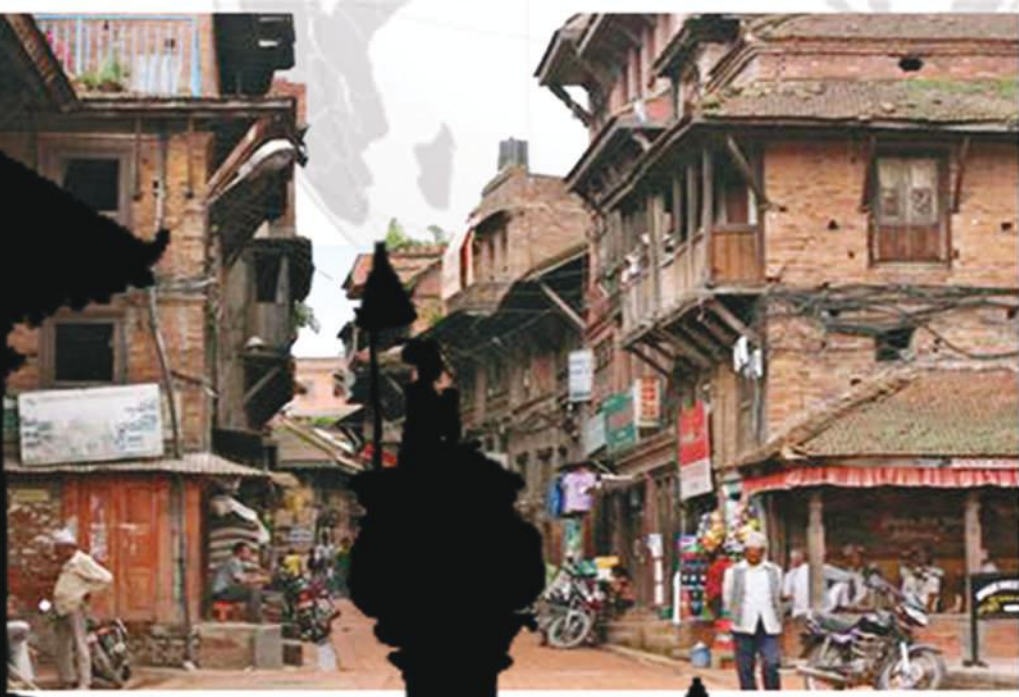


SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL

2016

ADOBE



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.

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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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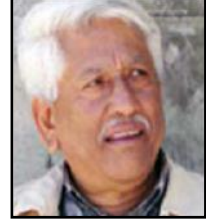
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MESSAGE

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.

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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.

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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 academician 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 helpful in raising the safety awareness and making to community disaster resilient.

My sincere thanks goes 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.

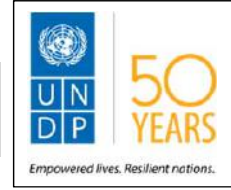
Also, my thanks go to United Nations Comprehensive Disaster Risk Management Program, CDRMP for the support during the preparation of the guideline and publication as well.

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 Illam 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 Julliard
UNDP Resident Representative &
United Nations Resident Coordinator

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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.

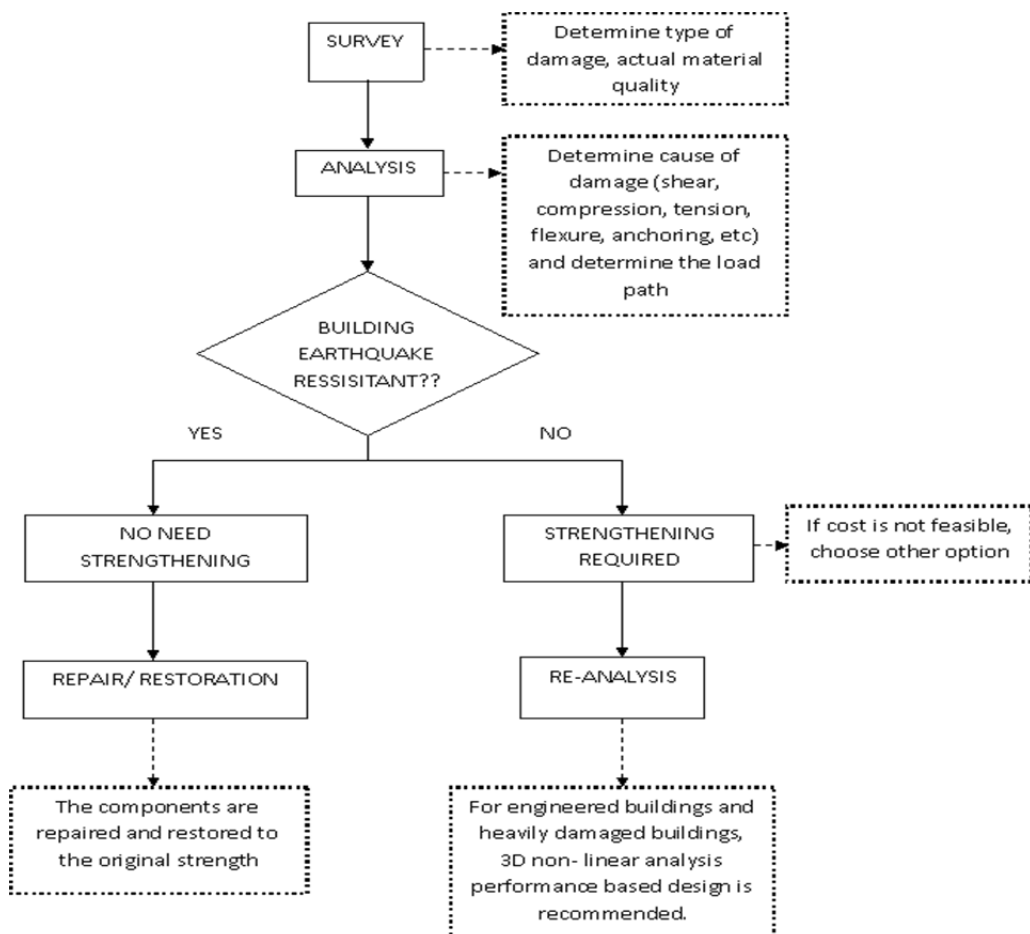


Figure 1.1 Stepwise Process of Seismic Retrofitting Of Building

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

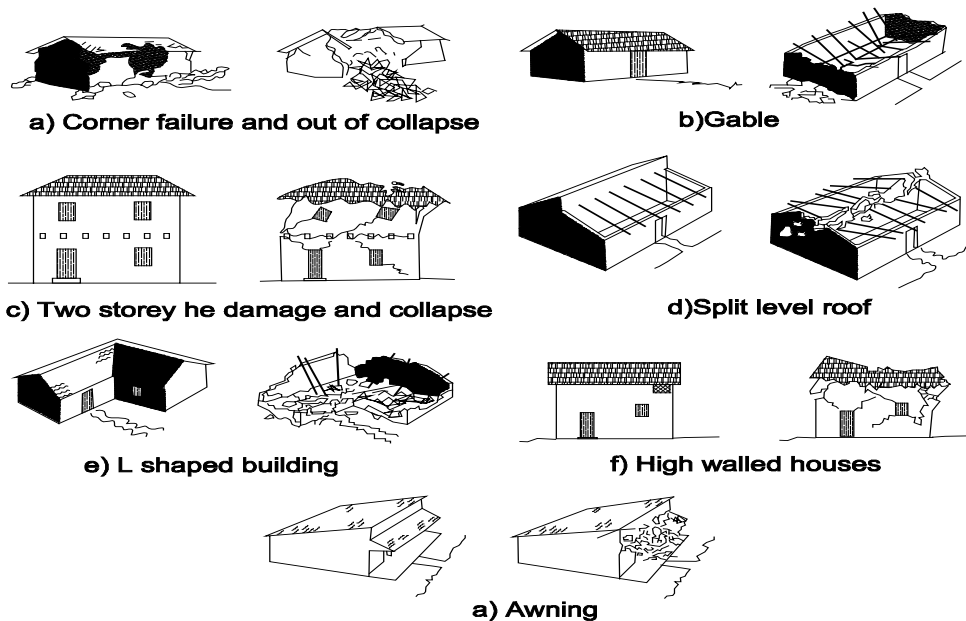


Figure 2.5 Damage Typology

(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.

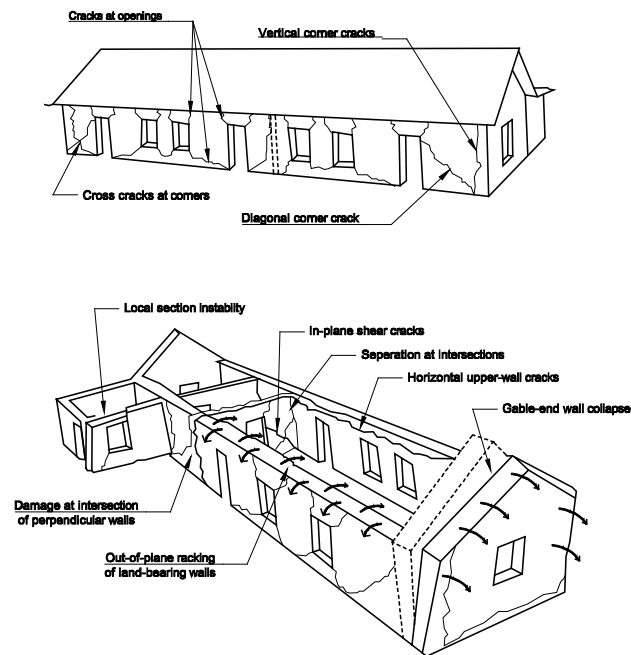


Figure 2.6 Typical damage modes observed in adobe buildings
(Source: Manual of The Getty Conservation Institute)

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.

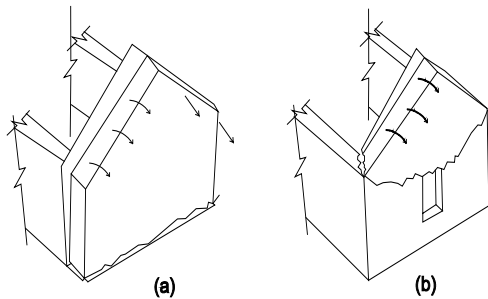


Figure 2.9 Gable-end wall collapse: (a) overturning at base of wall, and (b) mid-height collapse
(Source: Manual of The Getty Conservation Institute)



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.

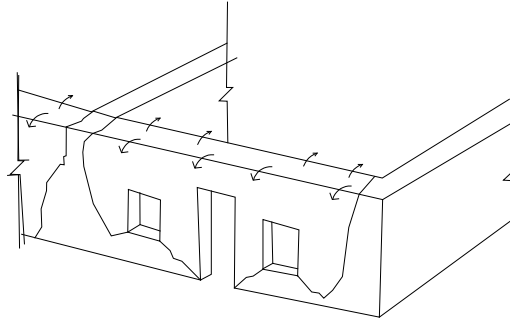


Figure 2.12 Out-of-plane flexure of load bearing wall
(Source: Manual of The Getty Conservation Institute)

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.

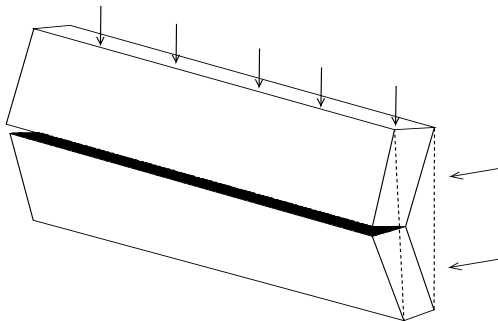


Figure 2.13 Sketch of mid-height out of plane failure
(Source: Manual of The Getty Conservation Institute)



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.

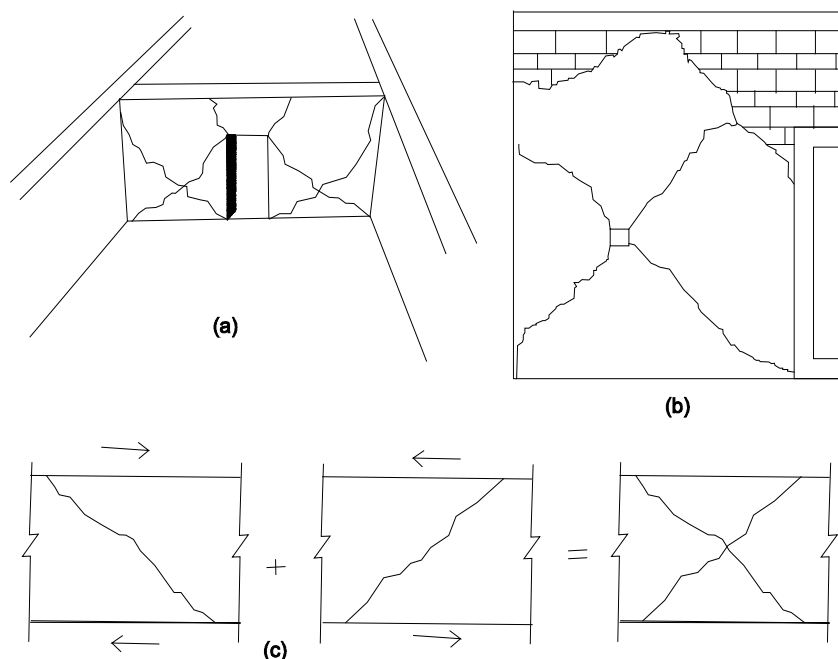


Figure 2.15 Illustrations show (a) drawing of X-shaped shear cracks in an interior wall; (b) typical X pattern (Leonis Adobe, Calabasas, Calif.); and (c) how X-shaped cracks result from a combination of shear cracks caused by alternate ground motions in opposite directions.

(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.

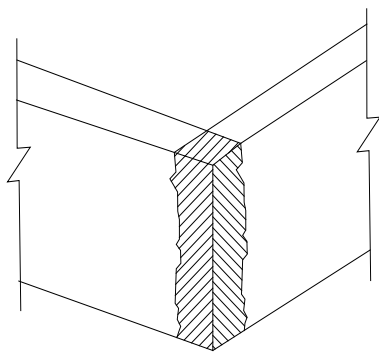


Figure 2.16 Illustrations showing (a) how vertical cracks at corner can lead to instability of intersection
(Source: Manual of The Getty Conservation Institute)



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.

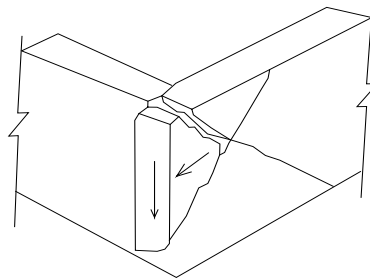


Figure 2.21 Illustration showing how combination of shear and flexural cracks can result in corner displacement or collapse
(Source: Manual of The Getty Conservation Institute)

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.

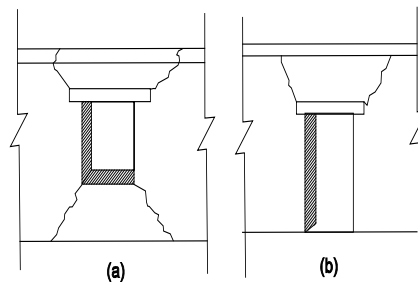


Figure 2.22 Illustration of cracks originating at stress concentration locations: (a) cracks appearing first at upper corners of window opening followed by lower corner cracks; and (b) cracks at upper corners of door opening.

(Source: Manual of The Getty Conservation Institute)

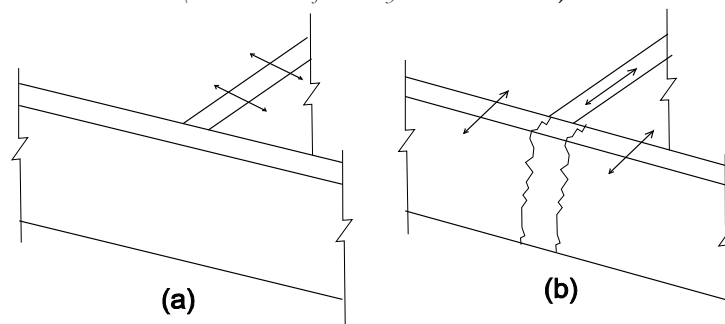


Figure 2.23 Illustrations showing (a) how separation can occur between in-plane and out-of-plane walls, and (b) how vertical cracks develop in out-of-plane walls at the intersection with perpendicular, in-plane walls.

