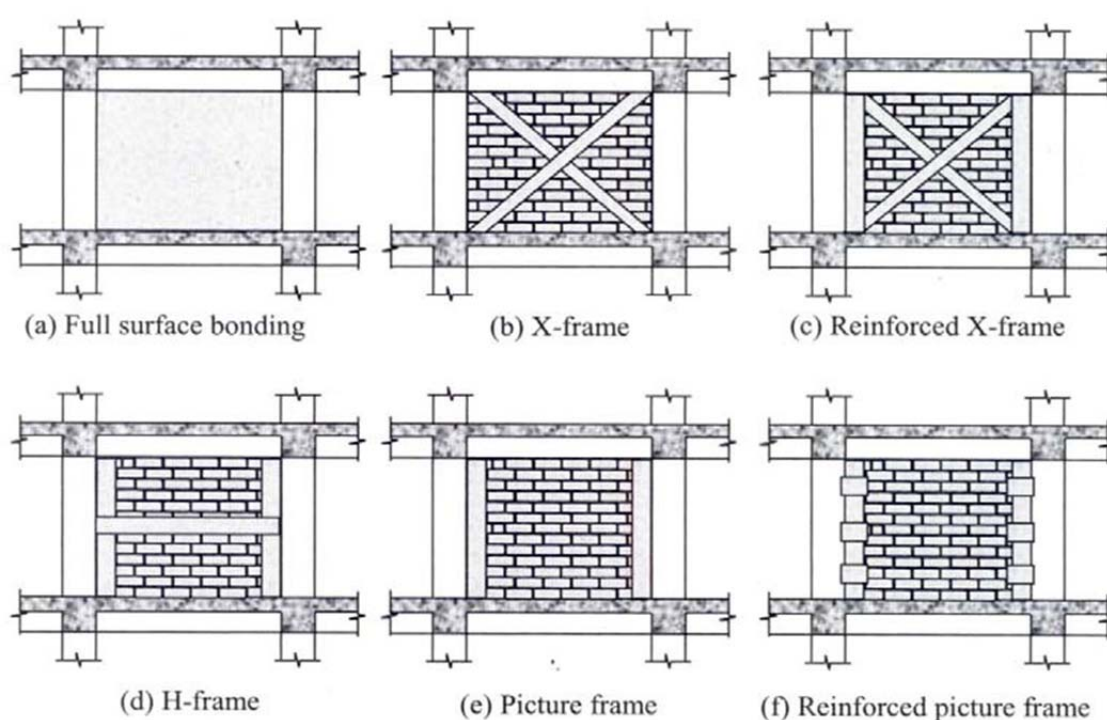


Fig A.I.7 Configurations of FRP Laminates of Masonry Walls

Comparison of Common Methods of Retrofitting for Masonry Building

Different options of possible retrofitting technique need to be compared for the building to be assessed considering its structural details and possible failure patterns. In general, the parameters that are considered are the effectiveness of retrofit system, its cost implication, importance of the building, economic and technical feasibility of the project.

Table A.I.3: Comparison of Different Retrofitting Options

	Retrofitting Options					
	Jacketing	Splint and Bandage	Bolting/ Pre stressing	Confinement with reinforced concrete elements	Base Isolation	Strengthening with FRPs
Maximum Nos. of Storey	Suitable up to 4 storey	Suitable up to 3 storey, preferable for 2 storey	Suitable up to 2 storey	Suitable up to 3 storey	Suitable for low to medium rise buildings with time period up to 0.5 sec	Suitable for low rise buildings up to 2 Stories
Architectural Changes	Extensive	Moderate	Less	Significant	Insignificant	Less
Intervention time	Long	Moderate	Short	Long	Long	Less
Cost	High	Moderate	Low	High	Extensive	High
Safety achieved up to MMI IX	Life safety -Immediate Occupancy	Life safety	Brittle collapse prevention	Life safety	Immediate Occupancy	Life safety

The study should consider the structural system of the building, its major structural problems, importance of the building and different available option so retrofitting to select appropriate retrofitting option. The above table compares different retrofitting options in various aspects. The suitable retrofitting option is adopted for a particular building.

Seismic Retrofitting Strategies of Reinforced Concrete Buildings

Major Weaknesses Revealed During Earthquake in Similar Building

Typology

The following are the major types of problems observed during earthquake sin this type of buildings:

- Absence of ties in beam column joints
- In adequate confinement near beam column joint
- In adequate lap length and anchorage and splice at in appropriate position
- Low concrete strength
- Improperly anchored ties (90°hooks)
- Inadequate lateral stiffness
- Inadequate lateral strength
- Irregularities in plan and elevation
- Irregular distribution of loads and structural elements
- Other most common structural efficiencies such as soft storey effect, short column effect, strong beam-weak column connections etc.

Common Retrofitting Methods for the Reinforced Concrete Buildings

Various methodologies are available for analysis and retro fitting of frame structures. Earthquake resistance in RC frame buildings can be enhanced either by:

a) *Increasing seismic capacity of the building*

This is a conventional approach to seismic retro fitting which increase the lateral force resistance of the building structure by increasing stiffness, strength and ductility and reducing irregularities. This can be done by two ways

1) *Strengthening of original structural members*

These include strengthening of

- Columns (reinforced concrete jacketing, steel profile jacketing, steel encasement, fiber wrap overlays)
- Beams (reinforced concrete jacketing, steel plate reinforcement, fiber wrap overlays), Beam Column joint (reinforced concrete jacketing, steel plate reinforcement, fiber wrap overlays)
- Shear wall (increase of wall thickness)
- Slab (increase of slab thickness, improving slab to wall connection)
- *In filled* partition wall (reinforce *in filled* wall and anchor them in to the surrounding concrete frame members).

2) *Introduction of New structural elements*

The lateral force capacity of an existing structure may be increased by adding new structural elements to resist part or all of these is mic forces of the structure, leaving the old structure to resist only that part of the seismic action for which it is judged to be reliable. Newly added structural elements maybe

- Shear wall sin a frame or skeleton structure
- *In filled* walls (reinforced concrete or masonry located in the plane of existing column and beams)
- Wing walls (adding wall segments or wings on each side of an existing column)
- Additional frame sin a frame or skeleton structure
- Trusses and diagonal bracing (steel or reinforced concrete) in a frame or skeleton structure

Establishing sound bond between the old and new concrete is of great importance. It can be provided by chipping away the concrete cover of the original member and roughening its surface, by preparing the surfaces with glues (for instances, with epoxy prior to concreting), by additional welding of bend rein for cement bars or by formation of reinforced concrete or steel dowels.

Perfect confinement by close, adequate and appropriately shaped stirrups and ties contributes to the improvement of the ductility of the strengthening members. Detailed consideration of the possibility of signify cant redistribution of the internal forces in the structures due to member stiffness changes is very important.

b) *Reducing seismic response of the building*

Increasing damping in the building by means of energy dissipation devices, reducing mass, or isolating the building from the ground enhance the seismic structural response. A more recent approach includes the use of base isolation and supplement all damping devices in the building. These emerging technologies can be used to retrofit existing RC frame structures; however their high cost and the sophisticated expertise required to design and implement such projects represent impediments for broader application at recent time.

Seismic strengthening measures identified for one RC frame building may not be relevant for another. It is therefore very important to develop retrofit solutions for each building on a case-by-case basis. Most of these retrofit techniques have evolved in viable upgrades. However, issues of costs, invasiveness, and practical implementation still remain the most challenging aspects of these solutions. In the past decade, an increased interest in the use of advanced non-metallic materials or Fiber Reinforced Polymers, FRP has been observed.

The following retrofit strategies for RC buildings are widely used after recent earthquakes in several places:

Reinforced Concrete Jacketing

This method involves addition of a layer of concrete, longitudinal bars and closely spaced ties on existing structural elements. The jacket increases both the flexural strength and shear strength of the column and beam. It helps to encase the member, hence improve its shear strength and ductility. This method also improves integrity and deformability. Main improvements in different structural elements of the building by this method are as follows:

Columns:

The jacketing not only increases the flexural strength and shear strength of the column but also increases its ductility. The thickness of the jacket also gives additional stiffness to the concrete column. Since the thickness of the jacket is small, casting self-compacting concrete or the use of shotcrete are preferred to conventional concrete. During retrofitting, it is preferred to relieve the columns of the existing gravity loads as much as possible, by propping the supported beams.

Beams:

Beams are retrofitted to increase their positive flexural strength, shear strength and the deformation capacity near the beam-column joints. The lack of adequate bottom bars and their anchorage at the joints needs to be addressed. Usually the negative flexural capacity is not enhanced since the retrofitting should not make the beams stronger than the supporting columns. The strengthening involves the placement of longitudinal bars and closely spaced stirrups.

Addition of Reinforced Concrete Shear Walls

Adding shear walls is one of the most popular and economical methods to achieve seismic protection. Their purpose is to give additional strength and stiffness to the building and could be added to existing and new buildings. They are positioned after careful planning and judgment by the structural engineer as to how they would affect the seismic forces in a particular building. However, it is desired to ensure an effective connection between the new and existing structure.



Fig A. I. 8(a) Jacketing of RC Column and Column Jacketing



Fig A. I. 8(b) Addition of Shear Wall

Steel Bracing

In this method diagonal braces are provided in the bays of the building. Diagonals stretch across the bay to form triangulated vertical frame and as triangles are able to handle stresses better than a rectangular frame the structure is also supposed to perform better. Braces can be configured as diagonals, X or even V shaped. Braces are of two types, concentric and eccentric. Concentric braces connect at the intersection of beams and columns whereas eccentric braces connect to the beam at some distance away from the beam-column intersection. Eccentric braces have the advantage that in case of buckling the buckled brace does not damage beam-column joint.



Fig. A.I.9 Retrofitting by Diagonal Steel Bracing

Base Isolation

In this method super structure is isolated from ground motion during earthquake shaking by using flexible layer between the structure and the ground as discussed. The only difference is that these isolators are introduced individually beneath column support while as in masonry building a flexible layer is introduced throughout the wall stretch at base.