(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*

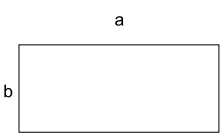
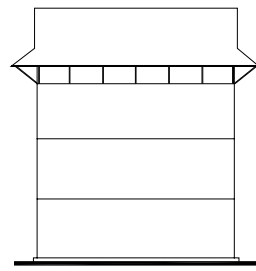

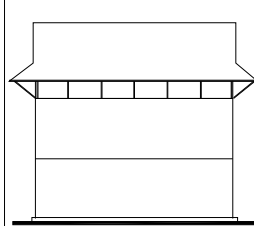
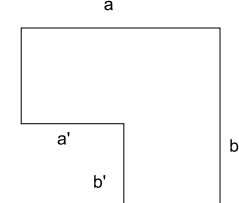
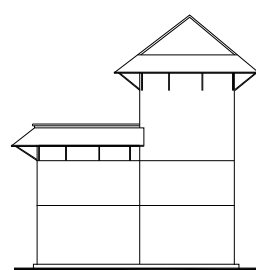
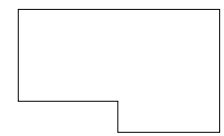
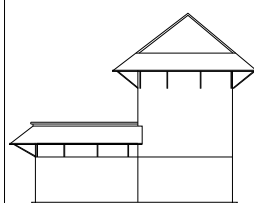
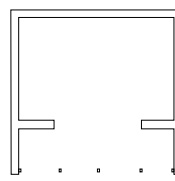
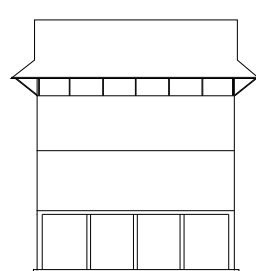
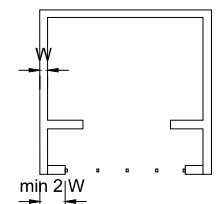
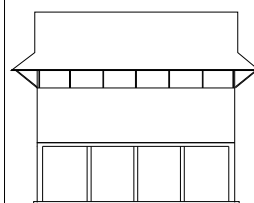
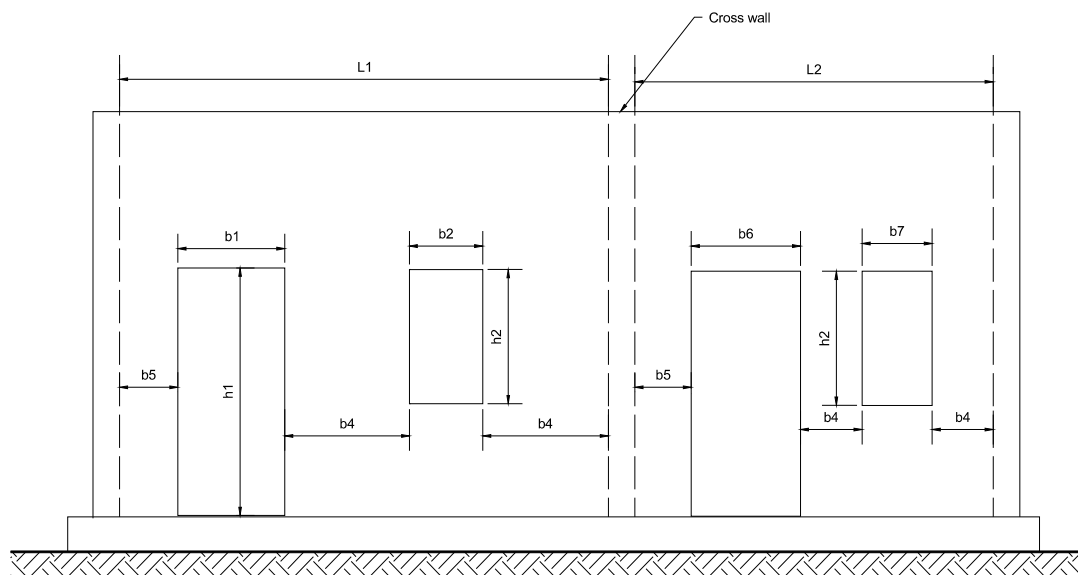
NO		YES	
PLAN	ELEVATION	PLAN	ELEVATION
 <p>$a \geq 3b$</p>		 <p>$a \leq 3b$</p>	
 <p>$a = b$ $a' = b'$</p>			
		 <p>W $\text{min } 2W$</p>	
	MORE THAN 3 STOREYS		MAXIMUM STOREYS 2 + ATTIC

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.

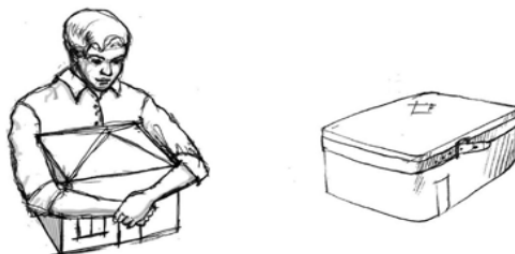


Figure 4.3 A seismic band acts like a belt
(Adopted from: GOM 1994)

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.

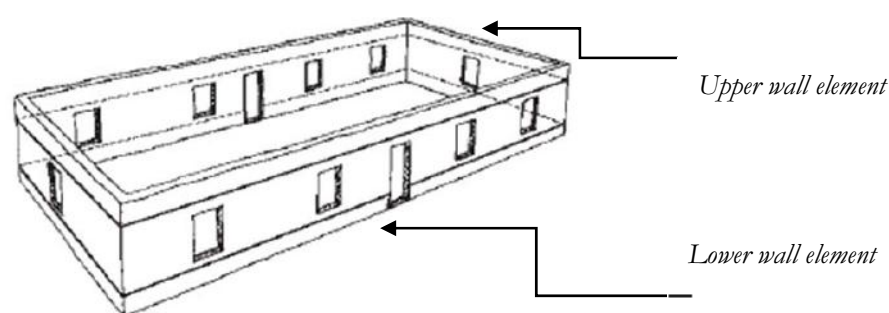


Figure 4.4 Seismic belt showing upper and lower wall elements
(Source: Manual of The Getty Conservation Institute)

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

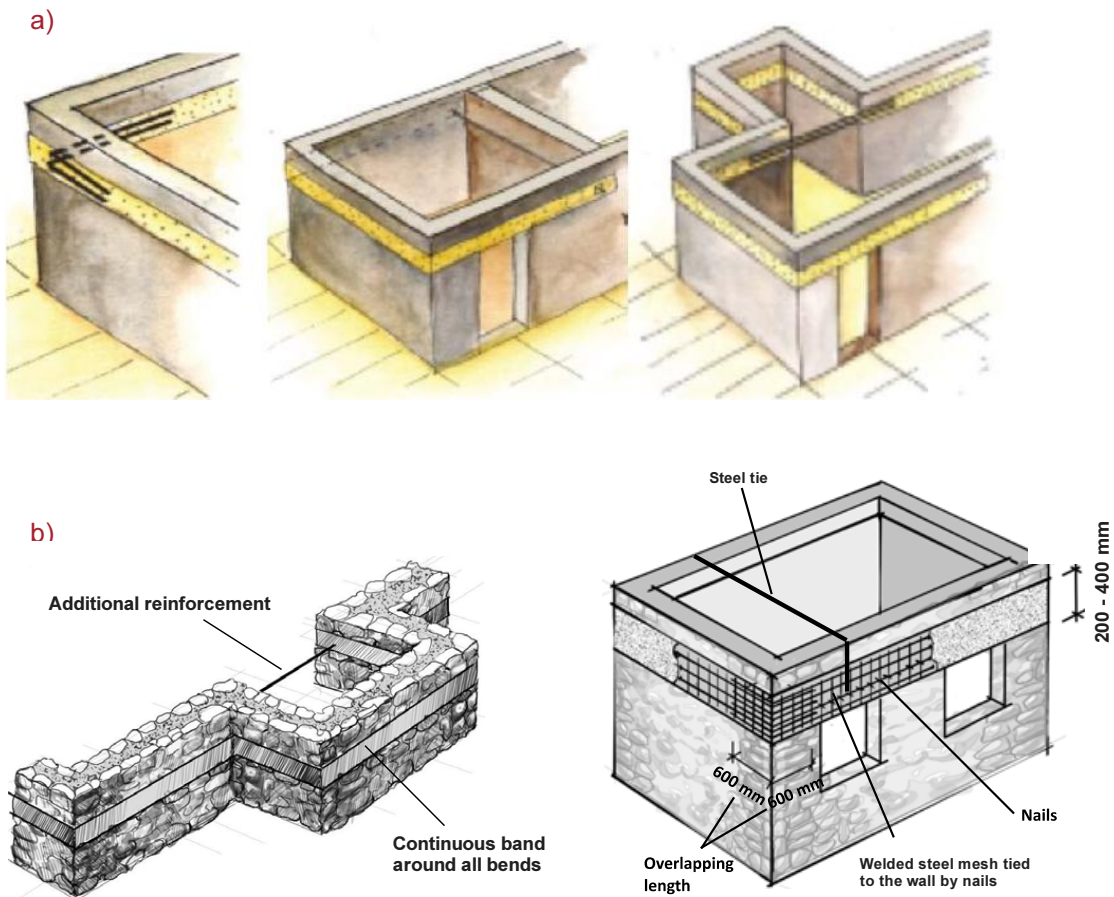


Figure 4.5 a) Seismic belts around various locations (Source: UNDP, UNESCO & GOI, 2007)
 b) Additional reinforcement (Source: R. Desai and GOM 1998)

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.

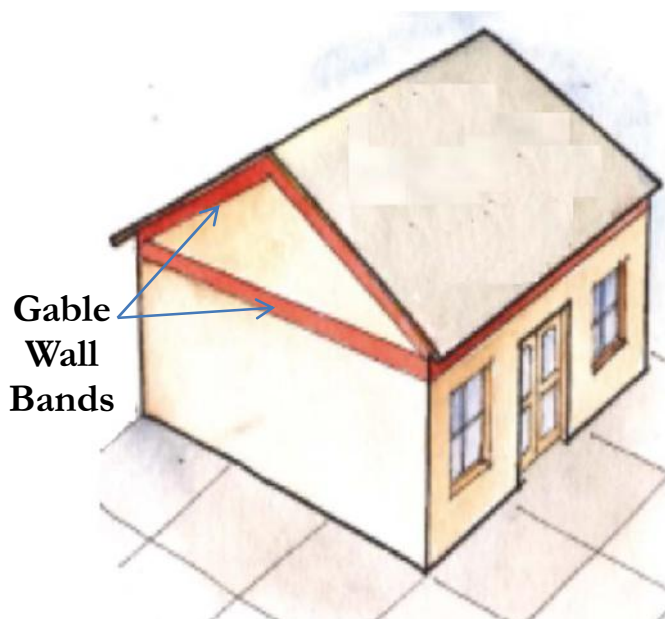


Figure 4.6 Gable band

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.

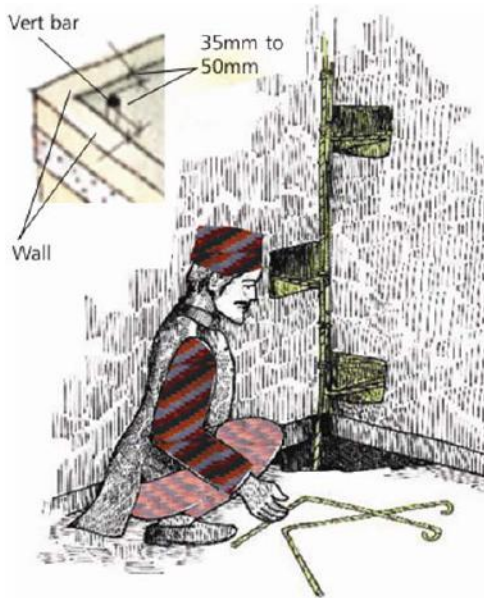


Figure 4.7 Vertical bar in corner
(Adapted from: UNDP, UNESCO & GOI, 2007)

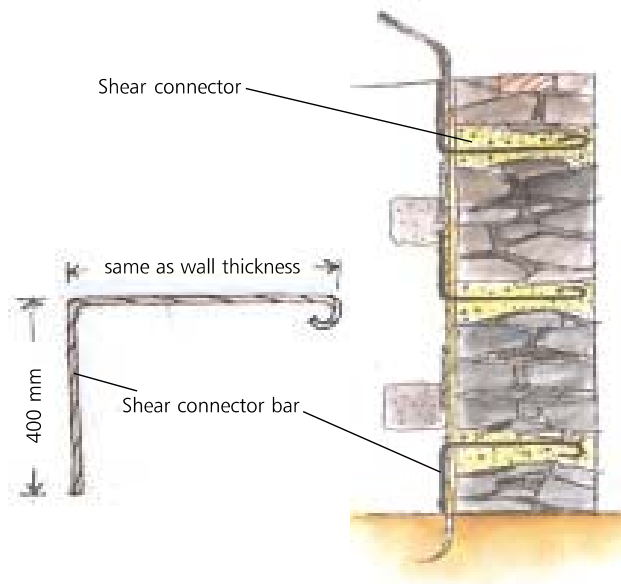


Figure 4.8 Shear connector with vertical bar details
(Source: UNDP, UNESCO & GOI, 2007)

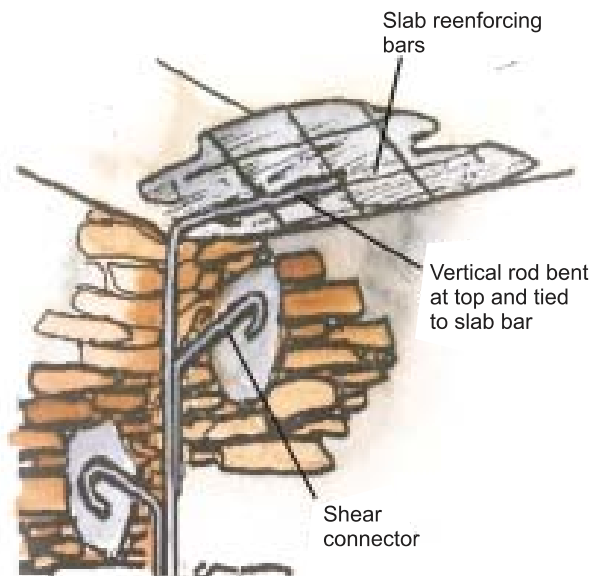


Figure 4.9 Connecting top bent end of vertical rod to slab
(Source: UNDP, UNESCO & GOI, 2007)

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.

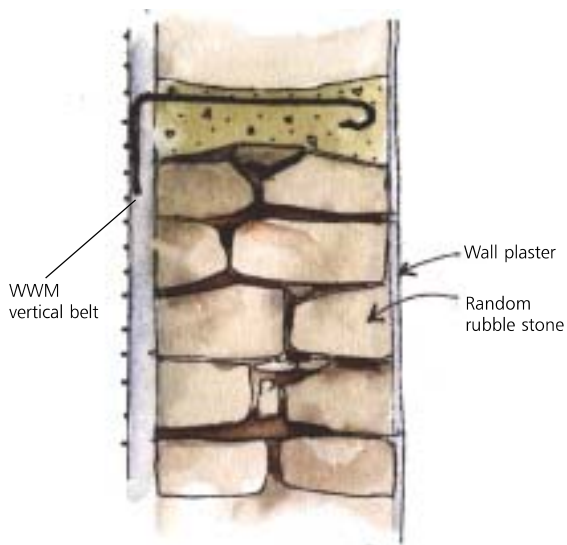


Figure 4.10 Anchoring WWM to random rubble wall with shear Connector
(Source: UNDP, UNESCO & GOI, 2007)



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

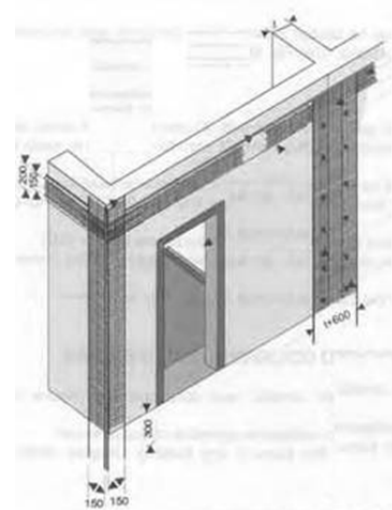
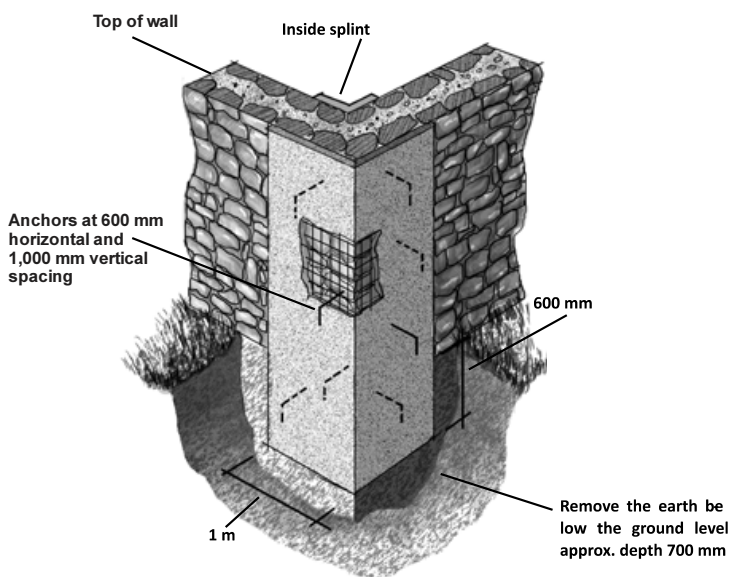


Figure 4.12 Vertical reinforcement with WWM
(Source: GOM 1998)

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

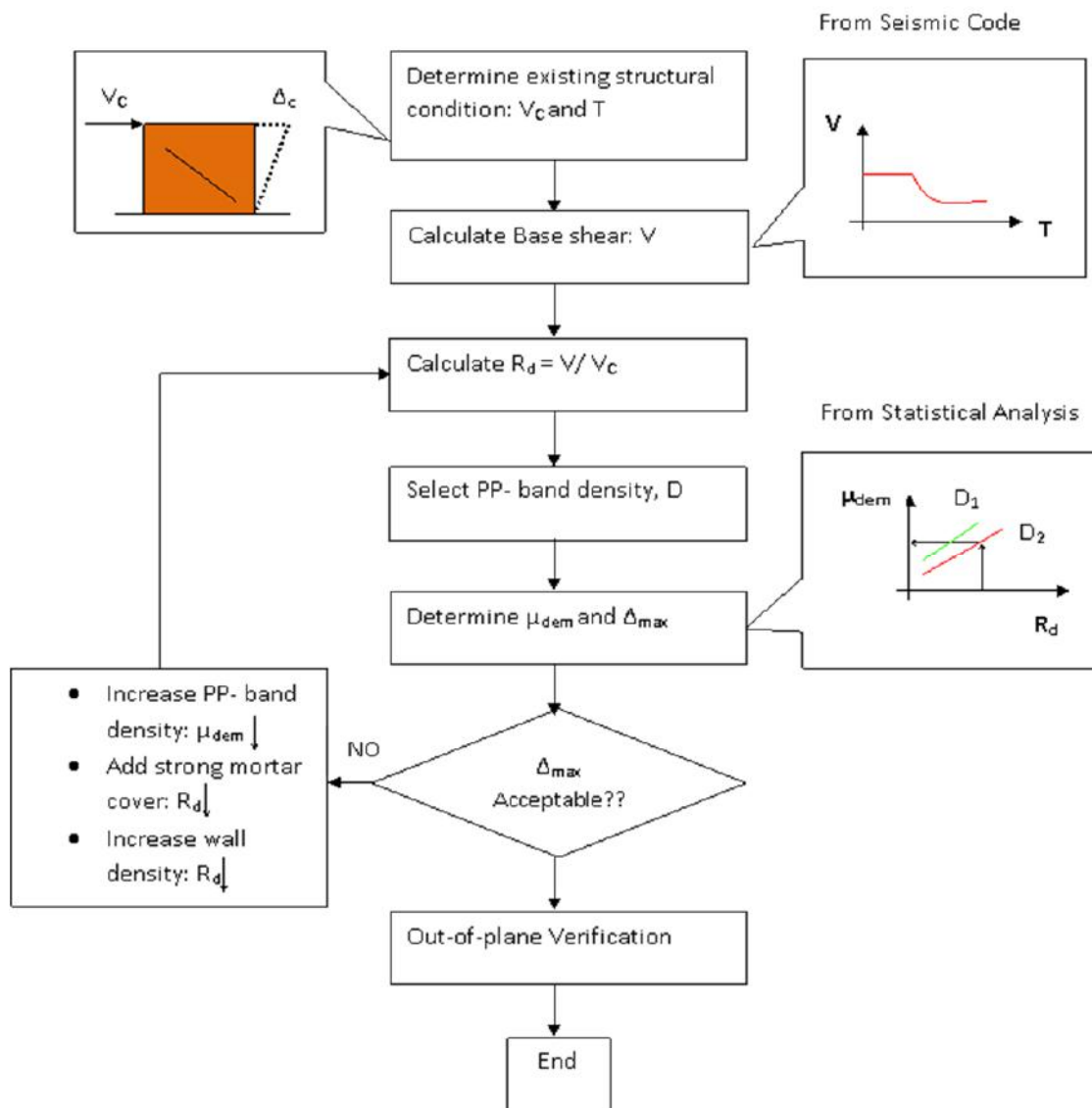


Figure 4.17 Flow chart of Design of PP band (Wall Jacketing)

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:

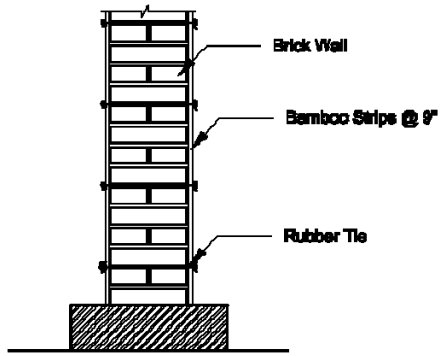


Figure 4.19 Plan showing bamboo reinforcing

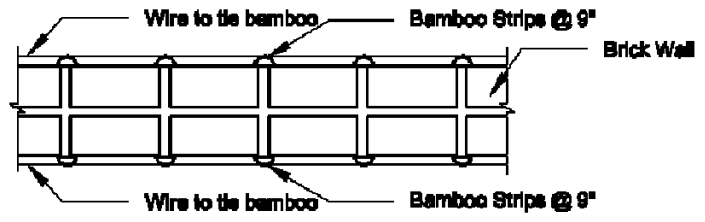


Figure 4.18 Wall section showing bamboo reinforcing



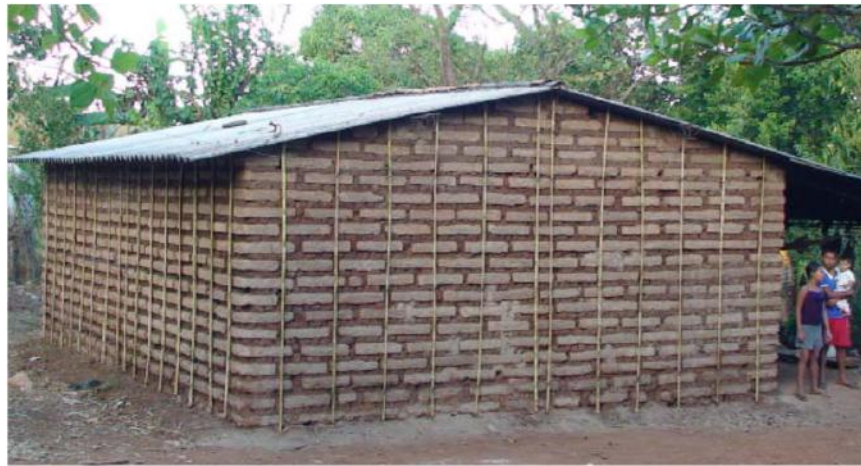


Figure 4.20 Construction procedure of bamboo reinforcement
(Source: www.wuakesafeadobe.net)

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.

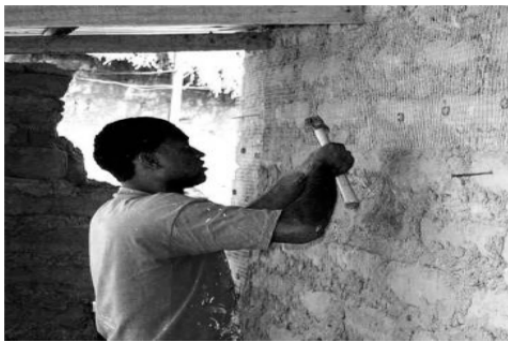


Figure 4.22 Placing the mesh on the wall

(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.

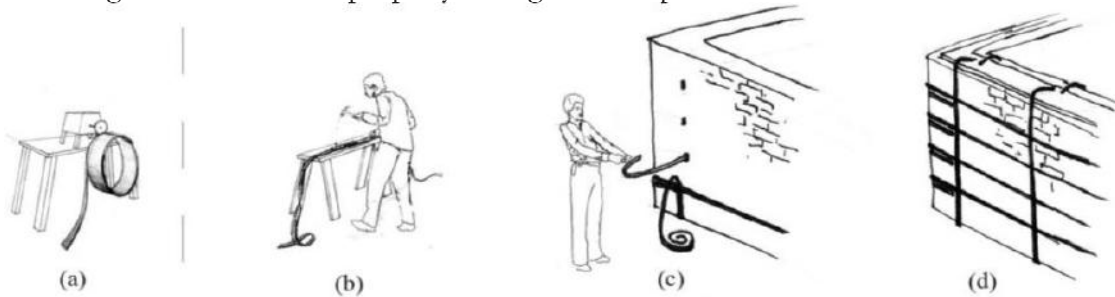


Figure 4.25 Steps in the process of reinforcing an earthen (adobe) house with tire straps.

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.

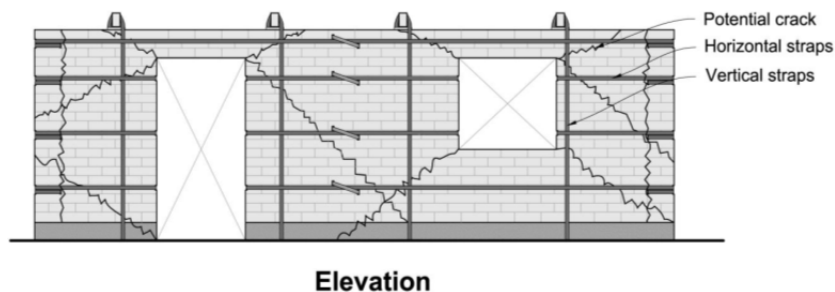


Figure 4.26 An elevation of a typical wall showing positions of expected cracks and strap

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

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*

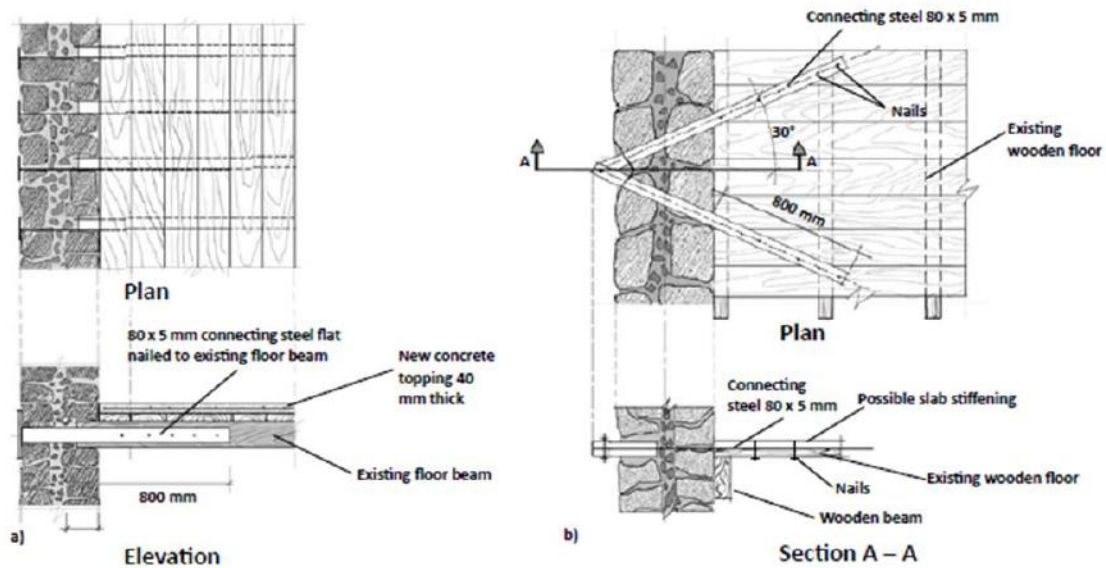


Figure 4.27 Steel straps for wall-to-floor anchorage: a) floor beams perpendicular to the wall, and b) floor beams parallel to the wall
(Source: UNIDO 1983)

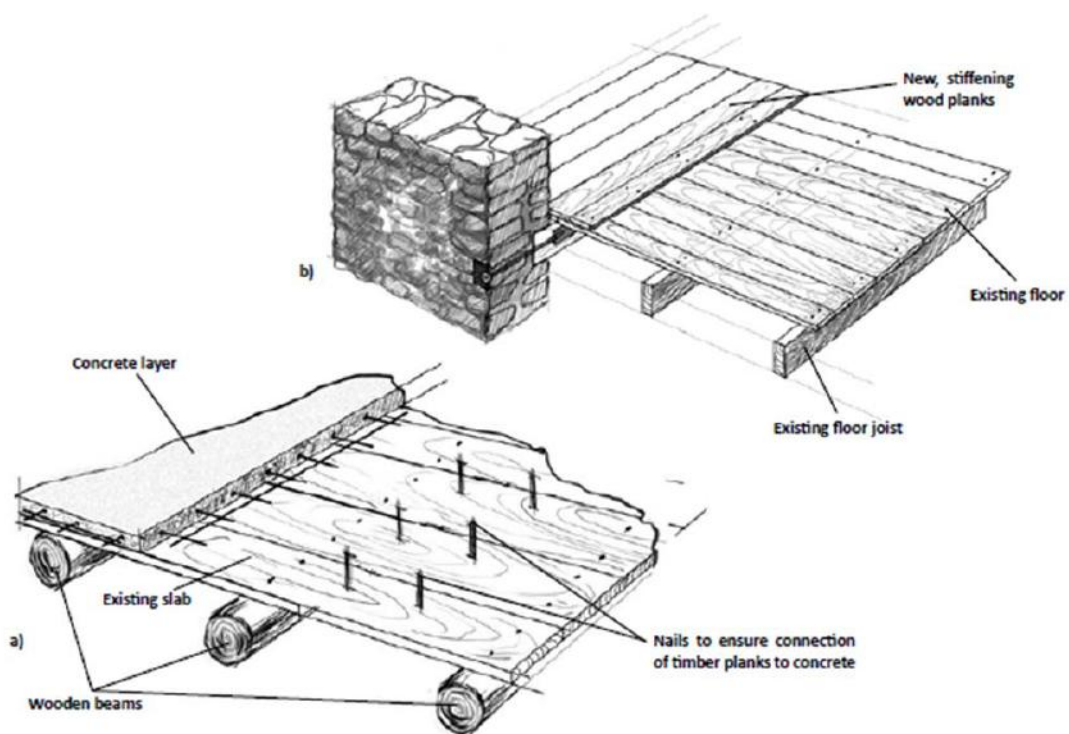


Figure 4.28 Stiffening the floor structures: a) RC topping, and b) new timber planks
(Source: UNIDO 1983)

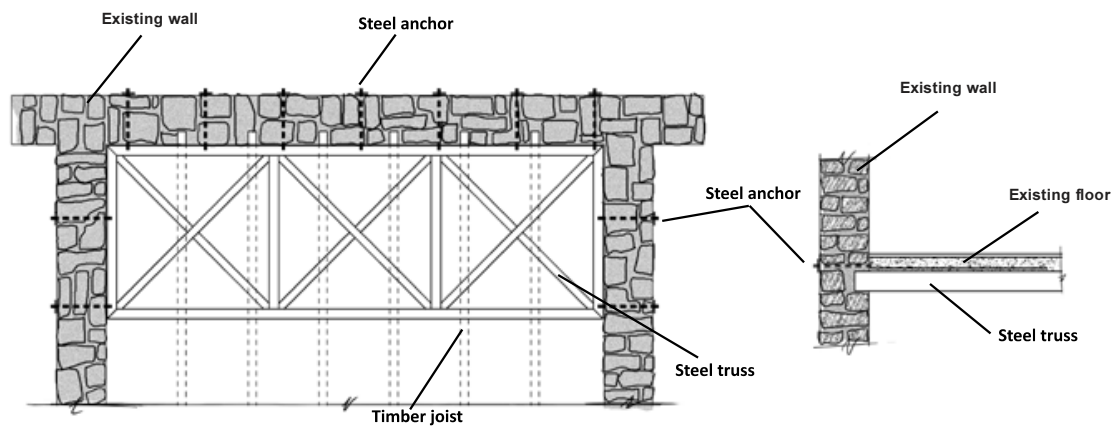


Figure 4.29 Retrofitting the floor and roof structures: a) diagonal brace
(Adapted from: Tomazevic 1999)

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.

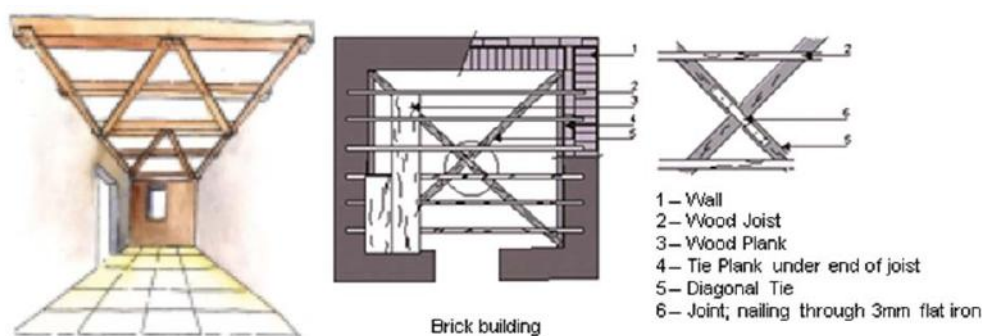


Figure 4.30 Stiffening flat wooden roof



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.

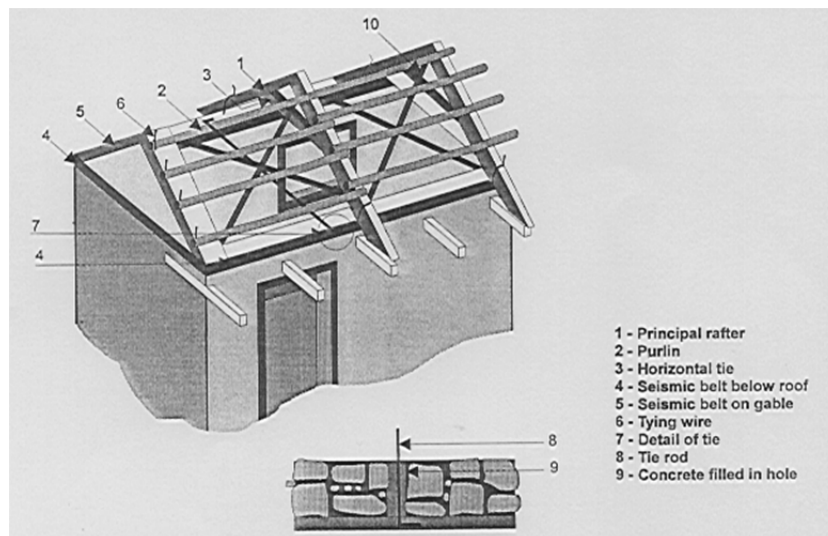


Figure 4.32 Stiffening of sloping roof structure

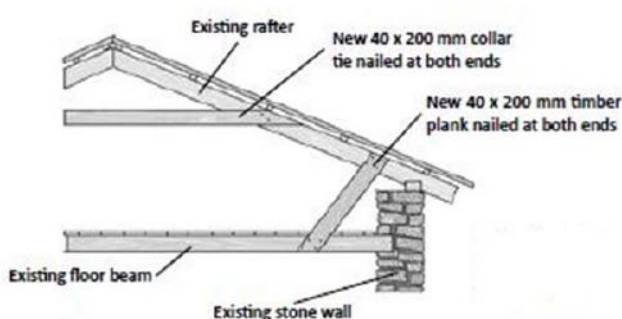


Figure 4.33 Roof rafters tying to ceiling joist



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. 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.