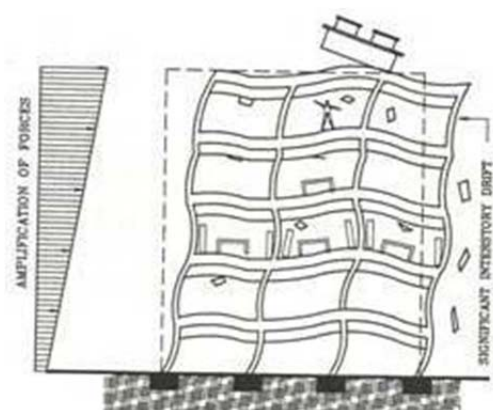


Fig A.I.10 (a) Without Base Isolation
(RC Frame Building)

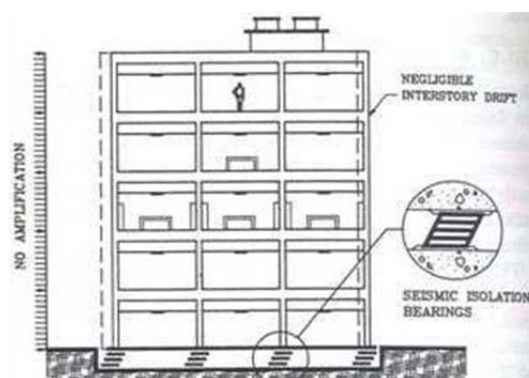


Fig A.I.10 (b) With base isolation (RC Frame)

Use of FRP (Fiber Reinforced Polymer)

Seismic resistance of frame buildings can be improved significantly by using Fiber Reinforced Polymer overlays on RC elements of the building. Strengthening with FRP is a new approach. FRP is light weight, high tensile strength material and has a major advantage of fast implement action. This method could be effectively used to increase strength and stiffness of RC frames. The effectiveness is strongly dependent on the extent of anchorage between the FRP strip sand the frame.

Comparison of Common Methods of Retrofitting for Reinforced Concrete Building

Different options of possible retrofitting technique are compared for the assessment of the building considering its structural details and possible failure patterns. In general, the parameters that are considered are the effectiveness of retro of it system, its cost implication, importance of the building, economic and technical feasibility of the project etc.

Comparative Chart of Different Retrofitting Options for RC Frame Buildings

Retro fitting Options for RC frame building					
	Installing new RC wall	Jacketing	Bracing	Strengthening existing frame and masonry in fill with CFRPs	Base Isolation
Architectural Changes	Moderate-significant	Moderate	Extensive	Less	Insignificant
Intervention time	Long	Long	Moderate	Less	Long
Cost	High	High	Moderate	High	Extensive
Increase of ductility	Significant	moderate	moderate	small	Not required as earthquake load is cut at foundation level
Safety achieved up to MMIIX	Minimum Life Safety	Minimum Life Safety	Life Safety	Life Safety	Immediate Occupancy

Foundation Intervention

An engineer should opt for aseismic strengthening measure with minimum work on the found action. If foundation intervention is desired, the retrofit strategy becomes in variably expensive. In some cases, retrofitting may not be economically and practically viable at all. Foundation treatment usually requires excavation under difficult circumstances. In addition, there are difficulties in pinning or attaching the existing footings to the new elements. And construction is very difficult and expensive. This great cost will occur due to inaccessibility of the existing footings and the great uncertainty regarding the characteristics of the soil and existing footings. Numerous seismic rehabilitation projects have been cancelled because of excessive cost.

Before undertaking any structural retrofitting measures and foundation work, an engineer should critically analyze the cost benefit and feasibility of the project. There are many issues to be considered, these include:

- Foundation failures may result in severe economic loss resulting in damage to structural and non-structural elements. But, failure of foundation may have smaller effect on the Life-safety and collapse prevention limit as large foundation movements are needed to cause structural collapse.
- Seismic strengthening or upgrade of the foundation may result in transmission of larger seismic force sin to the structure. Hence, foundation strengthening may increase the cost of structural upgrade since more structural work is required in response of foundation work. In some cases, foundation upgrade may adversely affect the life safety and collapse prevention limit states. The engineer must balance a range of economic, social and technical concerns, when evaluating these issues.
- However in general the foundation work will reduce the probability of serious economic damage during an earthquake.

Cost Estimate

After thorough analysis and selection of suitable retrofitting option, if necessary, preliminary cost is estimated. This should include the cost for materials, labor, taxes, contractor's profit and indirect cost such as relocation, etc. The tentative cost is calculated per unit area based on the current practice. Further, considering the uncertainty associated with the work, some additional 20% of the total cost needs to be added as unforeseen cost.

As the retrofitting work need strained mason, wages should be taken from prevailing market rates for special finishing and quality and specially trained manpower. The rates not covered by Government norms should be based on best engineering judgment and past experience.

The decision to repair and strengthening a structure depends not only on technical considerations but also on a benefit cost analysis of the different possible alternatives. It is suggested that the cost of retrofitting of a structure should remain below 25% of the replacement as major justification of retrofitting (*Nateghi and Shabbazian, 1992*).

Comparison of Possible Performance of the Building after Retrofitting

The probable performance of the building understudy is compared in terms of possible damage grade before and after retrofitting. This helps in identifying whether the acceptable level of seismic response in terms of Life safety as minimum requirement is achieved after implementation of retrofitting technique suggested for the building. This is very important as the client knows the level of safety to be attained and the benefit of retrofit scheme.

Conclusions and Recommendations

Conclusions

The principal objectives of this study are: (i) to identify weak links in the building based on observed behavior in similar buildings in past earthquakes and (ii) to develop possible intervention options to improve their seismic resistance with associated costs and level of incremental seismic safety. The conclusions arrived from the detail analysis are described as:

- Various retrofit options are compared and studied. Out of which, the most suitable retrofit technique is proposed for the particular building type keeping all factors, as mentioned in previous chapters, in consideration. The retrofit option should improve the building response with Life Safety as minimum requirement.
- The cost of retrofitting may differ to some extent if the actual structural strength and details are found different, than those as summed during retrofit design, once walls and roof are opened during field implementation.

Recommendations

To reduce the disastrous effects of earthquakes on buildings, function and life, the following recommendations are made:

- A time-bound program should be implemented to retrofit the building within corporation of seismic resistant measures as selected.
- Retrofitting is an advanced process and requires a higher level of expertise than that required for design and construction of new buildings. The process requires lots of destructive interventions such as hammering, drilling in walls, and removal of some parts of building. Such activities may cause additional damage if proper attention is not given during implementation. Hence, use of experienced and skilled labor with proper supervision is emphasized.
- Retrofit design may need revision once structural, architectural and ornamental elements of the building are removed for implementation and details differ from those as summed at design stage. Hence, it is suggested to clarify from the contractor's side, before signing of the contract, about such issues and seek flexibility in design details that are required to be implemented at site.
- During retrofitting process, the elements such as floor cornices, *chajjas*, cladding, false ceiling, that add beauty to the building, need to be removed. Prior to implementation of retrofitting plans, designer's advice may be sought for retaining good aesthetic view of the building after retrofitting.
- Supervision during the retrofitting works is very essential as it is a delicate work. Hence, it is extremely important to have proper supervision at the site during retrofitting.

Due consideration is to be given for uniform distribution of furniture and fixtures, equipment and other non-structural elements so that the load distribution is even. The non-structural elements (partitions, furniture, equipment, etc.) should be fixed properly for restricting their movement to prevent overturning, sliding and impacting during an earthquake. Masonry walls are recommended to be braced with reinforced concrete mesh or any other means to prevent non-structural damage during earthquakes of large intensity.

ANNEX A.I.I: BUILDING TYPOLOGY IDENTIFICATION

The major building types in Nepal are given in the following table. From the visual observation and study of the available drawings of the building, the buildings are categorized in the following typologies.

No.	Building Types in Kathmandu Valley	Description
1	Adobe, stone in mud, brick-in-mud (Low Strength Masonry).	<p>Adobe Buildings: These are buildings constructed in sun-dried bricks (earthen) with mud mortar for the construction of structural walls. The walls are usually more than 350 mm.</p> <p>Stone in Mud: These are stone-masonry buildings constructed using dressed or undressed stones with mud mortar. These types of buildings have generally flexible floors and roof.</p> <p>Brick in Mud: These are the brick masonry buildings with fired bricks in mud mortar</p>
2	Brick in Cement, Stone in Cement	These are the brick masonry buildings with fired bricks in cement or lime mortar and stone-masonry buildings using dressed or undressed stones with cement mortar.
3	Reinforced Concrete Ordinary-Moment-Resisting-Frame Buildings	These are the buildings with reinforced concrete frames and unreinforced brick masonry infill in cement mortar. The thickness of infill walls is 230mm (9") or even 115mm (4 1/2") and column size is predominantly 9"x 9". The prevalent practice of most urban areas of Nepal for the construction of residential and commercial complexes is generally of this type.
4	Reinforced Concrete Intermediate-Moment-Resisting-Frame Buildings	These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs. Lateral forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. These are engineered buildings designed without earthquake load or with old codes or designed for small earthquake forces. Some of the newly constructed reinforced concrete buildings are likely to be of this type.
5	Reinforced concrete special-moment-resistant-frames (SMRF)	These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs. Lateral forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. These buildings have joint reinforcing, closely spaced ties, and special detailing to provide ductile performance. Despite the fact that this system should be adopted ideally for all new RC frame buildings in Nepal, it is now only used as an exception.

ANNEX A.I.II VULNERABILITY FACTORS IDENTIFICATION CHECKLIST
Structural Assessment Check list for Type1 Buildings(Adobe, Stone in Mud ,Brick in Mud)
Building System

CNCN/A SHAPE: The building shall be symmetrical in plan and regular in elevation.

CNCN/A PROPORTIONINPLAN: The breadth to length ratio of the building shall be with in 1: 3. The breadth to length ratio of any room or area enclosed by load bearing walls inside the building shall be also with in 1 : 3. The building height shall be not more than three times the width of the building.

CNCN/A STOREY HEIGHT: The floor to floor height of the building shall be between 23m.

CNCN/A NUMBER OF STORIES : The building shall be up to two stories only.