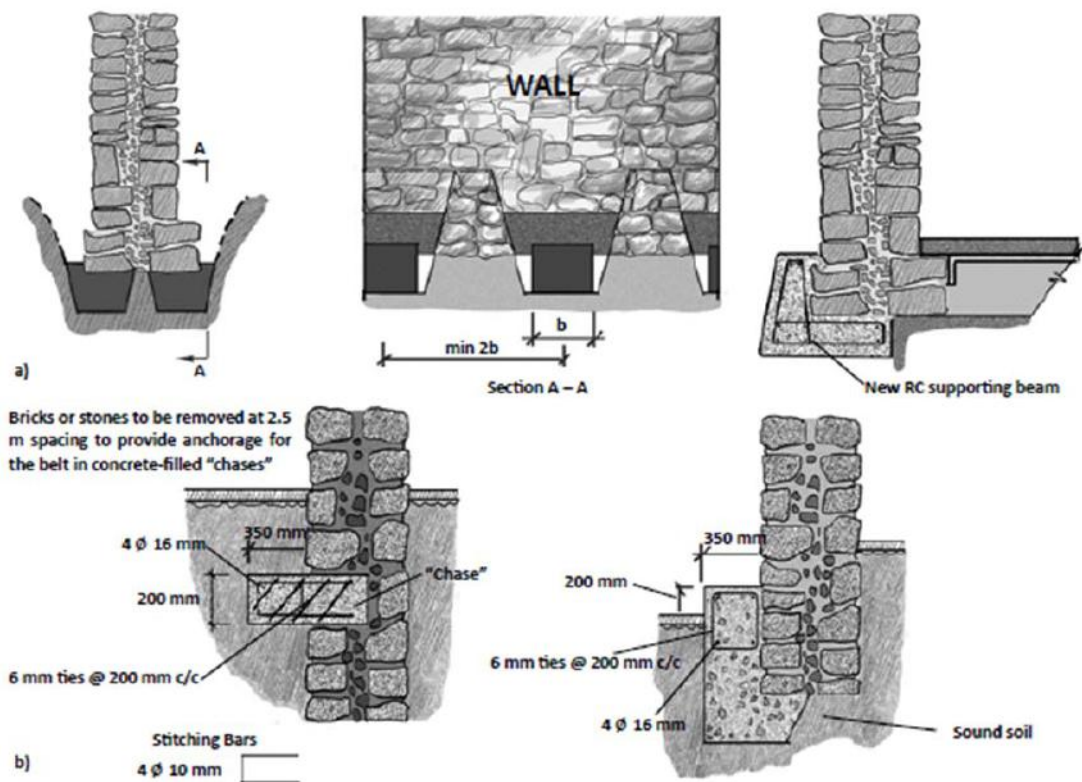


Figure 4.35 Strengthening existing foundations: a) underpinning the foundation, and b) external RC belt
(Adapted from: GOM 1998 and UNIDO 1983)

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.

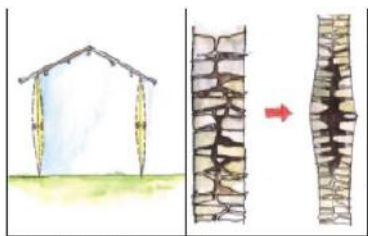


Figure 4.36 Bulging of wall wythes
(Source: UNDP, UNESCO & GOI, 2007)



Figure 4.37 Delamination

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

¹² Guidelines For Repair, Restoration And Retrofitting Of Masonry Buildings In Kachhbh Earthquake Affected Areas Of Gujarat, Gujarat State Disaster Management Authority Government Of Gujarat, March - 2002

Installing cast in-situ reinforced concrete bond element:

I)



Figure 4.38 Marking points for bond elements

II)



Figure 4.39 Removing stone by small rod

III)

\



Figure 4.40 Making dumb-bell shaped hole through wall

IV)

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Figure 4.41 Placing steel bar and filling concrete

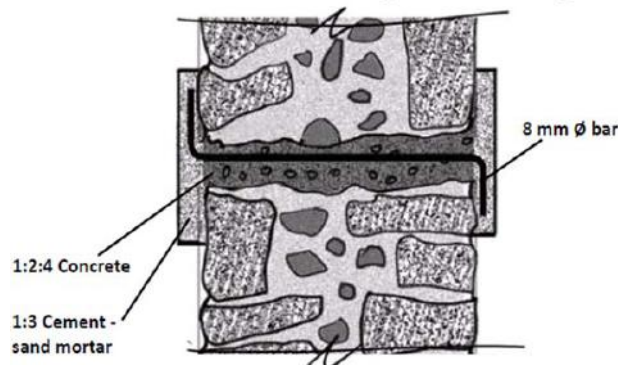


Figure 4.42 Cross section of through-stone
(Source: UNDP, UNESCO, GOI, 2007)

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.

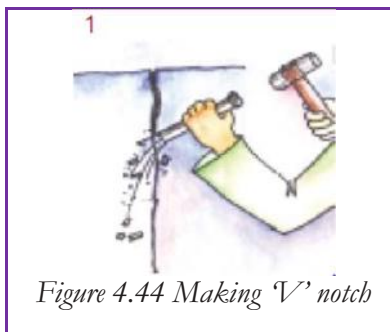


Figure 4.44 Making 'V' notch

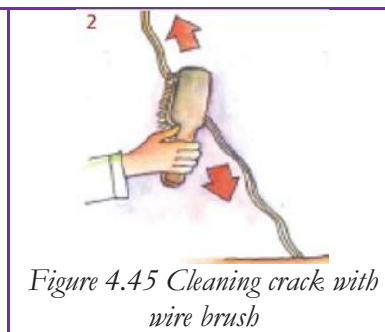


Figure 4.45 Cleaning crack with wire brush

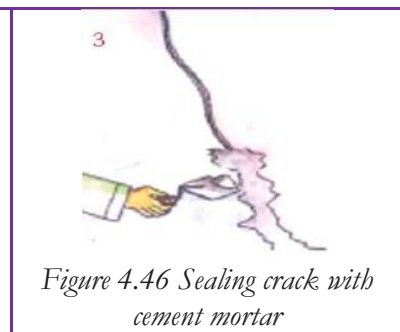


Figure 4.46 Sealing crack with cement mortar

(Source: UNDP, UNESCO & GOI, 2007)

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

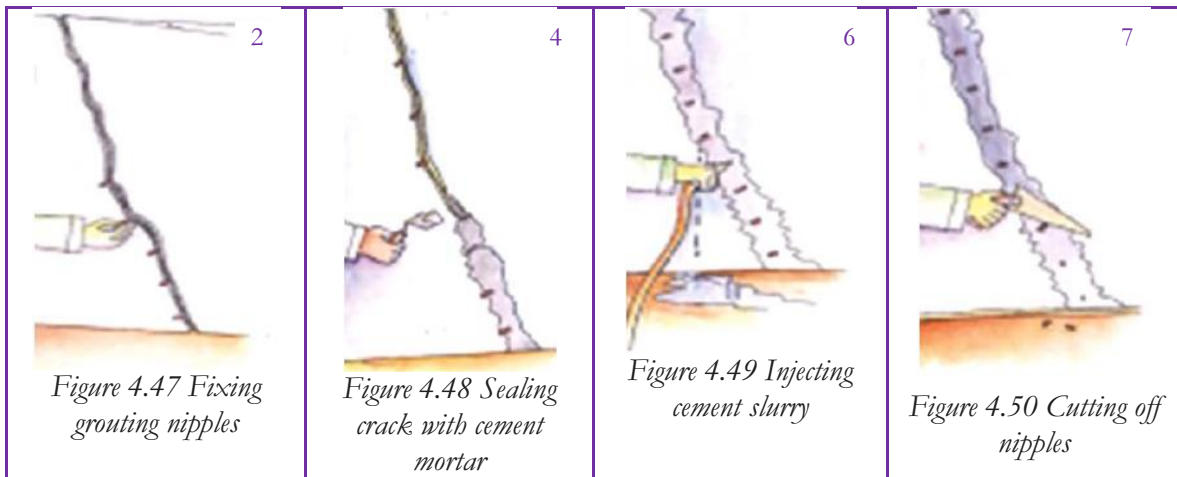
¹⁴L. Keeffe: Earth Building: Methods and materials, repair and conservation, Taylor & Francis, 2005, London, UK.

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.

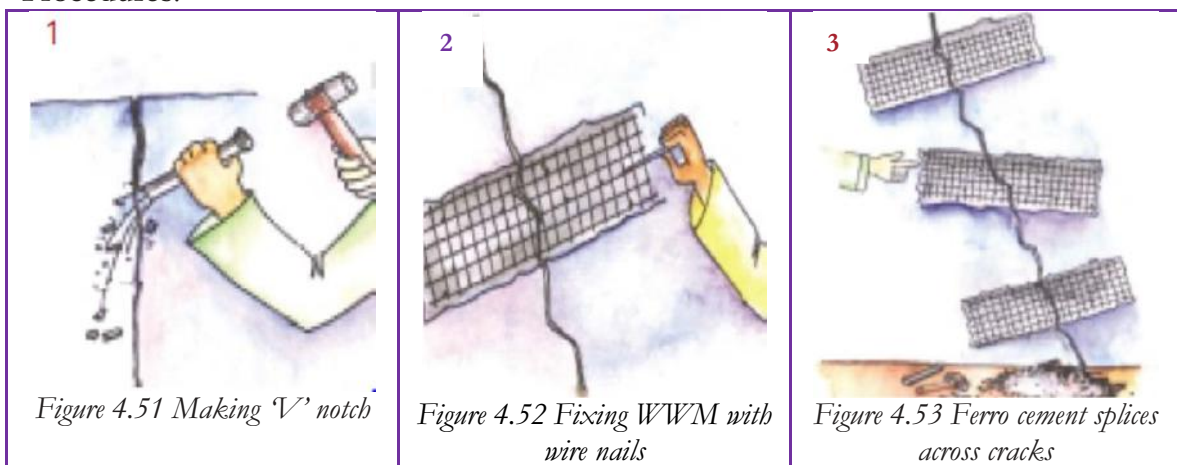


(Source: UNDP, UNESCO, GOI, 2007)

- 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:

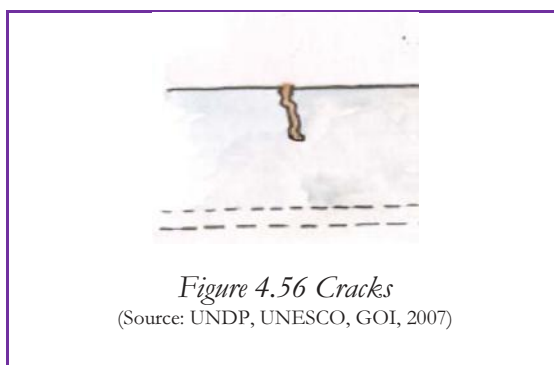


Figure 4.56 Cracks
(Source: UNDP, UNESCO, GOI, 2007)

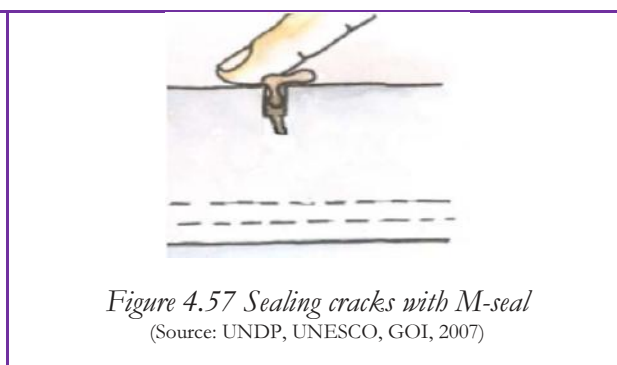


Figure 4.57 Sealing cracks with M-seal
(Source: UNDP, UNESCO, GOI, 2007)

- 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.1.1 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 at house and buildings including 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 guideline.

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 guidelines 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 allow able 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 the min 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 over all 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 event of an 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 are no drawings available, an attempt can be made to obtain some information based on interviews with those who were involved in the design and construction of the building or familiar with the contemporary method of construction, and the owners/residents. Different seismic vulnerability factors are checked and expected 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 by 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 characteristics, 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 knowledgeable 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 methods of 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 tests such as the use of bar scanner, test hammers and Ultrasonic testing instruments or by destructive tests as drilling in walls, scraping 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 evaluated is 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 into sets that have similar behavioral characteristics.

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/ems_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 inadequate and the building is in imminent danger of collapse in the event of an 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 a 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 in ANNEX I. The first phase assessment is a general seismic vulnerability assessment method based on a 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 a second phase assessment or concludes the evaluation and states that potential deficiencies are identified. The second phase assessment involves a more detailed seismic evaluation with a complete analysis of the building for seismic strengthening measures as modifications to correct/reduce seismic deficiencies identified during the evaluation procedure in the first phase. Detailed information about the building is required for this step of evaluation. Seismic retrofit becomes necessary if the building does not meet the 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 characteristics 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 there rebound testing has been performed. The core samples are then subjected to compression tests. There 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 are bard erector 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 brand allow 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 brand so 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 visuals pike in the read out is encountered.

The probe is passed back and for the 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 as 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 light 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 *nythe* of a multi-*nythe* wall. The engineer should verify that the exterior *nythe* 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 *nythes*.

Test values from exterior *nythes* may produce lower values when compared with tests conducted on inner *nythes*. The difference can be due to weathering of the mortar on the exterior *nythes*. 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 due 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.

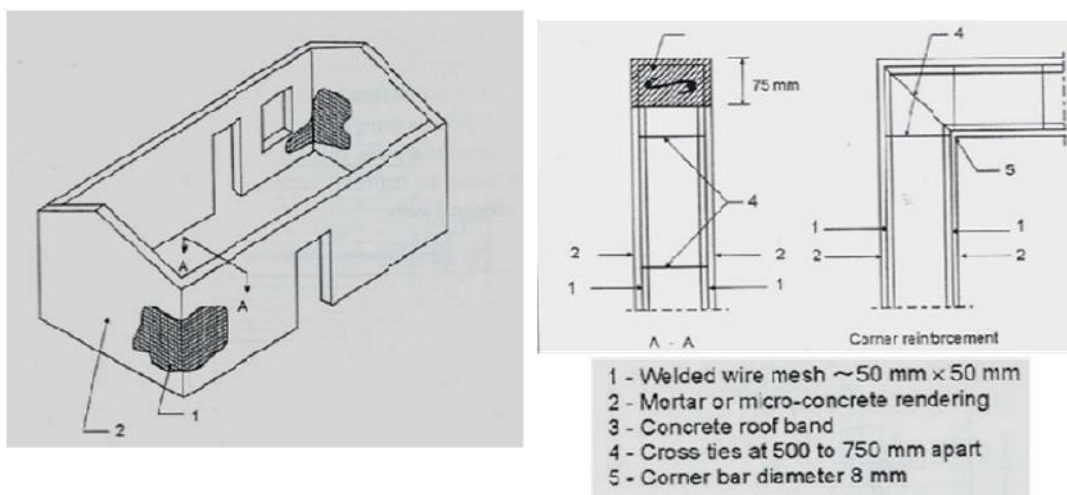


Fig A.I.4.(a) General Scheme of Jacketing



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

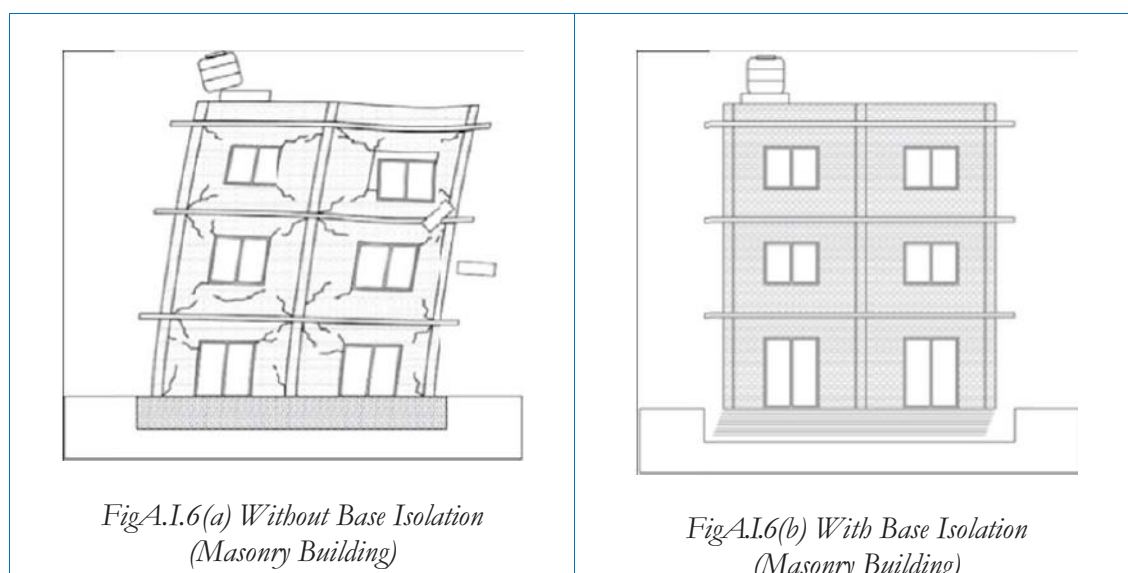
Confinement with reinforced concrete elements (beam and columns) make the existing masonry act as “confined masonry” in the sense that rein forced concrete elements a reinserted surrounding the wall panel or middle of the long wall, allowing the entire wall, or its portion, to act as a truss element, where the struts are inclined strip of unrein forced 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.

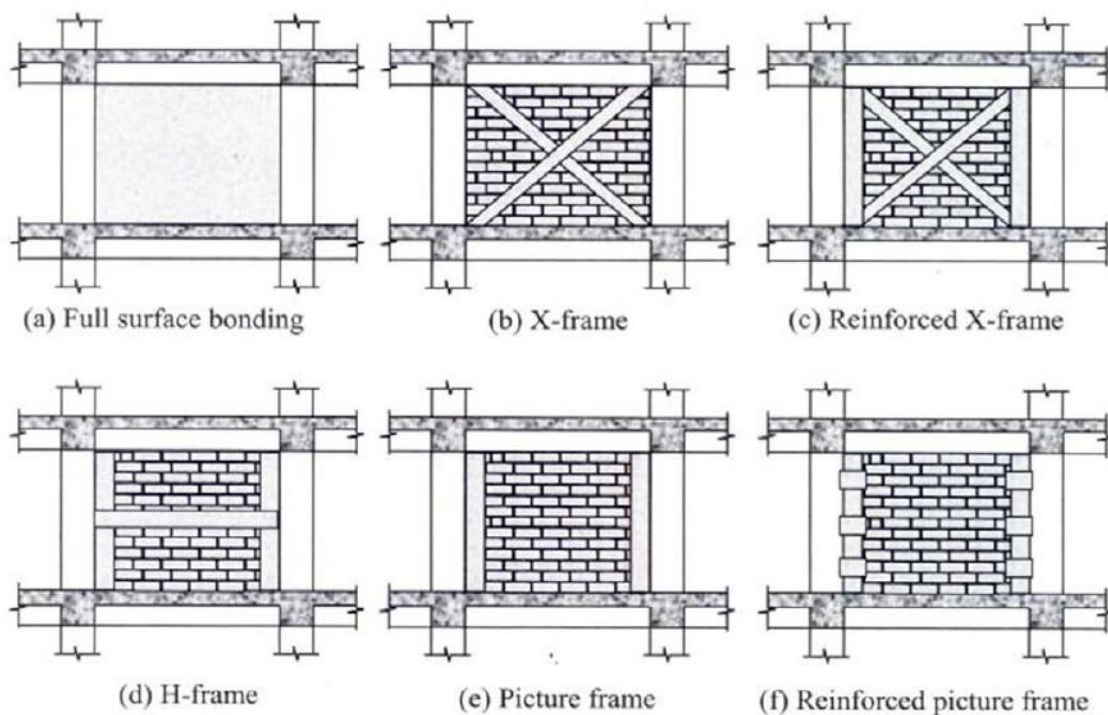


Fig A.I.7 Configurations of FRP Laminates of Masonry Walls

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 3storey	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 sand 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 sand 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 there for every important to develop retrofit solutions for each building on a case-by-case basis. Most of these retrofit technique shave 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 basket 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 short Crete 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 inter section 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 incase 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.

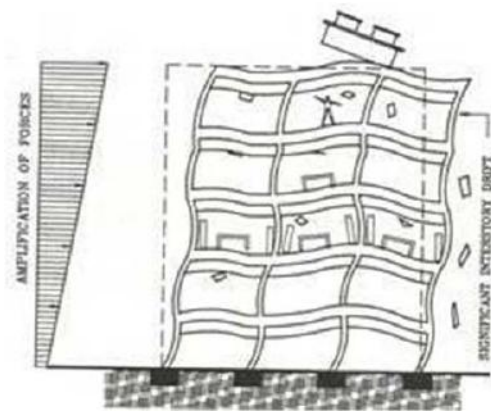


Fig A.I.10 (a) Without Base Isolation
(RC Frame Building)

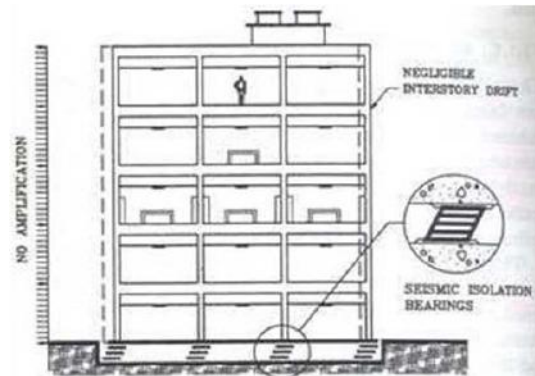


Fig.A.I.10 (b) With base isolation (RC Frame)

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 (*Nateghiand.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 incorporation 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 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 PROPORTION IN PLAN: The breadth to length ratio of the building shall be within 1 : 3. The breadth to length ratio of any room or area enclosed by load bearing walls inside the building shall be also within 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 within 1.2m 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.

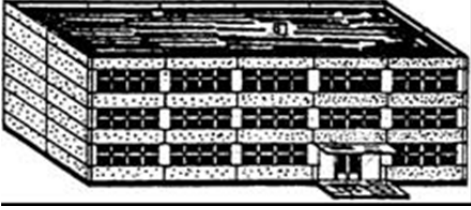




CNCN/A NK SLOPE FAILURE: The building site shall be sufficiently remote from potential earthquake induced slope failures or rock falls to be unaffected by such failures or shall be capable of accommodating any predicted movements without failure.

ANNEX A.I.III : CLASSIFICATION OF DAMAGE GRADES

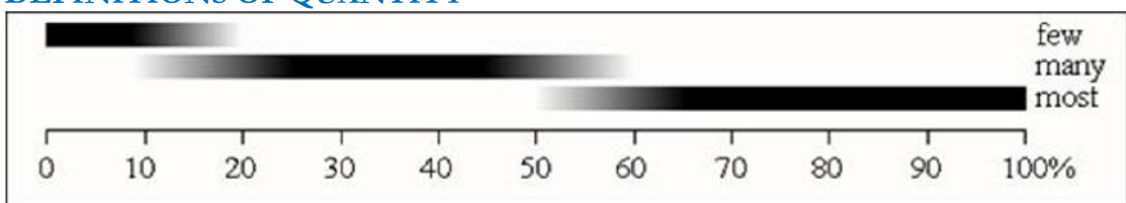
Classification of damage to masonry buildings

	<p>Grade 1: Negligible to slight damage (no structural damage, slight non-structural damage) 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) 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) 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) Serious failure of walls; partial structural failure of roofs and floors.</p>
	<p>Grade 5: Destruction (very heavy structural damage) 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) 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) 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) 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) 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) 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



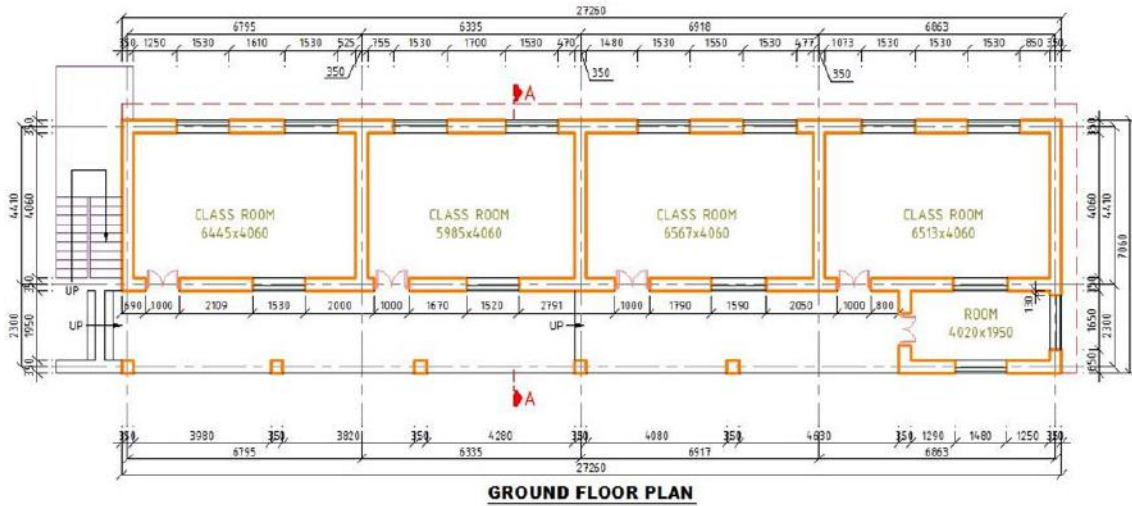
Exposed reinforcement



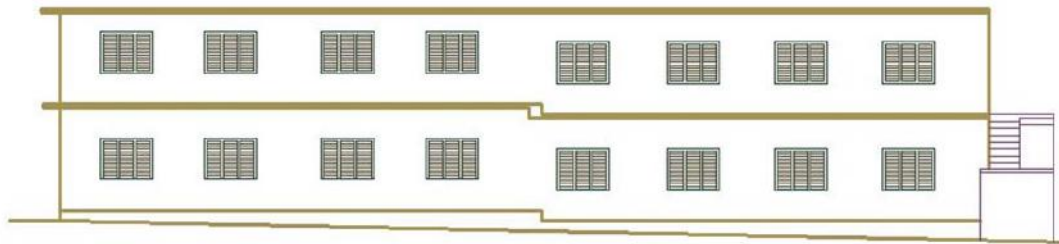
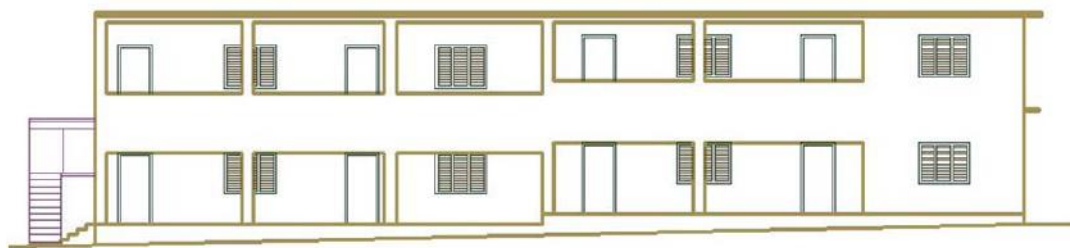
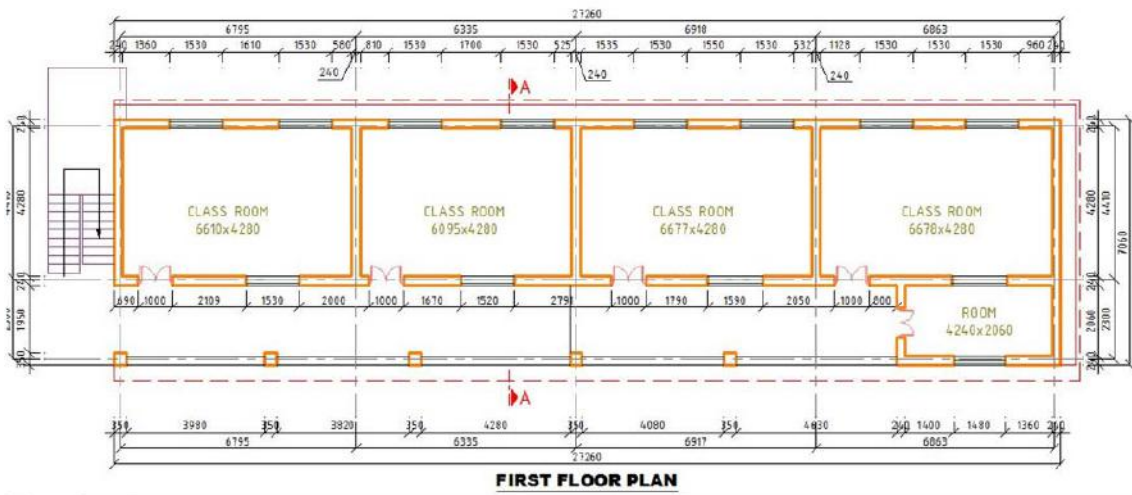
Cracks at slab & wall connection

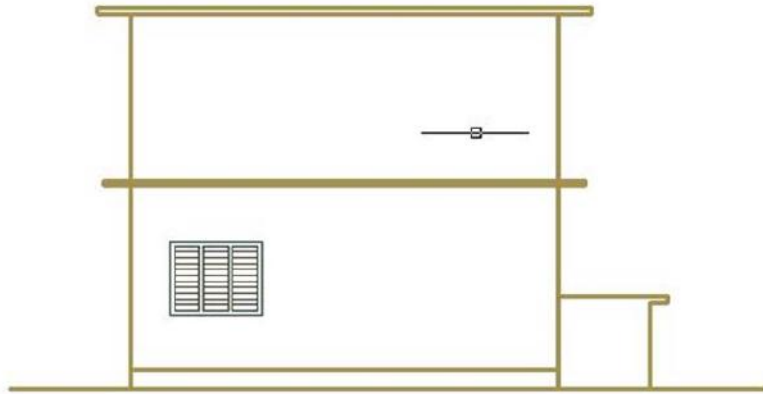
Figure A- 1: Different damage patterns in the buildings

A.1.2 BUILDING DRAWINGS

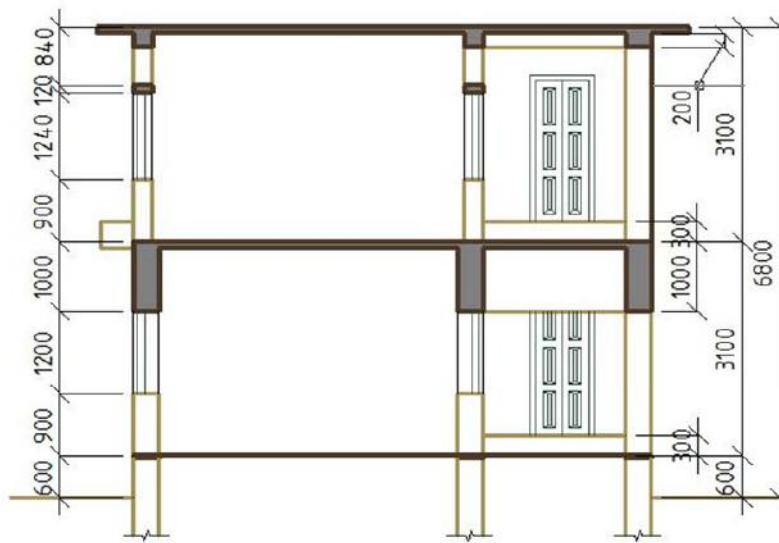


(Source Photos by: Hari. D. Shrestha Other than stated)

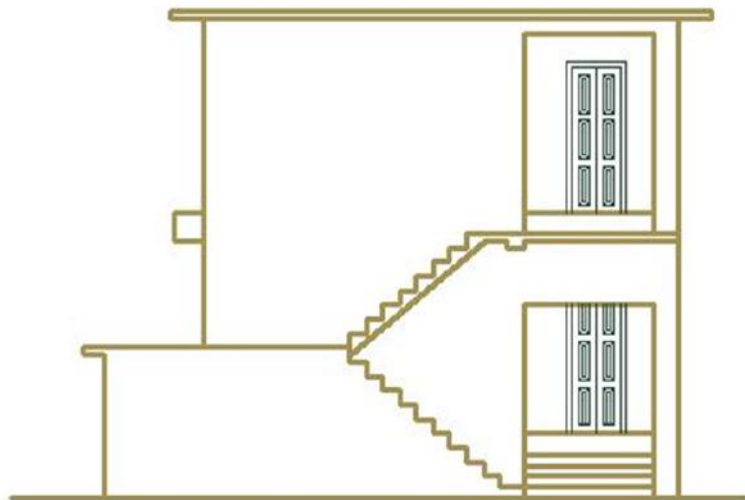




NORTH ELEVATION



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.

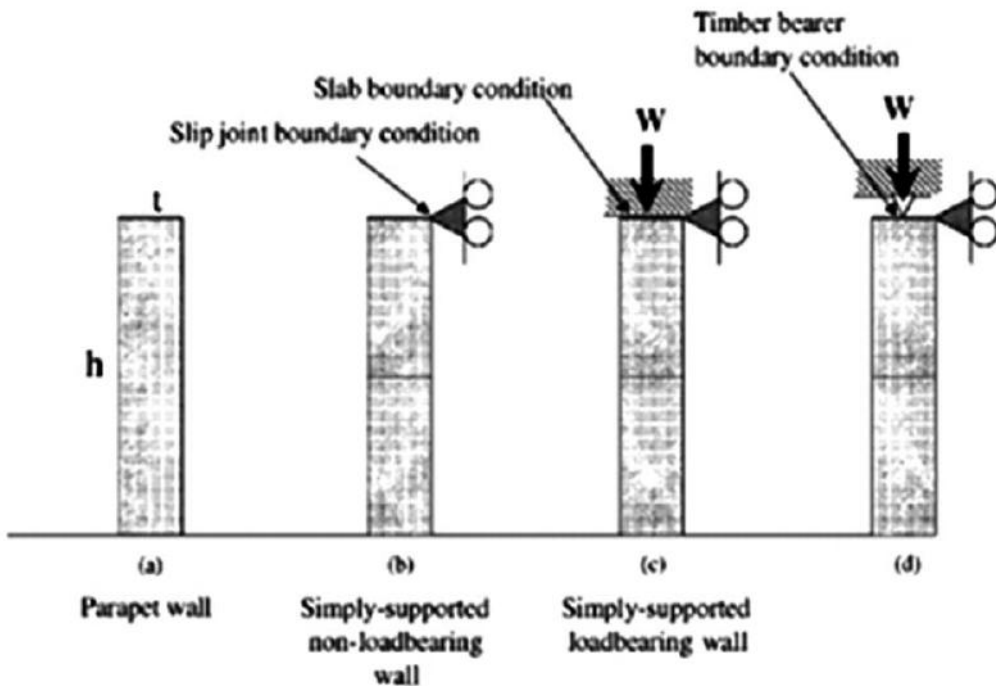


Figure A- 2: Unreinforced masonry wall support configurations (Doherty et al. [2002])

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.