CNCN/A FOUNDATION: The foundation width and depth shall be at least 75cm. Masonry unit shall be of flat-bedded stone so regular-sized well-burnt bricks. Mortar joints shall not exceed 20mm in any case. There shall be no mud-packing in the core of the foundation.

CNCN/A SLOPING GROUND : The slope of the ground where the building lies shall not be more than 20° (1:3, vertical : horizontal)

CNCN/A PLUMBLINE: Walls of the foundation and super structure shall be true to plumb line and the width of the wall shall be uniform.

CNCN/A WALL CORE: There shall be no mortar packing in the core of the wall.

CNCN/A THROUGH-STONES: In case of stone building, the walls shall have plenty of through-stones extending the whole width of the walls. The maximum spacing of such through-stones shall be within1.2m horizontally and 0.6m vertically.

CNCN/A WALL THICKNESS: The minimum wall thickness in mm for different storey heights shall not be less than

Masonry Type	No of Storey	
	One	Two
Stone	340-450	450
Brick	230	350

CNCN/A UNSUPPORTED WALL LENGTH: The maximum length of unsupported wall shall not be more than 12 times its thickness. If the length of unsupported wall is more than 12 times its thickness, buttressing shall be provided.

CNCN/A HEIGHT OF WALLS: The thickness to height ratio of a wall shall not be more than 1 : 8 for stone building and 1 : 12 for brick building.

CNCN/A OPENINGS IN WALL: The maximum combined width of the openings on a wall between two consecutive cross – walls shall not be more than 35% of the total wall length for one-storey building and not more than 25% of the total wall length in two-storey building.

CNCN/A POSITION OF OPENINGS: Openings shall not be located at corners or junctions of a wall .Openings shall not be placed closer to an internal corner of a wall than half the opening height or 1 . 5 times the wall thickness, whichever is greater. The width of pier between two openings shall not be less than half of the opening height or 1.5 times the wall thickness, whichever is greater. The vertical distance between two openings shall not be less than 0 . 6m or half the width of the smaller opening, whichever is greater.

CNCN/A LOAD PATH: The structure shall contain at least one rational and complete load path for seismic forces from any horizontal direction so that they can transfer inertial forces in the building to the foundation.

CNCN/A VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation.

CNCN/A MASS: There shall be no change in effective mass more than 100% from one storey to the next.

CNCN/ATORSION: The estimated distance between the storey center of mass and the storey center of stiffness shall be less than 30% of the building dimension at right angles to the direction of loading considered.

CNCN/A MASONRY UNITS: There shall be no visible deterioration of masonry units.

CNCN/A WALL CRACKS: There shall be no existing diagonal cracks in wall elements greater than 1/16" or out - of - plane off sets in the bed joint greater than 1/16".

CNCN/A MASONRY LAY-UP: Filled collar joints of *multistory* masonry walls shall have negligible voids.

CNCN/A VERTICAL REINFORCEMENT: There shall be vertical reinforcement at all corners and T-junctions of masonry walls and it shall be started from foundation and continuous to roof.

CNCN/A HORIZONTAL BANDS: There shall be steel or wooden bands located at the plinth, sill and lintel levels of the building in each floor.

CNCN/A CORNER STITCH: There shall be reinforced concrete or wooden elements connecting two orthogonal walls at a vertical distance of at least 0.5m to 0.7m.

CNCN/A GABLE BAND: If the roof is slopped roof, gable band shall be provided to the building.

Lateral Force Resisting System

CNCN/A REDUNDANCY: The number of lines of walls in each principal direction shall be greater than or equal to 2.

Diaphragms

CNCN/A DIAGONAL BRACING: All flexible structural elements of diaphragms such as joists and rafters shall be diagonally braced and each crossing of a joist/ rafter and a brace shall be properly fixed.

CNCN/A LATERAL RESTRAINERS: Each joists and rafters shall be restrained by timber keys in both sides of wall.

Geologic Site






CNCN/A NK AREA HISTORY: Evidence of history of landslides, mudslides, soil settlement, sinkholes, construction on fill, or buried on or at sites in the area are not anticipated.

CNCN/A NK LIQUEFACTION: Liquefaction susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils.

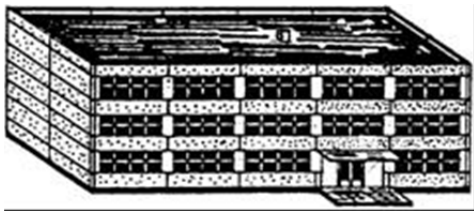




CNCN/A NK SLOPE FAILURE: The building site shall be sufficiently remote from potential earthquake induced slope failures or rock falls to be unaffected by such failures or shall be capable of accommodating any predicted movements without failure.

ANNEX A.I.III : CLASSIFICATION OF DAMAGE GRADES

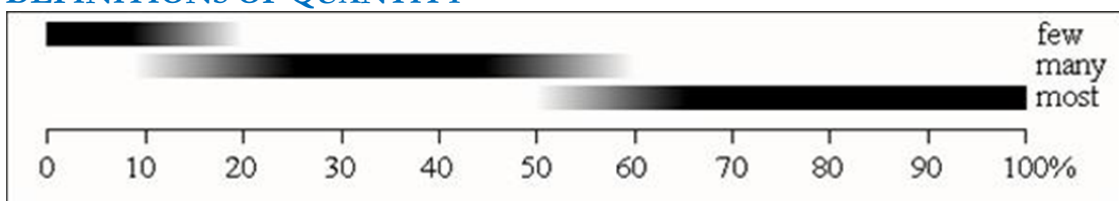
Classification of damage to masonry buildings

	<p>Grade 1: Negligible to slight damage (no structural damage, slight non-structural damage)</p> <p>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.</p>
	<p>Grade 2: Moderate damage (slight structural damage, moderate non-structural damage)</p> <p>Cracks in many walls. Fall of fairly large pieces of plaster. Partial collapse of chimneys.</p>
	<p>Grade 3: Substantial to heavy damage (moderate structural damage, heavy non-structural damage)</p> <p>Large and extensive cracks in most walls. Roof tiles detach. Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).</p>
	<p>Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage)</p> <p>Serious failure of walls; partial structural failure of roofs and floors.</p>
	<p>Grade 5: Destruction (very heavy structural damage)</p> <p>Total or near total collapse.</p>

Classification of damage to buildings of reinforced concrete

	<p>Grade 1: Negligible to slight damage (no structural damage, slight non-structural damage)</p> <p>Fine cracks in plaster over frame members or in walls at the base. Fine cracks in partitions and infills.</p>
	<p>Grade 2: Moderate damage (slight structural damage, moderate non-structural damage)</p> <p>Cracks in columns and beams of frames and in structural walls. Cracks in partition and infill walls; fall of brittle cladding and plaster. Falling mortar from the joints of wall panels.</p>
	<p>Grade 3: Substantial to heavy damage (moderate structural damage, heavy non-structural damage)</p> <p>Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. Spalling of concrete cover, buckling of reinforced rods. Large cracks in partition and infill walls, failure of individual infill panels.</p>
	<p>Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage)</p> <p>Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns. Collapse of a few columns or of a single upper floor.</p>
	<p>Grade 5: Destruction (very heavy structural damage)</p> <p>Collapse of ground floor or parts (e. g. wings) of buildings.</p>

DEFINITIONS OF QUANTITY



ANNEX A.I.IV: MODIFIED MERCALLI INTENSITY SCALE (MMI Scale)

I. Very Weak Intensity

- Can only be noticed or felt by people who are in the right situation and circumstance
- Furniture's or things which are not correctly positioned may move or be slightly displaced
- Slight shaking or vibrations will form on water or liquid surfaces in containers

II. Slightly Weak Intensity

- Can be noticed or felt by people who are resting inside homes
- Things that are hanged on walls would slightly sway, shake or vibrate
- The shaking or vibrations on water or liquid surfaces in containers would be highly noticeable

III. Weak Intensity

- Can be noticed and felt by more people inside homes or buildings especially those situated at high levels. Some may even feel dizzy. The quake at this stage can be described as though a small truck had passed nearby.
- Things that are hanged on walls would sway, shake or vibrate a little more strongly.
- The shaking or vibrations on water or liquid surfaces in containers would be more vigorous and stronger

IV. Slightly Strong Intensity

- Can be noticed and felt by most people inside homes and even those outside. Those who are lightly asleep may be awakened. The quake at this stage can be described as though a heavy truck had passed nearby.
- Things that are hanged on walls would sway, shake or vibrate strongly. Plates and glasses would also vibrate and shake, as well as doors and windows. Floors and walls of wooden houses or structures would slightly squeak. Stationary vehicles would slightly shake.
- The shaking or vibrations on water or liquid surfaces in containers would be very strong. It is possible to hear a slight reverberating sound from the environment.

V. Strong Intensity

- Can be felt and noticed by almost all people whether they are inside or outside structures. Many will be awakened from sleep and be surprised. Some may even rush out of their homes or buildings in fear. The vibrations and shaking that can be felt inside or outside structures will be very strong.
- Things that are hanged on walls would sway, shake or vibrate much more strongly and intensely. Plates and glasses would also vibrate and shake much

strongly and some may even break. Small or lightly weighted objects and furniture would rock and fall off. Stationary vehicles would shake more vigorously.

- The shaking or vibrations on water or liquid surfaces in containers would be very strong which will cause the liquid to spill over. Plant or tree stem, branches and leaves would shake or vibrate slightly.

VI. Very Strong Intensity

- Many will be afraid of the very strong shaking and vibrations that they will feel, causing them to lose their sense of balance, and most people to run out of homes or building structures. Those who are in moving vehicles will feel as though they are having a flat tire.
- Heavy objects or furniture would be displaced from original positions. Small hanging bells would shake and ring. Outer surfaces of concrete walls may crack. Old or fragile houses, buildings or structures would be slightly damaged.
- Weak to strong landslides may occur. The shaking and vibrations of plant or tree stem, branches and leaves would be strong and highly noticeable.