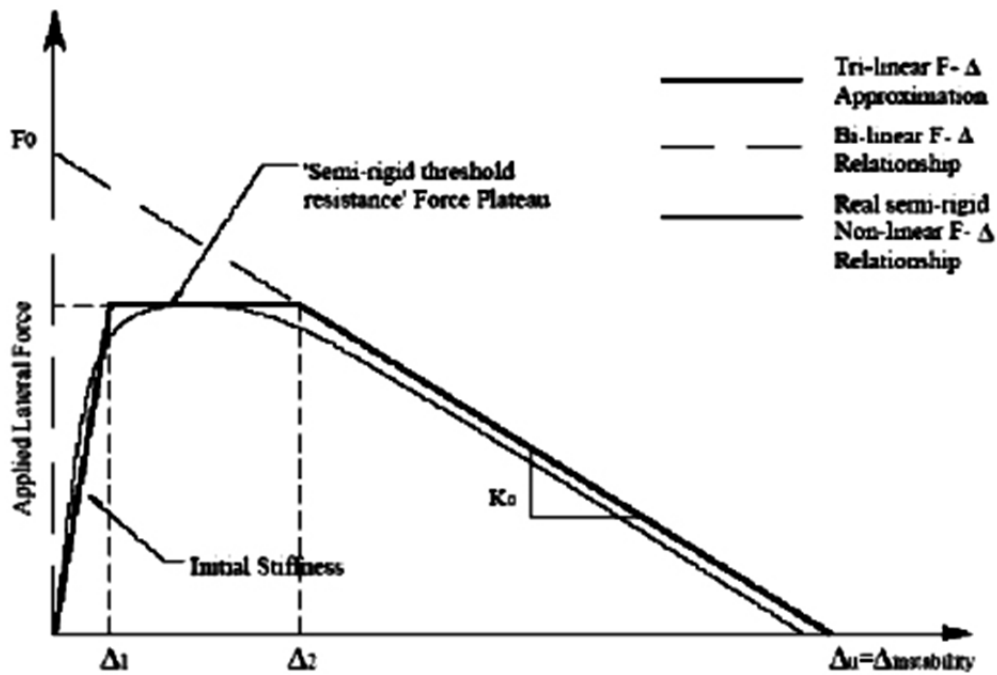


Figure A- 3: Tri-linear idealization of the static force- displacement relationship (Griffith et al. 2003)

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

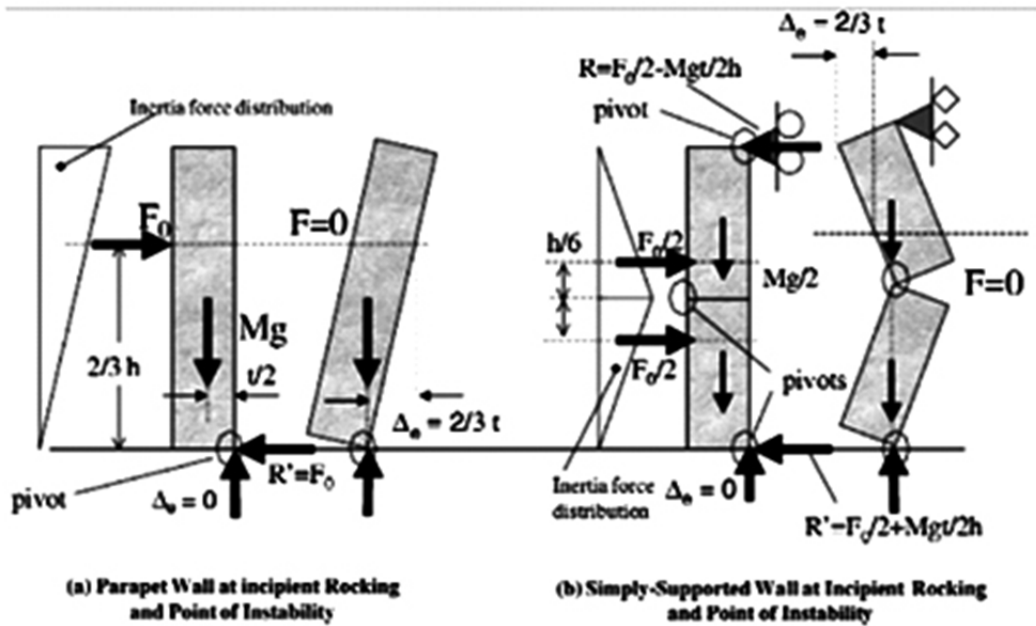


Figure A- 4: inertia forces and reactions on rigid URM walls (Doherty et.al 2002)

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	$\Delta = \varphi\Delta_u, \Delta_u = 0.8 t, \quad \varphi \approx 0.8 \sim 1.0$	$\approx 50\text{mm}$	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.

$$F = F_o \left(1 - \frac{\Delta_1}{\Delta_u} \right) \dots\dots\dots 0-2$$

$$K_{\Delta_1} = \frac{F}{\Delta_2} \dots\dots\dots 0-3$$

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 $\Delta_{i,LSi} = \phi \Delta_u$, 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_1) a value of 0.12 for Δ_1 / 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_1 (initial stiffness) as follows, Eq.(0-7):

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

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:

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

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

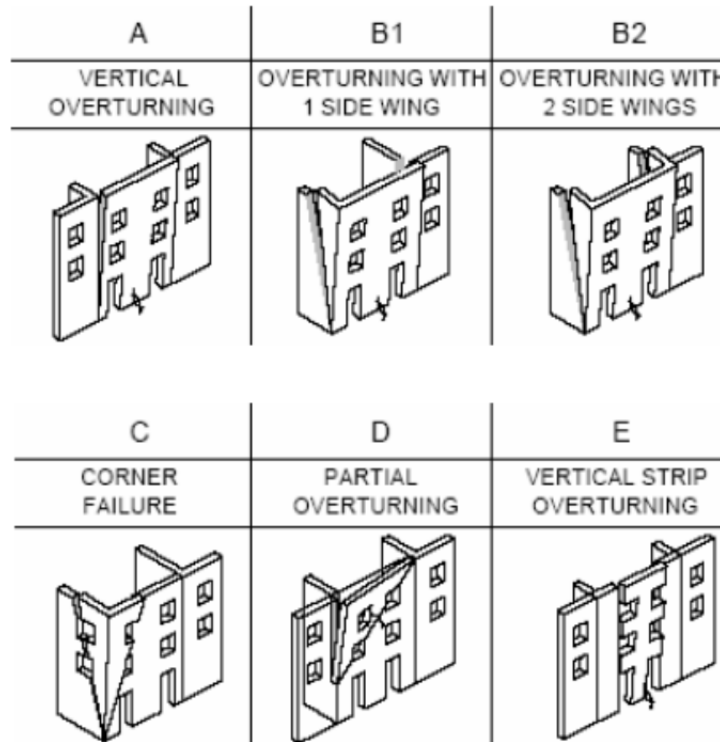


Figure A- 5: Collapse mechanism (D' analysis and Speranza 2003)

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

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

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

Mechanism A

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

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

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 B₂. 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 L₂ is equal to zero. r_{hs} is the number of courses within the storey height and n is the number of storeys.

Mechanism D

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

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].

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

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 ($\Delta_{L,S}$) 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.

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

$$\Delta_p = k_2 \cdot (\ddot{a}_{LS} - \ddot{a}_r) h_{sp} \dots\dots\dots 0-19$$

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

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

Figure A- 6).

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

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

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

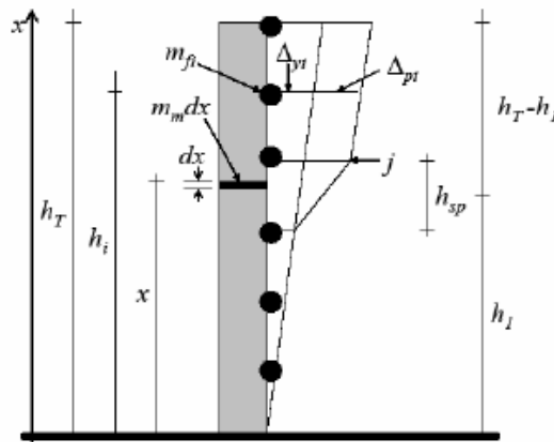


Figure A- 6: Simplified model for the definition of k_2 (Restrepo 2004)

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 ($m_{mr} = m_m \cdot h_T$).

$$\Delta_e = \frac{\ddot{a}_y^2 (h_T^2 \cdot m_{f1} + M_m)}{\ddot{a}_y (h_T \cdot m_{f1}) + N_m} \dots\dots\dots 0-24$$

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

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

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

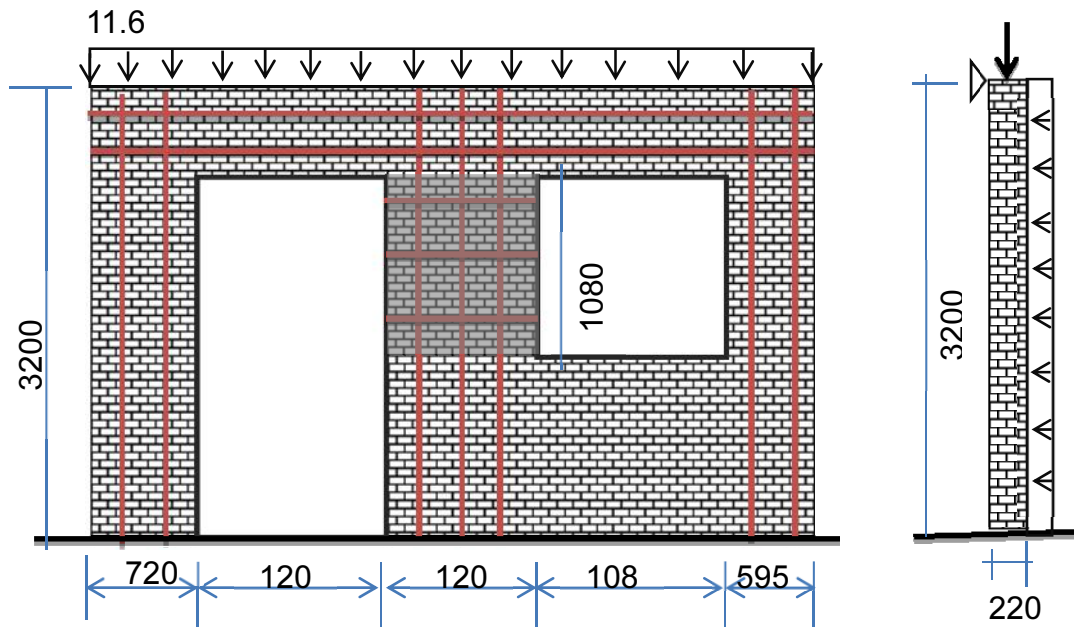
The limit state period of vibration of adobe walls is rewritten for convenience:

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

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.



$$l_w = 1.2 \text{ m} \quad q = 11.6 \text{ KN/m}$$

$$h_c = 3.2 \text{ m} \quad t_w = 0.22 \text{ m}$$

1 Establishing seismic demands

$$V^* = 44.3 \text{ KN} \quad \text{In-plane seismic force}$$

$$v^* = 3.8 \text{ KN/m}^2 \quad \text{Out-of-plane uniformly distributed seismic force}$$

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

$$\text{Masonry density} = 18 \text{ KN/m}^3$$

$$W_w = 15.2064 \text{ KN}$$

$$N_t + 0.5 W_w = 21.5232$$

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

$$f_y = 1104 \text{ Mpa}$$

$$A_h = 14.8 \text{ mm}^2 \quad 10 \text{ mmdia}$$

$$f_y A_s = 32678.4 \text{ N}$$

$$= 32.6784 \text{ KN}$$

$$d = 225 \text{ mm}$$

$$= 0.225 \text{ m}$$

$$a = 0.004966 \text{ m}$$

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

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

Thus, assumed 10 mmdia 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.

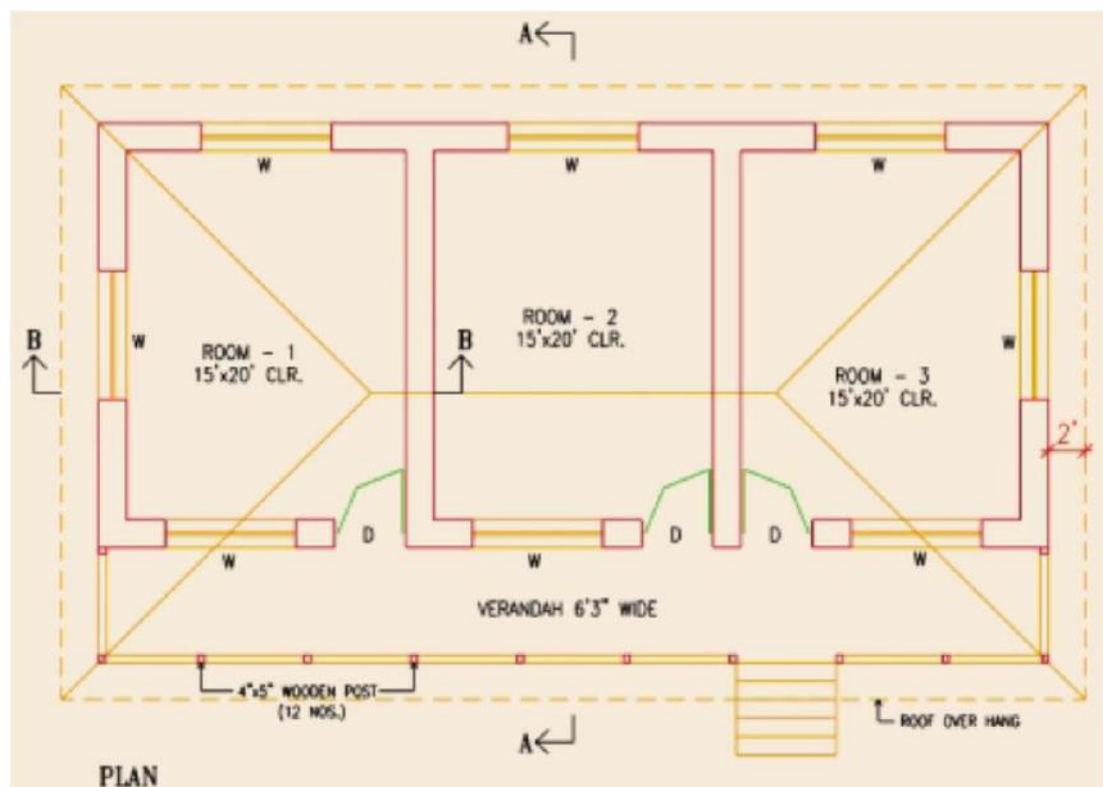
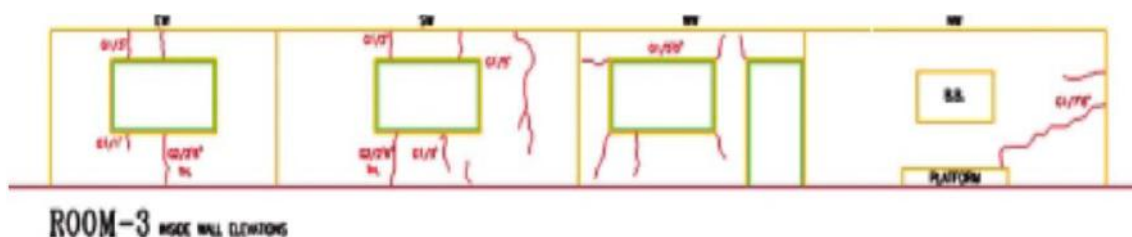
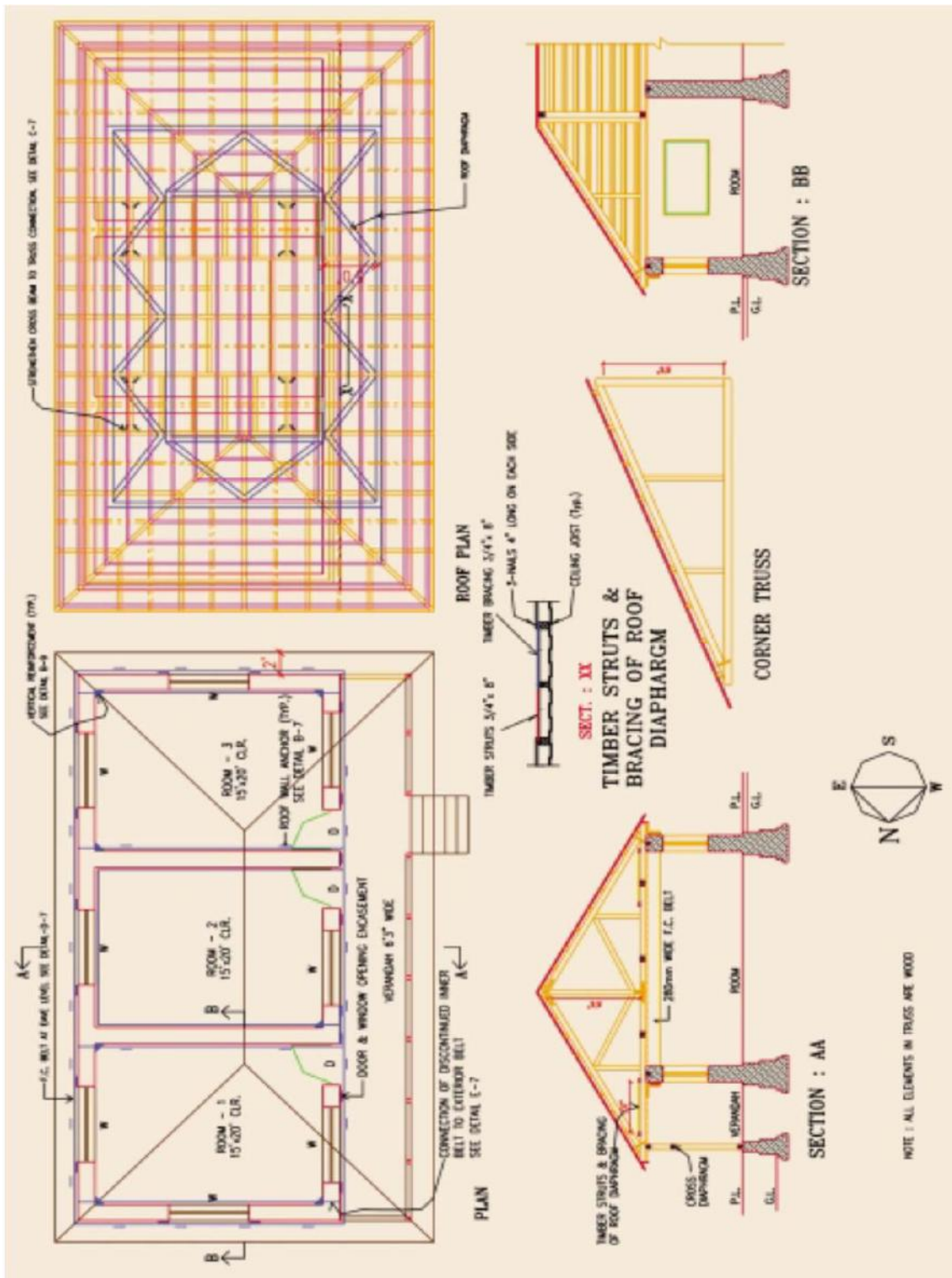
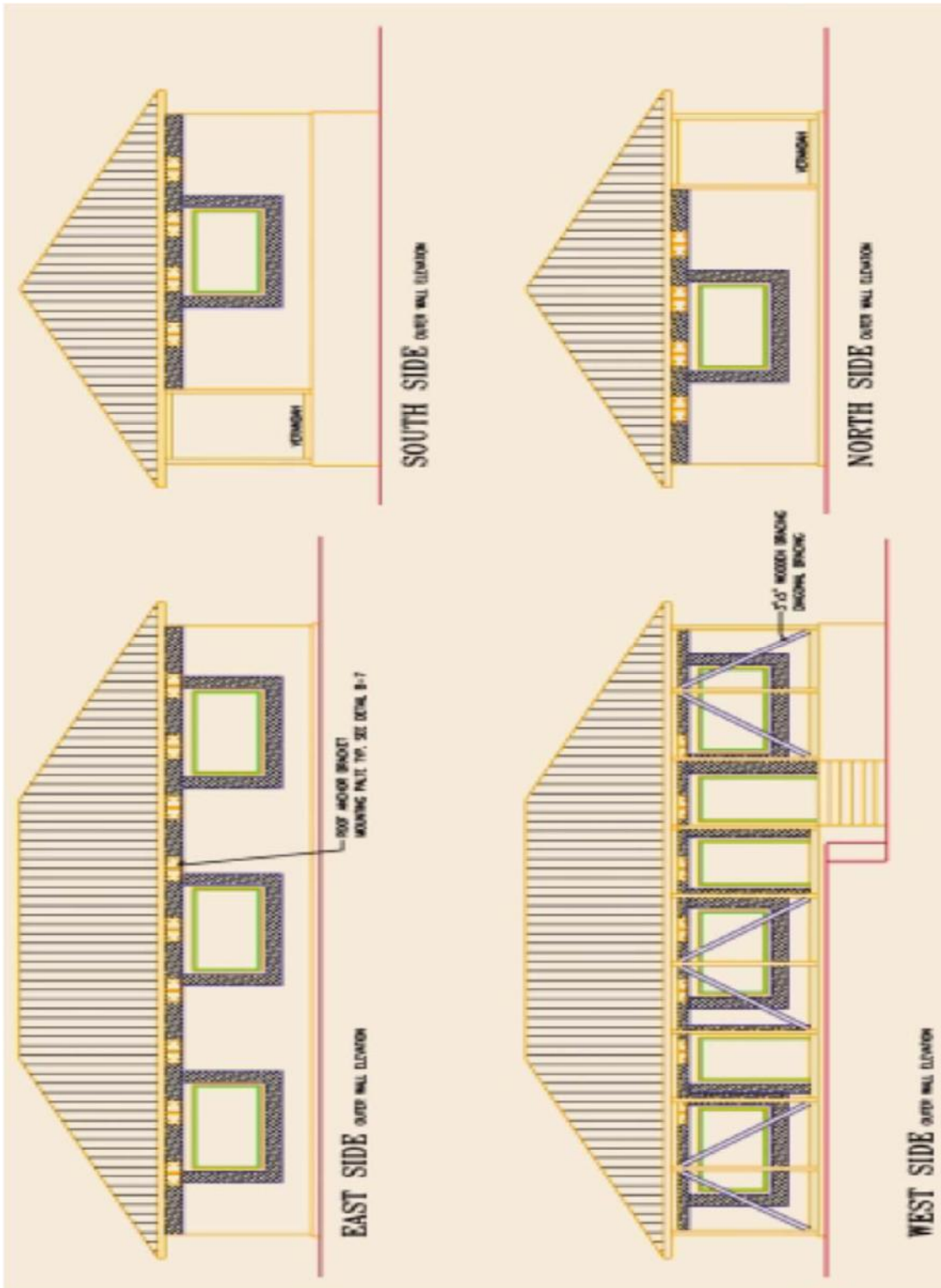