VII. Damaging Intensity

- Almost all people will be afraid of the very strong shaking and vibrations that they will feel. Those who are situated at high levels of buildings will find it very hard to keep standing.
- Heavy objects or furniture would fall and topple over. Large hanging bells will sound vigorously. Old or fragile houses, buildings or structures would most definitely be destroyed, while strong or new structures would be damaged. Dikes, dams, fishponds, concrete roads and walls may crack and be damaged.
- Liquefaction (formation of quicksand), lateral spreading (spreading of soil surface creating deep cracks on land) and landslides will occur. Trees and plants will vigorously shake and vibrate.

VIII. Highly Damaging Intensity

- Will cause confusion and chaos among the people. It makes standing upright difficult even outside homes/structures.
- Many big buildings will be extremely damaged. Landslides or lateral spreading will cause many bridges to fall and dikes to be highly damaged. It will also cause train rail tracks to bend or be displaced. Tombs will be damaged or be out of place. Posts, towers and monuments may bend or completely be destroyed. Water and canal/drainage pipes may be damaged, bend, or break.
- Liquefaction and lateral spreading causes structures to sink, bend or be completely destroyed, especially those situated on hills and mountains. For places near or situated at the earthquake epicenter, large stone boulders may be thrown out of position. Cracking, splitting, fault rupture of land may be seen.

Tsunami-like waves will be formed from water surfaces whether from rivers, ponds or dams/dikes. Trees and plant life will very vigorously move and sway in all directions.

IX. Destructive Intensity

- People would be forcibly thrown/fall down. Chaos, fear and confusion will be extreme.
- Most building structures would be destroyed and intensely damaged. Bridges and high structures would fall and be destroyed. Posts, towers and monuments may bend or completely be destroyed. Water and canal/drainage pipes may be damaged, bend, or break.
- Landslides, liquefaction, lateral spreading with sand boil (rise of underground mixture of sand and mud) will occur in many places, causing the land deformity. Plant and trees would be damaged or uprooted due to the vigorous shaking and swaying. Large stone boulders may be thrown out of position and be forcibly darted to all directions. Very-very strong tsunami-like waves will be formed from water surfaces whether from rivers, ponds or dams/dikes.

X. Extremely Destructive Intensity

- Overall extreme destruction and damage of all man-made structures
- Widespread landslides, liquefaction, intense lateral spreading and breaking of land surfaces will occur. Very strong and intense tsunami-like waves formed will be destructive. There will be tremendous change in the flow of water on rivers, springs, and other water-forms. All plant life will be destroyed and uprooted.

ANNEX A: EARTHQUAKE VULNERABILITY ASSESSMENT OF BUILDING (EXAMPLES)

A.1 EXAMPLE 1

INTRODUCTION

This section includes the qualitative earthquake vulnerability analysis public Building. The analysis is based on available information and the information gathered during the site survey of the existing structure.

A.1.1 DESCRIPTION OF THE EXISTING STRUCTURAL SYSTEM

The building is two storied with a storey height of 3 meters. The site is mildly sloped with gravel mixed soil type. The construction in the building has been done in four phases. The building is in two levels, the upper block is at 300mm above the lower block. The ground floor of the upper block was constructed 13 years ago, and then the ground floor of the lower block was constructed about 10 years ago. The first floor of the upper block was then constructed 8 years ago, after which, the first floor of the lower block was constructed 2 years back. The structural system is the load bearing wall system. The ground floor wall is 350mm thick and of brick in mud mortar masonry. The first floor wall is 230mm thick and of brick in cement mortar. The slab of first floor is of reinforced brick concrete 100mm thick with 10mm bar @ 250mm center to center. The slab of the roof is of reinforced cement concrete 100mm thick with 10mm bars @ 200mm center to center. No tie beam, ceiling band, sill band and lintel band has been provided at ground floor. However, ceiling band has been provided at the first floor. A 40mm thick damp proof concrete has been provided at the plinth level. The internal walls and ceilings are plastered and painted. The front exterior wall is finished with cement plaster and painted. The rest of the exterior walls are fair-faced.

The reinforcement is in exposed condition in many places due to insufficient cover. Deflections are noticed in the beams in some places. Cracks are observed between slab and wall connection. Dampness is observed in many places due to seepage. The beam reinforcement is exposed at the end as a result of which it is susceptible to rusting.



Exploration of foundation



Void between masonry units



Rebar exposed in RBC slab



Rebar exposed in RCC slab



Rebar exposed at beam ends



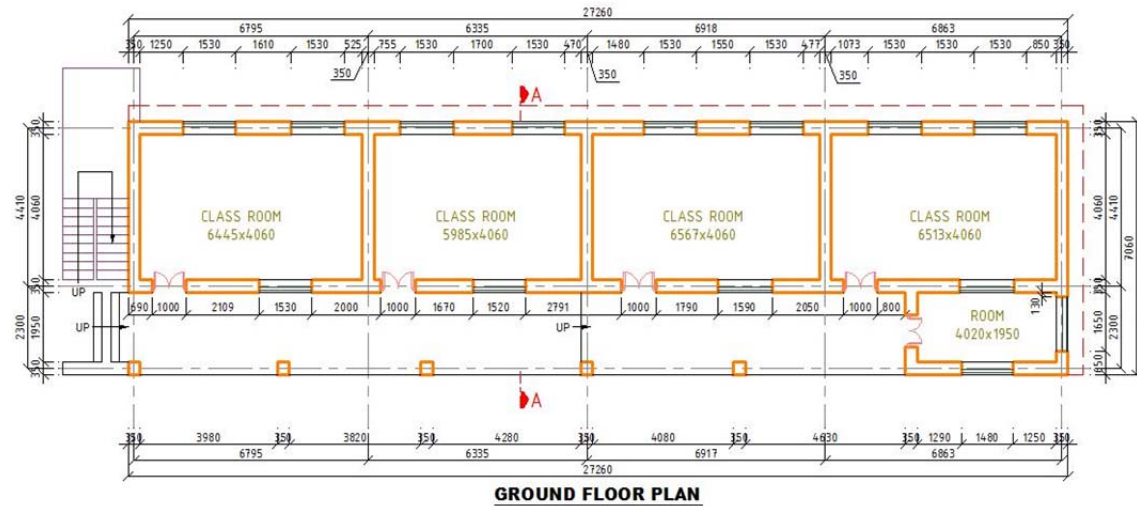
Exposed reinforcement



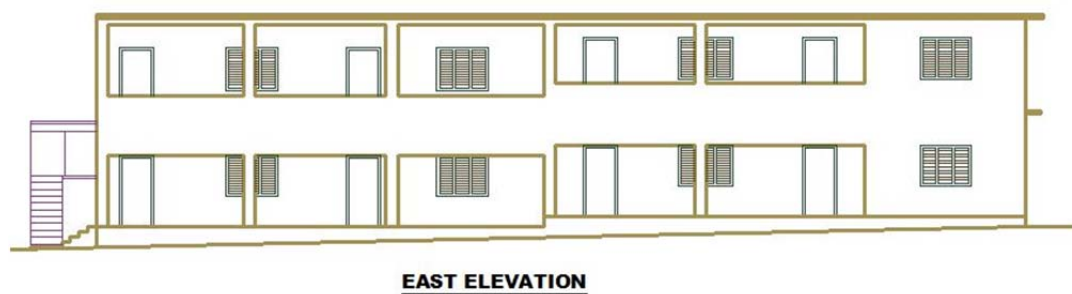
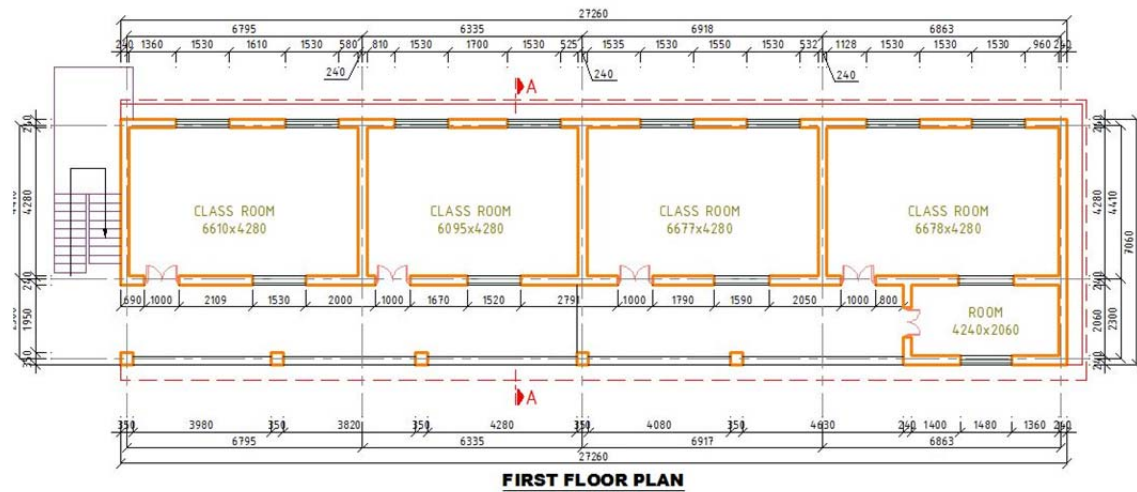
Cracks at slab & wall connection

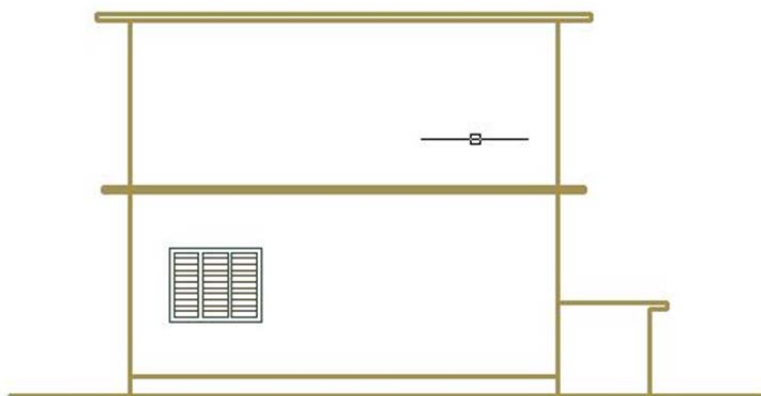
Figure A- 1: Different damage patterns in the buildings

A.1.2 BUILDING DRAWINGS



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





NORTH ELEVATION



SECTION AT -A-A



SOUTH ELEVATION

A.1.3 PROBABLE DAMAGE GRADE OF THE EXISTING BUILDING TYPOLOGY AT DIFFERENT INTENSITIES

From the visual observation and study of the available drawings of the building, the building under study is identified as Type 1 or Adobe Brick in mud masonry as the walls are composed of brick in mud mortar masonry in the ground floor and brick in cement masonry in the first floor. See Annex D for details of the identification of different building typology.