Figure A-7: Drawing for the restoration and retrofitting



(Source: UNDP, UNESCO, GOI, 2007)





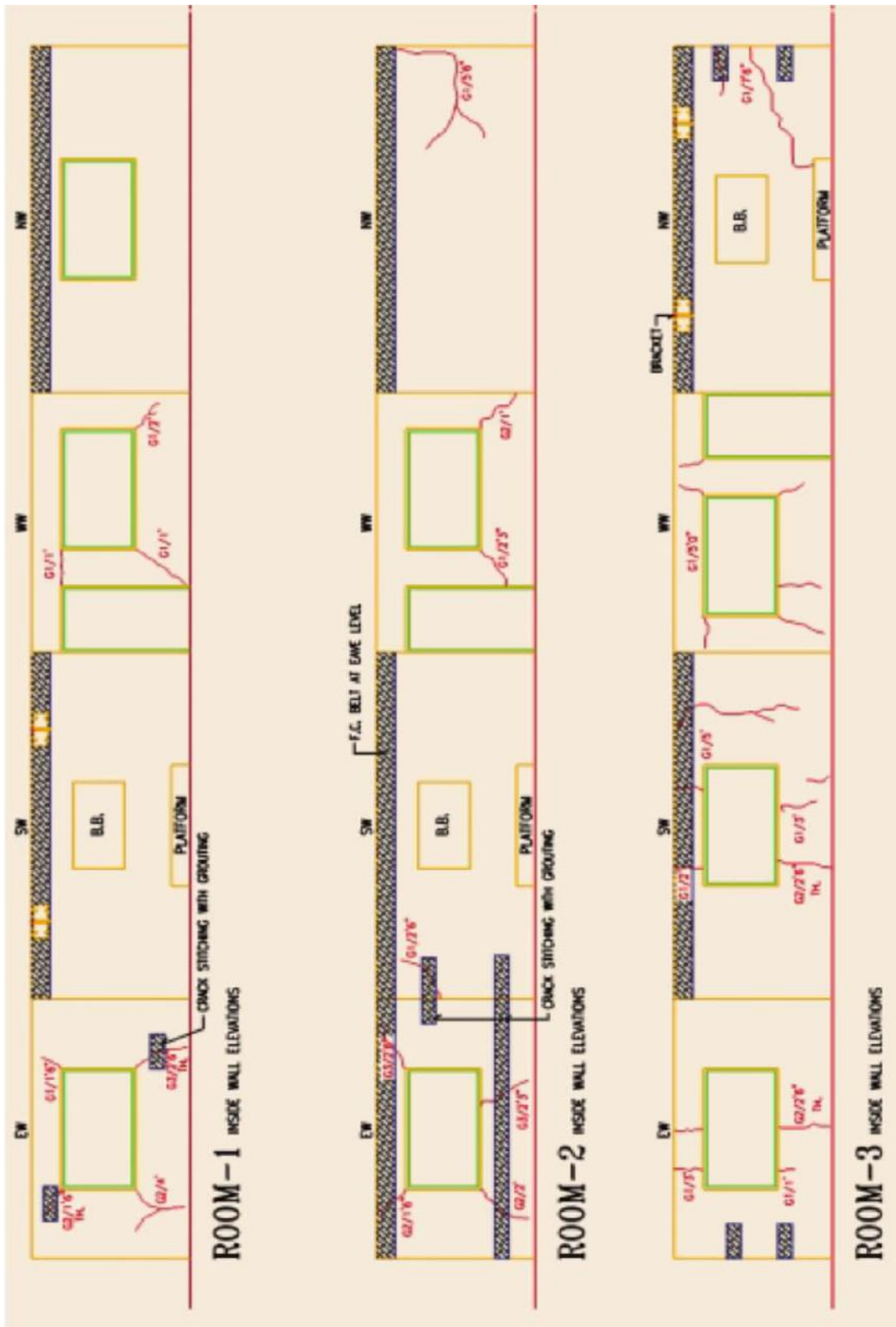




Figure A- 2: Existing 3 room school building before retrofitting



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 aroll



Figure A- 9: Tying encasement reinforcement to bear 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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SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL

ADOBE STRUCTURES



SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL

2016

MASONRY



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.

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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.

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SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL, 2016
MASONRY STRUCTURES

Government of Nepal

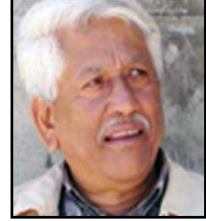
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MESSAGE



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.

Arjun Narasingha K.C.
Honorable Minister
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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

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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 academician 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 helpful in raising the safety awareness and making to community disaster resilient.

My sincere thanks goes 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.

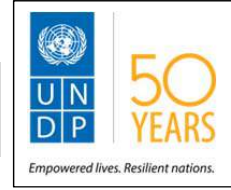
Also, my thanks go to United Nations Comprehensive Disaster Risk Management Program, CDRMP for the support during the preparation of the guideline and publication as well.

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 Illam 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 Julliard
UNDP Resident Representative &
United Nations Resident Coordinator

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LIST OF ACRONYM

Ab	Sum of net mortared area of bed joints
An	Area of net mortar/ grouted section
As	Area of shear reinforcement
C	Modification factor
Cm	Effective mass factor to account for higher mode participation
D	In plane length of masonry wall
Dd	Maximum in plane diaphragm displacement
dv	Wall length in the direction of shear force
Ese	Expected modulus of elasticity
f_{dt}	Lower bound masonry diagonal tension
f_m	Lower bound masonry compressive strength
f_a	Axial compressive stress due to gravity
F_p	Design force for anchorage of walls to diaphragms
FRC	Fiber reinforced composite
f_y	Lower bound yield strength of shear reinforcement
f_{ye}	Expected yield strength
H'	Least clear height of opening on either side
heff	Height to resultant of lateral force
k	Lateral stiffness
K	Knowledge factor
K_e	Elastic lateral stiffness
K_i	Elastic lateral stiffness
M	Moment of the masonry section
m	Modification factor to account for expected ductility of failure mode
NBC	National Building Code
PCE	Expected gravity compressive force applied
PD	Superimposed dead load at the top of the
PL	Lower bound axial compressive force due to
P_w	Weight of the wall
QCE	Expected strength of component
QCL	Lower bound strength component
QUD	Deformation controlled design action
QUF	Force-controlled design action
S	Spacing of shear reinforcement
S_a	Response spectrum acceleration
SXS	Spectral response acceleration for short period
t	Thickness of wall
T_e	Effective fundamental period
T_i	Elastic fundamental period calculated by elastic dynamic analysis
V	Shear on the masonry section
UNDP	United Nations Development Programme

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1. INTRODUCTION

1.1 BACKGROUND

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

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

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

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

1.3 OBJECTIVE AND SCOPE

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

1.4 CONCEPT OF REPAIR, RESTORATION AND RETROFITTING¹

1.4.1 REPAIR

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

- i. Patching up of defects such as cracks and fall of plaster.
- ii. Repairing doors, windows, replacement of glass panes.

¹ Adapted from LAEE Manual

- iii. Checking and repairing electric wiring.
- iv. Checking and repairing gas pipes, water pipes and plumbing services.
- v. Re-building non-structural walls, smoke chimneys, boundary walls, etc.
- vi. Re-plastering of walls as required.
- vii. Rearranging disturbed roofing tiles.
- viii. Relaying cracked flooring at ground level.
- ix. Redecoration, whitewashing, painting, etc.

Repair restores only the architectural damages but do not restore the original structural strength of cracked walls or columns. So a repaired building may be very illusive as it will hide all the weaknesses and the building will suffer even more severe damage if shaken again by an equal shock since the original energy absorbing capacity will not be available.

1.4.2 RESTORATION

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

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

- i. Removal of portions of cracked masonry walls and piers and rebuilding them in richer mortar. Use of non-shrinking mortar will be preferable.
- ii. Addition of reinforcing mesh on both -faces of the cracked wall, holding it to the wall through spikes or bolts and then covering it suitably. Several alternatives have been used.
- iii. Injecting epoxy like material, which is strong in tension, into the cracks in walls, columns, beams, etc.

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

1.4.3 SEISMIC STRENGTHENING (RETROFITTING)

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

2. DAMAGE PATTERNS

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

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

2.1 CATEGORIZATION OF DAMAGE / VULNERABILITY

Table 2-1 :Damage Categories²

S.No.	Categories	Wall	Floor / Roof
1	No Damage	No Damage	No Damage
2	Slight Non-Structural Damage	Thin cracks in plaster, falling of plaster bits in limited parts	Thin cracks in small areas, tiles only slightly disturbed
3	Slight Structural Damage	Small cracks in walls, falling of plaster in large areas: damage to non-structural parts like chhajjas, parapets	Small cracks in slabs / A.C sheets; tiles disturbed in about 10% area: minor damage in under-structure of sloping roof
4	Moderate Structural Damage	Large and deep cracks in walls; widespread cracking of walls, columns and pier; or collapse of one wall. The load carrying capacity of structure is partially reduced	Large cracks in slabs; some A.C sheets, broken; upto 25% tiles disturbed / fallen moderate damage to understructure of sloping roofs
5	Severe Structural Damage	Gaps occur in walls; two or more inner or outer walls collapse; Approximately 50% of the main structural elements fail. The building takes a dangerous state	Floor badly cracked, part may fall; under-structure of sloping roof heavily damaged, part may fall; tiles badly affected & fallen
6	Collapse	A large part or whole of the building collapses	A large part or whole floor and roof collapses or hang precariously

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

2.2 DAMAGE TYPOLOGIES³

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

2.2.1 SHEAR/ DIAGONAL CRACKS

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

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

In-plane shear cracking, damage at wall and tie-rod anchorages, and horizontal cracks are relatively low-risk damage types. However, while in-plane shear is not considered hazardous from the perspective of life safety, it is often costly in terms of loss to historic fabric. In-plane shear cracks often cause severe damage to plasters and stuccos that may be of historic importance, such as those decorated with murals.

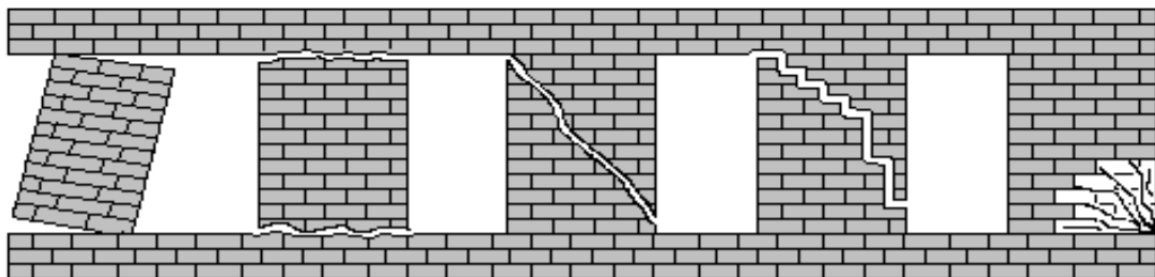
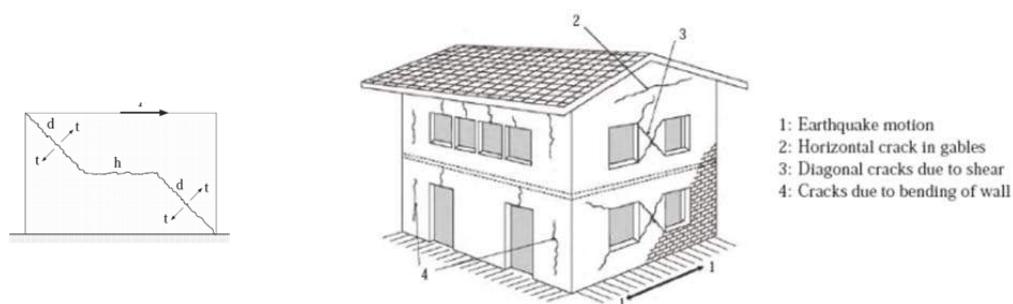













Figure 2-1 Different crack patterns of un-reinforced masonry pier
(Source Yi, 2004)



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




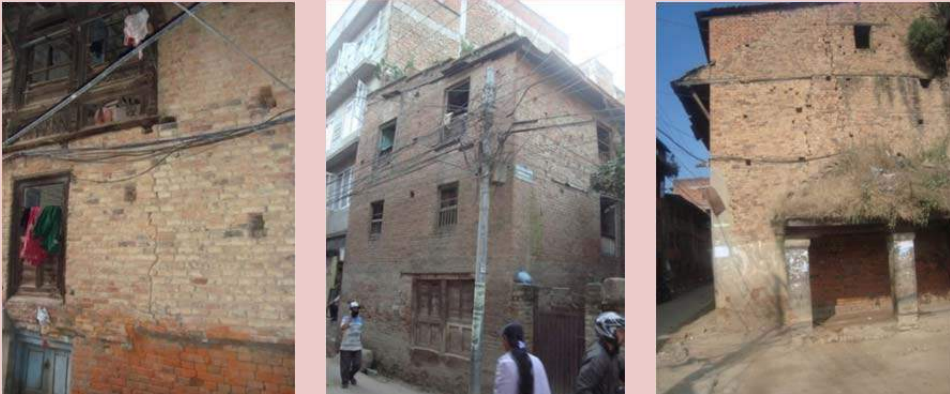
Cracks and Damages	Remark
<div style="display: flex; justify-content: space-around;">   </div> <p style="text-align: center;"><i>Damage at corners of openings, Indonesia</i></p>	
<div style="display: flex; justify-content: space-around;">   </div> <p style="text-align: center;"><i>Heavily damaged single storey rubble masonry wall with concrete roof in Manukawa & Sukhpur, India Walls survived due to diaphragm action from roof. Cantilever beams embedded in walls also helped this.</i></p> <p><i>Note: Window openings are also not close to corners.</i></p>	<p><i>Ref: Repair and strengthening guide for earthquake damaged low rise domestic buildings in Gujarat, India</i></p>
<div style="display: flex; justify-content: space-around;">   </div> <ol style="list-style-type: none"> <i>1. Large window openings close to corners and short column failures</i> <i>2. Diagonal cracking at corner column caused by twisting of frame and short column failure.</i> 	<p>The picture also shows cracks at corner along with diagonal cracks.</p> <p><i>Ref: Repair and strengthening guide for earthquake damaged low rise domestic buildings in Gujarat, India</i></p>
<div style="text-align: center;">  </div> <p style="text-align: center;"><i>Infill panels to an reinforced concrete frame building acting as non-structural shear walls, provided stability to the overall frame – Bharasar, Gujrat</i></p>	<p><i>Ref: Repair and strengthening guide for earthquake damaged low rise domestic buildings in Gujarat, India</i></p>


Cracks and Damages	Remark
 <p data-bbox="196 533 1066 607"><i>Uri town – Civil Defense Bldg.– Diagonal cracks in window openings and tilting of walls in Burnt Brick Cement Masonry wall, Kashmir, India.</i></p>	<p data-bbox="1094 271 1390 443"><i>Ref: Repair, Restoration and retrofitting of Masonry Buildings in EQ affected Areas of Jammu & Kashmir</i></p>
 <p data-bbox="357 958 1070 1025"><i>Shear crack in a building during Haiti earthquake Photo: Er. Hari Darshan Shrestha</i></p>	
 <p data-bbox="515 1368 1070 1435"><i>Pakistan earthquake Photo: Er. Hari Darshan Shrestha</i></p>	
 <p data-bbox="300 1794 1070 1856"><i>Diagonal crack seen in a local building in Kathmandu, Nepal Photo: Er. Subin Desai</i></p>	

2.2.2 VERTICAL CRACKS

Vertical cracks often develop at corners during the interaction of perpendicular walls and are caused by flexure and tension due to out-of-plane movements. This type of damage can be



particularly severe when vertical cracks occur on both faces, allowing collapse of the wall section at the corner.