Table A- 1: Probable damage grades of type-1 building typology at different intensities

MMI		VI	VII	VIII	IX	X
Damage Grades for Different Classes of Buildings	Weak	DG4	DG5	DG5	DG5	DG5
	Average	DG3	DG4	DG5	DG5	DG5
	Good	DG2	DG3	DG4	DG4	DG5

(Note: The description of different damage degrees is provided in Annex E and the details of the MMI scale is given in Annex F)

We can see from table that even the good buildings in the type-1 category suffer a damage grade of 5 at an intensity of X. This building can be categorized as an average building in the type-1 typology as we can see cracks in the structural system of the building.

A.1.4 IDENTIFICATION OF VULNERABILITY FACTORS

Different Vulnerability factors associated with the particular type of buildings are checked with a set of appropriate checklist from FEMA 310, “Handbook for the Seismic Evaluation of Buildings”. The basic vulnerability factors related to Building system, lateral force resisting system, connections, and diaphragms are evaluated based on visual inspection and review of drawings. The checklist used for checking different vulnerability factors of the assessed building is given in Table A- 2 of this report. However, the critical vulnerability factors of the building required quantitative checking. Some of the important calculation sheets are attached within the following section of this annex.

The influence of different vulnerability factors to the building on the basis of visual inspection for the different buildings

Table A- 2: Influence of Different Vulnerability Factors

Vulnerability Factors		Increasing Vulnerability of the Building by different vulnerability factors				
		High	Medium	Low	N/A	N/K
General	Load Path			√		
	Weak Story			√		
	Soft Story			√		
	Geometry			√		
	Vertical Discontinuity			√		
	Mass			√		
	Torsion			√		
	Deterioration of Material			√		
	Masonry Units			√		

Vulnerability Factors		Increasing Vulnerability of the Building by different vulnerability factors				
		High	Medium	Low	N/A	N/K
	Masonry Wall Cracks	√				
Lateral Force Resisting System	Redundancy		√			
	Shear Stress	√				
Connection	Wall Anchorage		√			
	Transfer of Shear Walls		√			
Diaphragm	Plan Irregularities		√			
	Diaphragm Reinforcement at Openings		√			

A.1.5 CONCLUSION

The existing structure is likely to undergo heavy structural damage during earthquakes of high intensity. The presence of cracks between the roof and walls indicate that the diaphragm is not attached to the load bearing wall system properly. The difference in levels of the two blocks might attract stress during earthquakes due to the short column effect. Also, the fact that the building was constructed at four different stages and four different time frames makes the building even more vulnerable to earthquakes. The good part here, however, is that cracks are not observed at the connection of the new and old constructions. The beam sections seem inadequate for the beam span as the deflection of the beam is visible. Masonry in mud mortar are generally weak in shear hence, proper strengthening should be provided to withstand the shear forces that are bound to be imposed on the structure during earthquakes.

A.1.6 RECOMMENDATIONS

The following recommendations can be made to improve the structural performance of the building:

- Provide continuous bands for improving integrity of the structure as a whole
- Provide corner stitches at corners and wall junctions.
- The roofing system should be properly braced to the walls to withstand lateral loads.
- The long walls should be strengthened by providing buttress to break span.
- The existing cracks should be grouted.
- The beams should be checked for deflection and shear stress and jacketed if necessary.

A.2. EXAMPLE 2

A.2.1 CAPACITY ASSESSMENT OF ADOBE BUILDING

A.2.1.1. 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).

A.2.1.1.1. 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.

a) *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 A- 2). 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 (K_2) 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 $0.5\Delta u$ [Griffith et al. 2003]. For maximum displacements less

than $0.5\Delta_u$ the stiffness can be represented as function of Δ_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].

b) 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:

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

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 T_1 is the period of vibration of the structure.

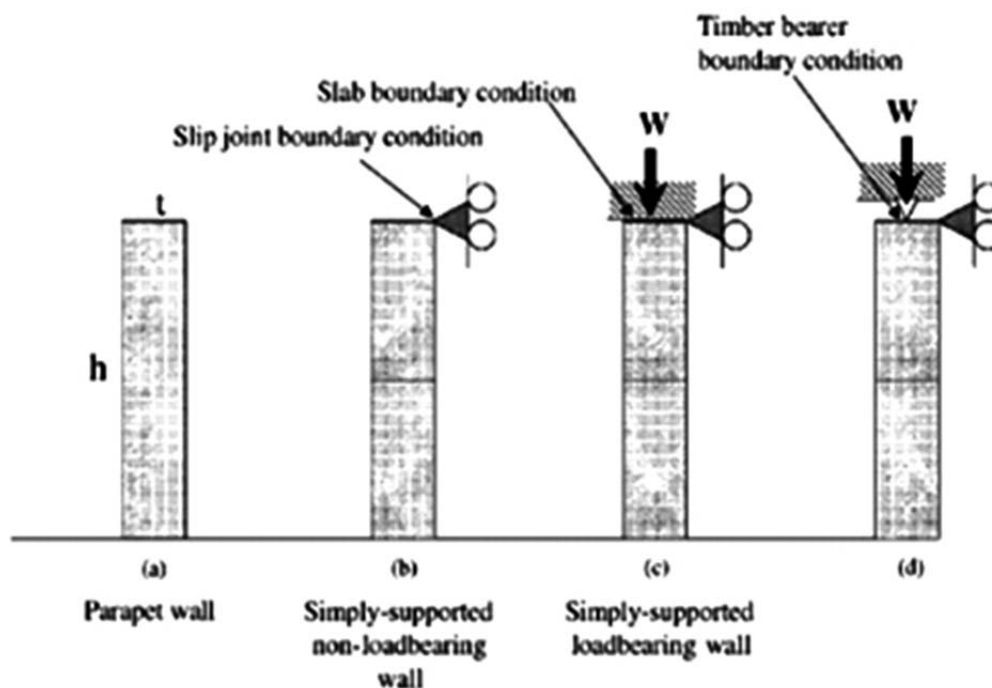


Figure A- 2: Unreinforced masonry wall support configurations (Doherty et al. [2002])

c) 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.

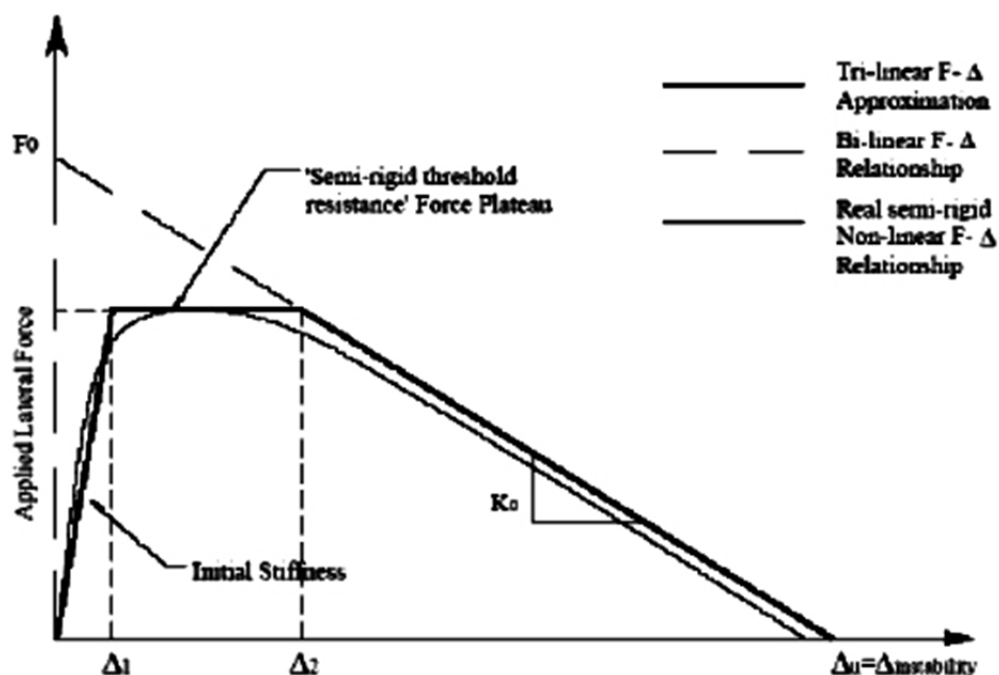


Figure A- 3: Tri-linear idealization of the static force- displacement relationship (Griffith et al. 2003)

Δ_1 is related to the end of the initial stiffness and Δ_2 is related to the secant stiffness. Δ_u 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 Δ_u mean that the wall will collapse. $F_0 = \lambda W$ is the force at incipient rocking and is also called the “Rigid Threshold Resistance”, λ is the collapse multiplier factor (See Section A.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 A- 4). 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 (Δ_u) 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 A- 3).

Table A- 3: Displacement ratios for tri-linear model

State of degradation at cracked joint	—	—
New	6	28
Moderate	13	40
Severe	20	50

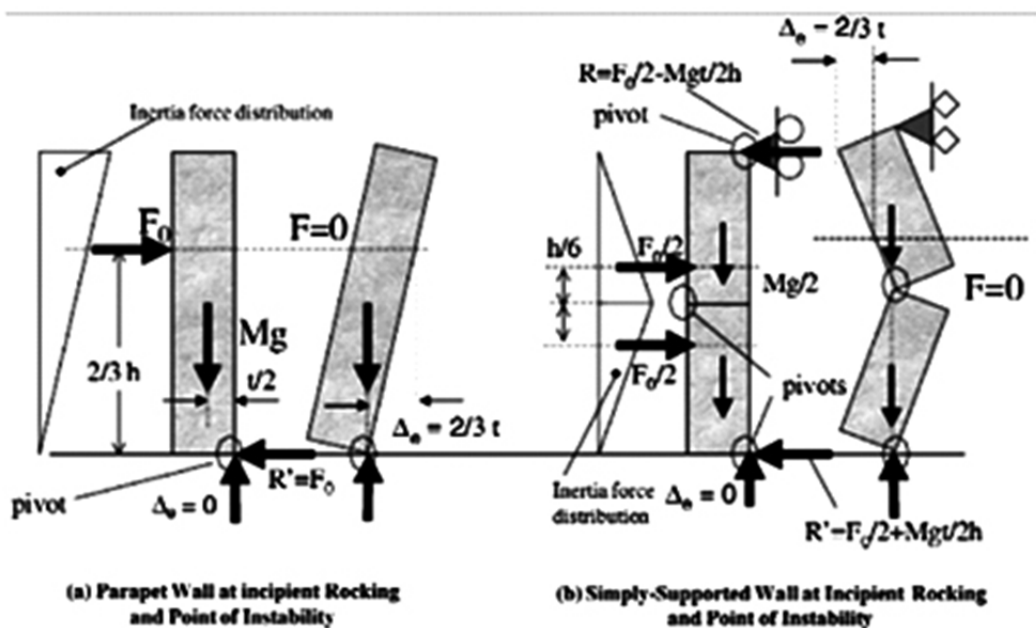


Figure A- 4: inertia forces and reactions on rigid URM walls (Doherty et.al 2002)

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 $\Delta_u \approx 0.8 t$ 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 K_1 should be considered when we are dealing with maximum displacements less than $0.5\Delta_u$ [Griffith et al. 2003]. The following limit states described in Table A- 4 have been assumed for the out-of-plane behavior. 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 A- 4: Limit states for adobe walls subjected to out of plane forces

Limit State	Top displacement	Crack width at the base	ζ (%)
LS1	$\approx 17\text{mm}$	$\approx 3\text{mm}$	5
LS2	$\approx 40\text{mm}$	$\approx 7\text{mm}$	5
LS3	$\Delta_1 \approx 0.12\Delta_u + \sigma_{SD} \approx 45\text{mm}, \quad \sigma_{SD}=0.01$	$\approx 8\text{mm}$	5
Ultimate LS	$\Delta = \phi\Delta_u, \Delta_u = 0.8 t, \quad \phi \approx 0.8 \sim 1.0$	$\approx 50\text{mm}$	5

d) 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 $F_o = \lambda W$, where λ is the collapse multiplier factor (see section A.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 $F_o = \lambda W$ is the force necessary to trigger overturning.

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

$$K_{\Delta_2} = \frac{F}{\Delta_2} \dots\dots\dots 0-3$$

The lateral static strength F and the ultimate displacement Δ_u 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 T_2 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 ($\Delta u > 0.7 \Delta_{max}$), 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 .

e) Period of vibration

The period of vibration for the ultimate limit state can be obtained from .

$$T = 2\pi \left(\frac{M}{K} \right)^{\frac{1}{2}}$$

So, using Eq.(4-2) and (4-3) it is obtained Eq.(0-4):

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

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

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

Where $\Delta_{i, su} = \phi \Delta_u$, with ϕ is a factor that can be assumed from 0.8 to 1 just to reduce the ultimate limit state, and $\rho_2 = \Delta_2 / \Delta_u$ (Table A- 4).

For intermediate limit states (where displacement limits are less or equal to Δ_i) a value of 0.12 for Δ_i / Δ_u is assumed with a standard deviation of 0.01 (Table A- 3). 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.

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

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

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

Replacing Eq.(0-7) into

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

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

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

f) 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, $\lambda = F / W$) 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

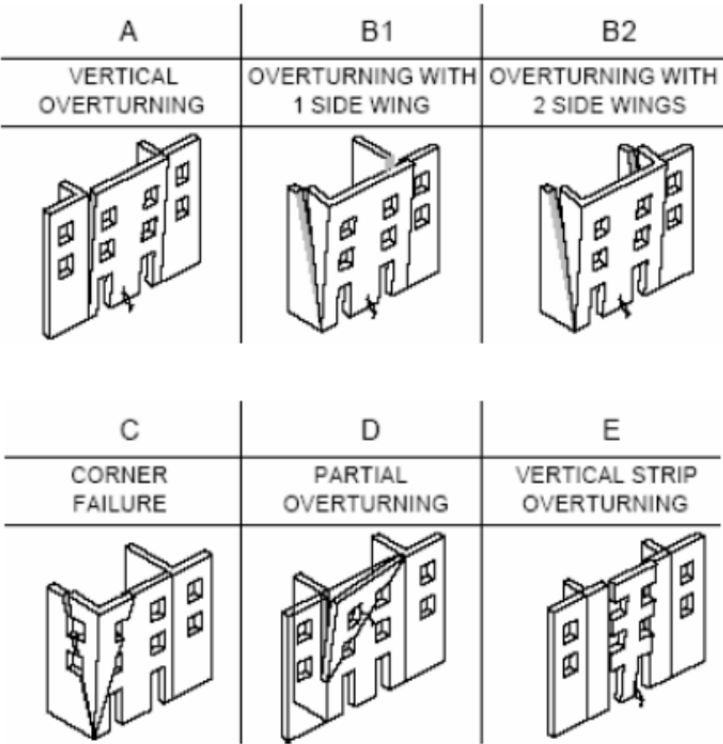


Figure A- 5: Collapse mechanism (D' analysis and Speranza 2003)

Rest repo [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

$$\lambda = \frac{\left(T^2L/2 + \beta\Omega_{pef} \frac{h_s}{3} \mu sb \frac{(r+1)}{2} + \frac{K_rLT}{2}\right)}{h_s \left(\frac{LT}{2} + K_{\#}L\right)} \dots\dots\dots 0-10$$

$$K_r = \frac{Q_r}{\gamma_m h_s} \dots\dots\dots 0-11$$

Where T and L are the thickness and length of the front walls, β is the number of edge and internal perpendicular walls, Ω is a partial efficiency factor to account for the limited effect of the friction, h_s 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 γ_m is the unit weight of the masonry (18 N/m^3).

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

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

Even Eq.(0-1) results in a collapse multiplier that represents a collapse mechanism between A and B_2 . The friction coefficient for adobe blocks varies from $\tan 30^\circ \approx 0.6$ [Corazao and Blondet 1974] to $\mu = 1.09$ [Tejada 2001]. In this report a value of 0.8 will be assumed.

Mechanism C

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

$$L_1 = \min(r, nr_{hs}) \cdot s \quad \dots\dots\dots 0-14$$

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

It is important to remark that when the height of the mechanism is less than the total height of the façade wall, then L_2 is equal to zero. r_{hs} is the number of courses within the storey height and n is the number of storeys.

Mechanism D

$$\ddot{e} = \left[\frac{3T}{2 \min \left(rh, \frac{Lh}{s} \right)} \cdot \frac{L - L_2}{L} + \frac{T}{rh} \cdot \frac{L_2}{L} \right] \quad \dots\dots\dots 0-16$$

A.2.1.1.2. 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.

a) 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.

b) 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].

$$\zeta = \sqrt{\frac{7}{2 + \xi}} \dots\dots\dots 0-17$$

where the damping ξ is given in %.

c) Limit States and displacement capacities

As it specified in above, the limit states for adobe walls shown in Table A- 5 have been derived from some experimental tests.

Table A- 5 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 A- 6).

Table A- 6 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

d) 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.

$$\Delta_y = k_1 \cdot h_r \cdot \ddot{a}_y \dots\dots\dots 0-18$$

$$\Delta_p = \Delta_y + \Delta_p \dots\dots\dots 0-20$$

Figure A- 6).

$$\Delta_e = \frac{\ddot{a}_y^2 \sum_{i=1}^n h^2 m_{f_i} + 2\ddot{a}_y \ddot{a}_p h_{sp} \sum_{i=1}^n h_i m_{f_i} + \ddot{a}_p^2 h_{sp}^2 \sum_{i=1}^n m_{f_i} + M_m}{\ddot{a}_y \sum_{i=1}^n h_i m_{f_i} + \ddot{a}_p h_{sp} \sum_{i=1}^n m_{f_i} + N_m} \dots\dots\dots 0-21$$

$$M_m = \int_0^h m_m (x \ddot{a}_y)^2 dx + \int_{h_1}^{h_f} m_m (x \ddot{a}_y + \ddot{a}_p h_{sp})^2 dx \quad \dots\dots\dots 0-22$$

$$N_m = \int_0^h m_m(x \ddot{a}_y) dx + \int_{h_1}^{h_f} m_m(x \ddot{a}_y + \ddot{a}_p h_{sp}) dx \quad \dots\dots\dots 0-23$$

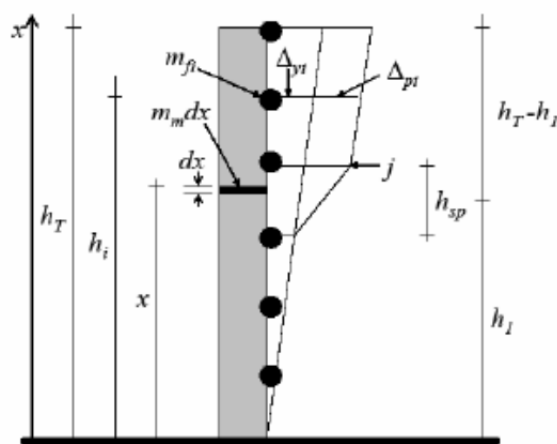


Figure A- 6: Simplified model for the definition of k_2 (Restrepo 2004)

a) *Evaluation of k_1*

The coefficients k_1 can be evaluated in an explicit way equaling the effective displacement Δ_e (having $\mu = 1$) to Δ_{LS} . For example, assuming that for 1-storey building h_1 is measured at the mid-height and $\mu = 1$ ($\delta_p = 0$), then Eq. (0-24), (0-25) and (0-26) are found, where is the total mass of the wall ($m_{wr} = m_w h_T$).

$$\Delta_e = \frac{\ddot{a}_y^2 (h_T^2 \cdot m_{f_1} + M_m)}{\ddot{a}_y (h_T \cdot m_{f_1}) + N_m} \dots\dots\dots 0-24$$

$$M_m = \frac{m_{mr} \ddot{a}_y^2 h_T^2}{3} \dots\dots\dots 0-25$$

$$N_m = \frac{m_{mr} \ddot{a}_y h_T}{2} \dots\dots\dots 0-26$$

Doing $\Delta_e = \Delta_{LS}$ and solving for 1 k it is obtained Eq.(0-27):

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

b) Evaluation of k_2

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

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

$$M_m = \frac{19m_{mT} \delta_y^2 h_T^2}{12} \dots\dots\dots 0-29$$

$$N_m = \delta_y h_T m_{mT} \dots\dots\dots 0-30$$

Replacing the last expressions into Eq(0-21) where $\Delta_e = \Delta_{LS}$ it is obtained the expression for

k_2 , Eq.(0-31):

$$k_2 = \frac{4m_f + \frac{19}{12}m_{mT}}{2m_f + m_{mT}} - k_1 \dots\dots\dots 0-31$$

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

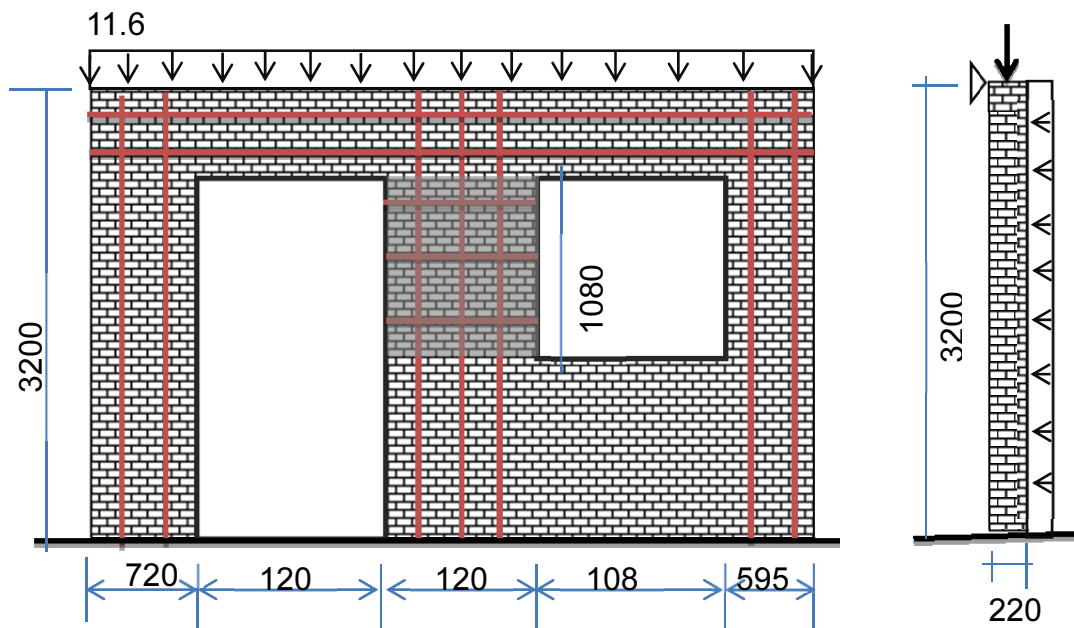
The limit state period of vibration of adobe walls is rewritten for convenience:

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

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.

A.2.1.2. 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 $v_o^* = 3.8$ 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.



$$\begin{aligned}
 l_w &= 1.2 \text{ m} & q &= 11.6 \text{ KN/m} \\
 h_e &= 3.2 \text{ m} & t_w &= 0.22 \text{ m}
 \end{aligned}$$

1 Establishing seismic demands

$$\begin{aligned}
 V^* &= 44.3 \text{ KN} & \text{In-plane seismic force} \\
 v^* &= 3.8 \text{ KN/m}^2 & \text{Out-of-plane uniformly distributed seismic force}
 \end{aligned}$$

2 In-plane seismic demands

$$M^* = 141.76 \text{ KN-m}$$

3 Out-of-plane seismic demands

$$M^* = 5.8368 \text{ KN-m}$$

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

$$\begin{aligned}
 \text{Masonry density} &= 18 \text{ KN/m}^3 \\
 W_w &= 15.2064 \text{ KN} \\
 N_t + 0.5 W_w &= 21.5232
 \end{aligned}$$

Using a permissible maximum TS bar stress at nominal strength

$\sigma_s = f_y$, the nominal out-of-plane flexural strength of the wall is established as,

Assume 10 mmdia twisted bars

$$\begin{aligned}
 f_y &= 1104 \text{ Mpa} \\
 A_h &= 14.8 \text{ mm}^2 & 10 \text{ mmdia} \\
 f_y A_s &= 32678.4 \text{ N} \\
 &= 32.6784 \text{ KN} \\
 d &= 225 \text{ mm} \\
 &= 0.225 \text{ m} \\
 a &= 0.004966 \text{ m}
 \end{aligned}$$

Substituting the values of a and d ,
nominal strength yields,

$$8.148 \text{ KN-m} > 5.84 \text{ KN}$$

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

$$\begin{aligned} b_w &= 220 \text{ mm} \\ l_w &= 1200 \text{ mm} \\ h_e &= 1080 \text{ mm} \\ W_w &= 5.13 \text{ KN} \\ N_t + W_w &= 20.76 \text{ KN} \end{aligned}$$

7 Check for diagonal shear strength

$$V_{dt} = 51.18 \text{ KN}$$

Nominal shear resistance, V_n , is determined as the minimum of resistance corresponding to the bed-joint sliding failure mode, V_s , resistance corresponding to the diagonal tension failure mode, V_{dt} , and resistance corresponding to the toe crushing failure mode, V_{tc}

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

As $\Phi_{fv} V_{dt} < V^*$, the assumed A_v 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.

$$\begin{aligned} V_{dt} &= 63.79 \text{ KN} \\ \Phi_{fv} V_{dt} &= 47.84 \text{ KN} > V^* = 44.3 \text{ KN} \end{aligned}$$

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

ANNEX B: ANALYSIS AND RETROFITTING DESIGN OF SULTAN DAKI HIGH SCHOOL, URI BLOCK, DIST. BARAMULA, KASHMIR (CASE STUDY)

One of the six buildings in the school had some earthquake damage up to G3. It is a typical school building of the area with CGI sheet roof supported on stone masonry walls. The large-scale casualties among school children in Pakistani Kashmir clearly points to the need for vulnerability reduction in these structures. In addition, the building system is no different from the typical houses of the area, so the lessons learnt in this case are also relevant to houses. The building has three rooms in a line with a verandah in front and a roof supported on wood posts. The roof is of CGI sheets with timber under-structure. The entire building has an attic floor which keeps the winter cold out. Similar to other buildings in the area the openings are large with small piers in between on front wall.

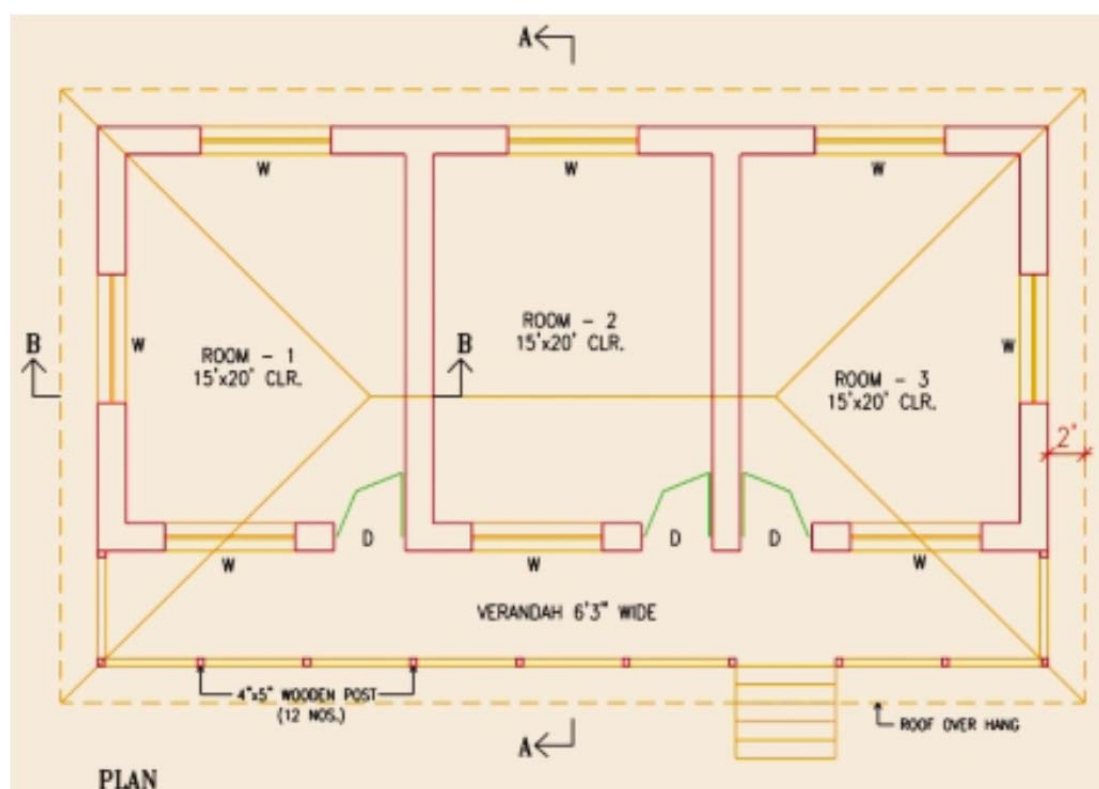
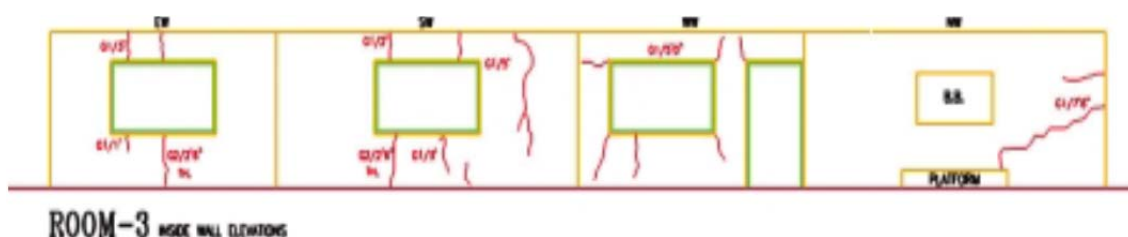
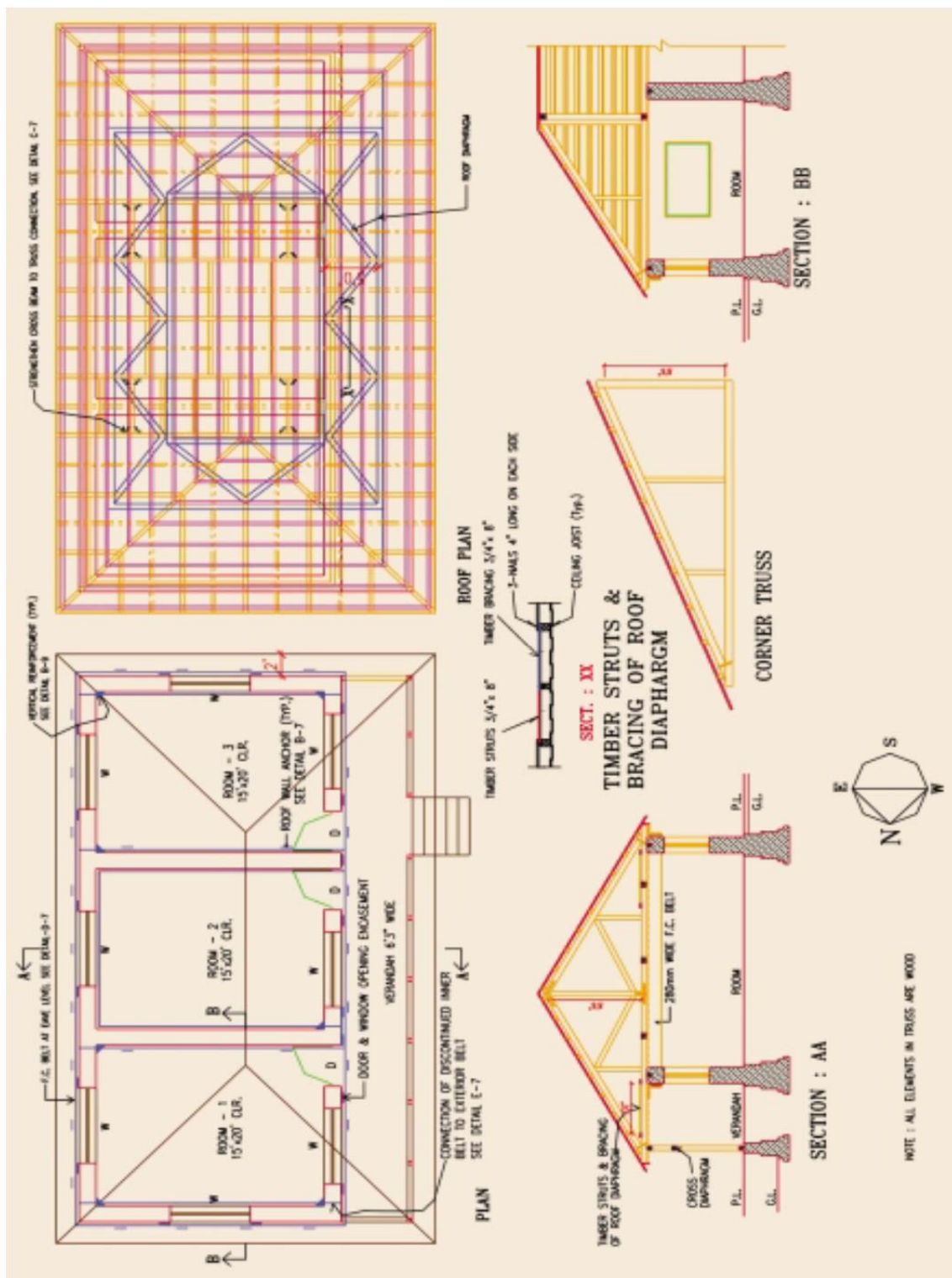


Figure A- 7: Drawing for the restoration and retrofitting



(Source: UNDP, UNESCO, GOI, 2007)



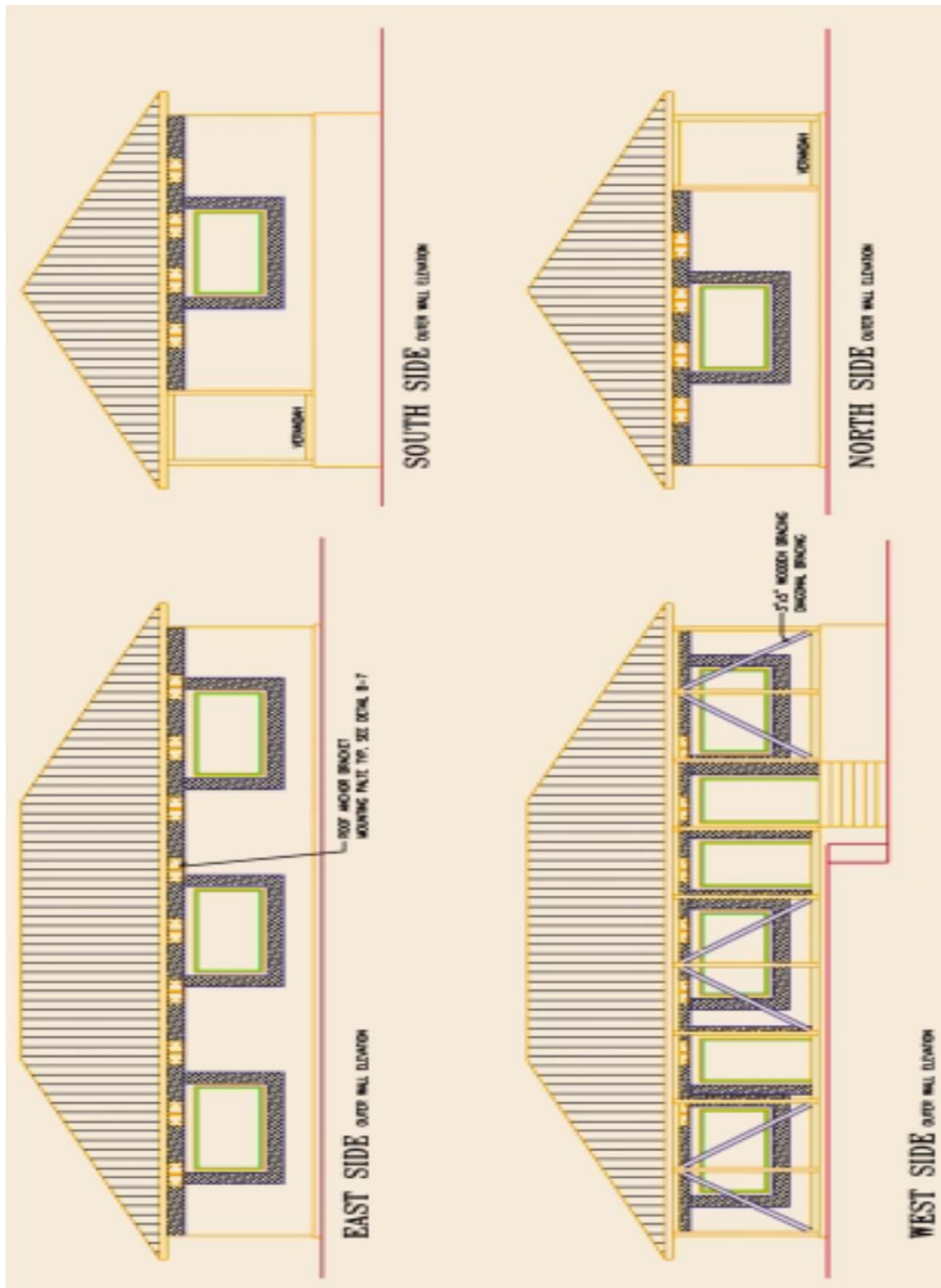






Figure A-1: Pictures of Restoration & Retrofitting of the School Building
(Source: UNDP, UNESCO, GOI, 2007)



Figure A- 2: Existing 3 room school building before retrofitting



Figure A- 3: Typical cracking in corners and at corners of opening such as windows



Figure A- 4: Bond elements yet to be completed in RR wall



Figure A- 5: Grouting no-shrink grout in crack with hand pump



Figure A- 6: Corner crack splicing with WWM

Installing Seismic Belt at Eave Level and Around the Opening



Figure A- 7: Preparing WWM with 6 mm rebars



Figure A- 8: Installing WW mesh from a roll



Figure A- 9: Tying encasement reinforcement to shear connector dowel



Figure A- 10: Installing belt reinforcement

After Deciding the Alignment of the Seismic Belt Details were Evolved to Ensure its Continuity at Every Point in its Alignment



Figure A- 11: Inserting dowel for belt to belt connection at corner



Figure A- 12: continuity at corner behind wood post obstruction



Figure A- 13: Installing 'L' shaped dowel at corner to ensure continuity



Figure A- 14: plying cement-sand plaster

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



Figure A- 15: Installing vertical reinforcement in the corner



Figure A- 16: Vertical reinforcement concreting formwork



Figure A- 17: Anchoring vertical bars over wall-plate at attic deck



Figure A- 18: Installing bolts and brackets for roof-wall connection



Figure A- 19: Installing bolts and brackets for roof-wall connection



Figure A- 20: Retrofitted building with belts, roof anchoring and diagonal timber bracings

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

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