Cracks and Damages			Remark
	<p>PADANG , INDONESIA</p> <p><i>Photo : Er. Hari Darshan Shrestha</i></p>		
	<p><i>Vertical crack in a building during Haiti earthquake</i></p> <p><i>Photo : Er. Hari Darshan Shrestha</i></p>		
	<p><i>Vertical crack in a building during an earthquake, Pakistan</i></p> <p><i>Photo : Er. Hari Darshan Shrestha</i></p>		
	<p><i>Vertical cracks in old buildings in Kathmandu, Nepal</i></p> <p><i>Photo : Er. Hari Darshan Shrestha</i></p>		

Cracks and Damages	Remark
 <p data-bbox="309 562 1034 595"><i>Side wall of a building damaged during Gorkha Earthquake, 2001</i></p>	

2.2.3 CRACKS AT CORNERS

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

Damages and Cracks	Remark
 <p data-bbox="309 1182 991 1216"><i>Modern cut-stone masonry building in Mirzapur, Gujrat, India</i></p>	<p data-bbox="1129 927 1401 1099"><i>Ref: 2. Repair and strengthening guide for earthquake damaged lowrisedomestic buildings in Gujarat, India</i></p>
 <p data-bbox="209 1554 1091 1621"><i>Uri town – Civil Defence Bldg.– Corner cracks and vertical cracks in Burnt Brick Cement Masonry wall, India</i></p>	<p data-bbox="1129 1261 1401 1433"><i>Ref: Repair, Restoration and retrofitting of Masonry Buildings in EQ affected Areas of Jammu & Kashmir</i></p>



*Cracks seen at corners in old buildings in Lalitpur, Nepal
Photo: Ar. Anjali Manandbar*

2.2.4 OUT OF PLANE / BULGING

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

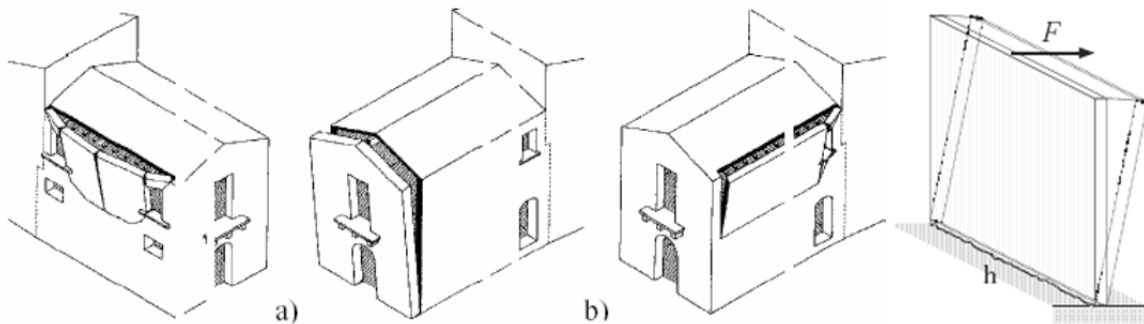











Figure 2-2 In-plane stiffness of wooden floor


Damages and Cracks		Remark
 <p><i>1994 Lina Earthquake , Indonesia</i></p>	 <p><i>2006 Central Java Earthquake, Indonesia</i></p>	
 <p><i>Damage to stone masonry during Northern Pakistan Earthquake (Photo: Er. Hari Darshan Shrestha)</i></p>		
 <p><i>Partially collapsed buildings showing out of plane defect during Kashmir earthquake, India</i></p>		<p><i>Ref: Repair, Restoration and retrofitting of Masonry Buildings in EQ affected Areas of Jammu & Kashmir</i></p>
 <p><i>Partial collapse of gable wall for a single storey random masonry wall in Kera, Gujrat, India</i></p>		<p><i>Ref: 2. Repair and strengthening guide for earthquake damaged low rise domestic buildings in Gujarat, India</i></p>

Damages and Cracks	Remark
  <p data-bbox="416 853 1142 916" style="text-align: center;"><i>Out of plane damage during Haiti earthquake (Photo: Hari Darsban Shrestha)</i></p>	<p data-bbox="1166 266 1406 338"><i>Ref: In-plane stiffness of wooden floor</i></p>
 <p data-bbox="416 1314 1142 1377" style="text-align: center;"><i>Out of plane damage during Haiti earthquake (Photo: Hari Darsban Shrestha)</i></p>	
 <p data-bbox="948 1946 1142 1975" style="text-align: right;"><i>(Photo: Subin Desai)</i></p>	

Damages and Cracks	Remark
 <p data-bbox="421 629 911 667"><i>Damages during Gorkha Earthquake, 2001</i></p>	

2.2.5 BED JOINT SLIDING

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

Damages and Cracks	Remark
 <p data-bbox="379 1823 1018 1899"><i>An example of bed joint can be seen in following pictures of buildings devastated after Haiti earthquake.</i></p> <p data-bbox="868 1899 1203 1921"><i>(Photo: Er. Hari Darshan Shrestha)</i></p>	

2.2.6 TOE CRUSHING

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

Damages and Cracks	REMARK
 <p data-bbox="365 815 983 887"><i>Toe crushing defect can be seen in the picture beside, taken after Haiti earthquake.</i></p> <p data-bbox="820 889 1155 913"><i>(Photo: Er. Hari Darshan Shrestha)</i></p>	
 <p data-bbox="304 1825 1042 1897"><i>After an earthquake in Pakistan, a building can be seen with cracks and major defects at the toe.</i></p> <p data-bbox="820 1899 1155 1924"><i>(Photo: Er. Hari Darshan Shrestha)</i></p>	

3. VULNERABILITY ASSESSMENT OF EXISTING BUILDINGS

Vulnerability assessment of buildings can be performed according to methodology prescribed in “SEISMIC VULNERABILITY EVALUATION GUIDELINE FOR PRIVATE AND PUBLIC BUILDINGS” developed by Ministry of Urban Development and Building Construction under Earthquake Risk Reduction and Recovery Preparedness Program for Nepal. For detailed Evaluation please refer Chapter 4 (Retrofitting Criteria) following this chapter.

4. RETROFITTING CRITERIA

4.1 BUILDING SYSTEM AND GENERAL REQUIREMENTS

Masonry structures gain stability from the support offered by cross walls, floors, roof and other elements such as piers and buttresses. Load bearing walls are structurally more efficient when the load is uniformly distributed and the structure is so planned that eccentricity of loading on the members is as small as possible. Avoidance of eccentric loading by providing adequate bearing of floor/roof on the walls providing adequate stiffness in slabs and avoiding fixity at the supports, etc, is especially important in load bearing walls in multistory structures⁴. These matters should receive careful consideration during the planning stage of retrofitting of masonry structures.

4.1.1 BUILDING LIMITATION

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

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

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

4.1.2 SEISMIC ZONES

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

Three seismic zones as recommended in NBC 109

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

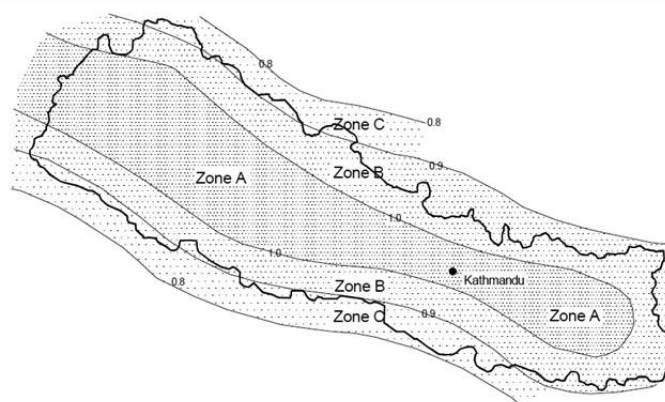


Figure 4-1 Seismic zones for masonry structures (NBC 109)

⁴ IS 1905

4.1.3 FLOOR SYSTEM

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

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

4.1.4 LOAD PATH

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

- a. Lack of Redundancy
- b. Vertical Irregularities
- c. Plan Irregularities

a. *Lack of redundancy*

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

b. *Vertical irregularities*

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

i) **Weak storey**

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

ii) **Soft storey**

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

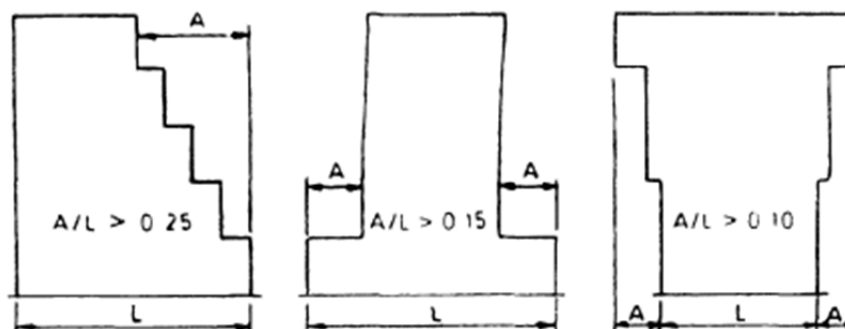


Figure 4-2 Vertical irregularities limit up to which treatment at discontinuity point or at element level is possible.

c. Plan irregularities

Plan irregularities that create torsion should be eliminated with the addition of lateral-force-resisting bracing elements that will support all major diaphragm segments in a balanced manner. For the irregularities limit specified in Fig. 4-3, it is possible to allow the irregularity to remain by strengthening those structural components that are overstressed such as the re-entrant corners.

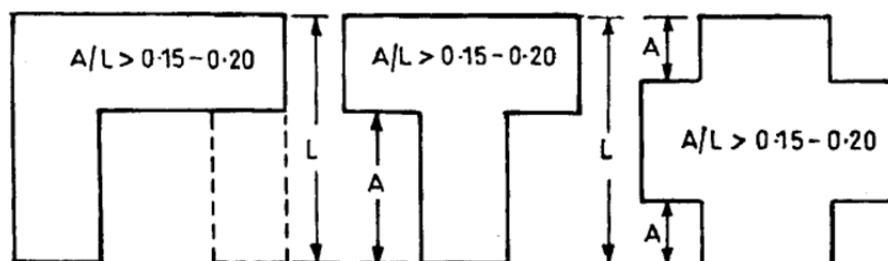


Figure 4-3 Plan irregularity limits that require strengthening of overstressed elements and re-entrant corners

For buildings which exceed irregularity limits specified in Fig. 4-4, it is recommended that those buildings are either strengthened by adding additional lateral load resisting elements or by separating the different parts of the building (as shown in Fig. 4-5) as separate system.

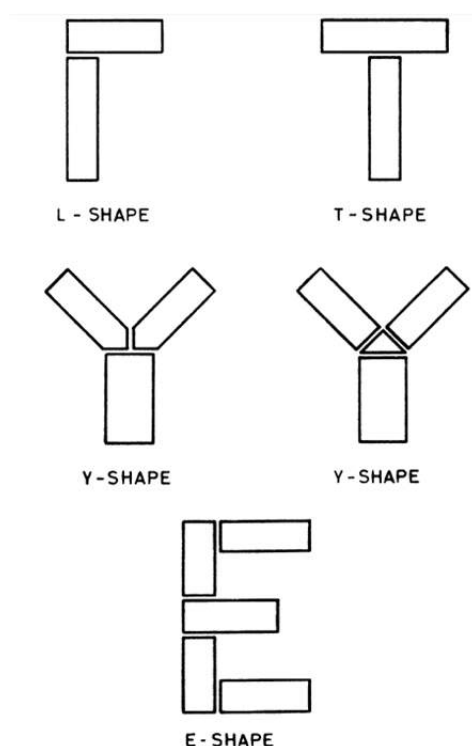
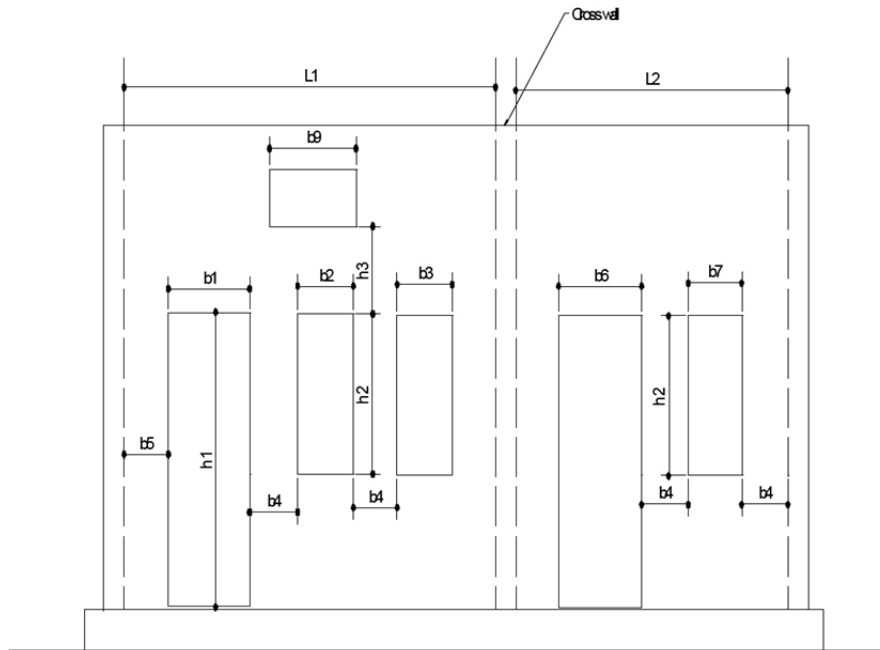


Figure 4-4 Recommended shapes for buildings with irregular plans

4.1.5 OPENINGS

4.1.5.1 Openings in walls

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



Note:

- i) $b_1+b_2+b_3$ $<0.5 L_1$ for one storey, $<0.42 L_1$ for two-storeyed, $<0.33 L_1$ for three-storeyed
- ii) b_6+b_7 $<0.5 L_2$ for one storey, $<0.42 L_1$ for two-storeyed, $<0.33 L_1$ for three-storeyed
- iii) b_4 $> 0.5 h$, but not less than 600 mm
- iv) b_5 $>0.25 h$, but not less than 600 mm
- v) b_3 >600 mm and >0.5 (bigger or b_2 and b_9)

Figure 4-5 Recommended sizes of the openings

4.1.5.2 Opening in diaphragms

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

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

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

4.1.6 HEIGHT TO THICKNESS RATIO

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

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

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

4.1.7 HEIGHT OF THE WALL

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

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

4.1.8 CROSS WALLS

Cross-walls provision (see Annex)

5. ANALYTICAL PROCESS

5.1 ANALYTICAL METHODS

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

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

5.2 CHOICE OF METHOD

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

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

5.2.1 Linear analysis:

This method is applicable, if following conditions are met:

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

5.2.2 Non-linear analysis:

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

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

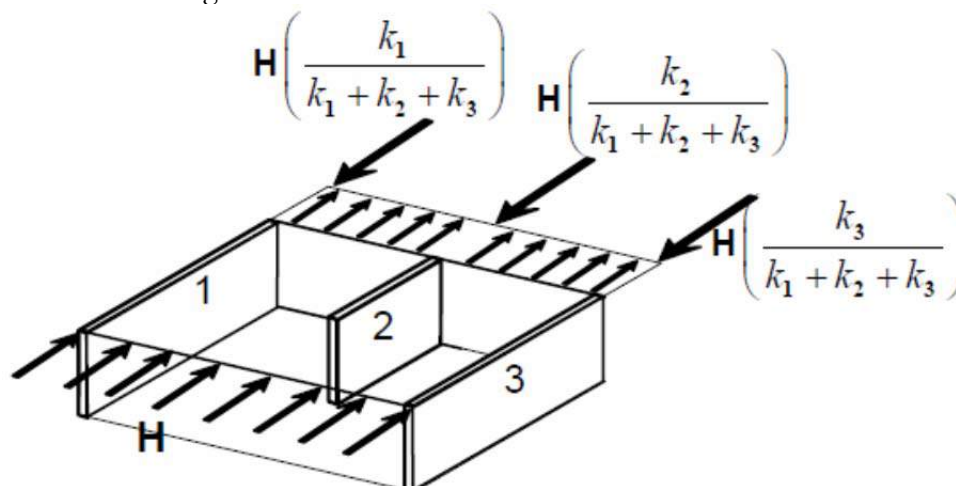
5.3 HORIZONTAL FORCES

The base shear shall be calculated and distributed at floor level according to NBC 105. For the purpose of obtaining stiffness of individual elements, the process described in Annex 1 can be used.

The horizontal seismic forces as obtained according to NBC 105 shall be distributed to each wall parallel to the direction of the force according to the following criteria:

5.3.1 FOR RIGID DIAPHRAGM

The distribution to vertical elements will be in proportion to the relative stiffness with respect to each other as shown in Figure 5-1.



k_1 , k_2 and k_3 are lateral stiffness of walls 1, 2 and 3 respectively.

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

5.3.2 FOR FLEXIBLE DIAPHRAGM

The distribution to vertical elements will be in proportion to the relative stiffness with respect to the contributing area (tributary area) basis as shown in Figure 5-2 below.

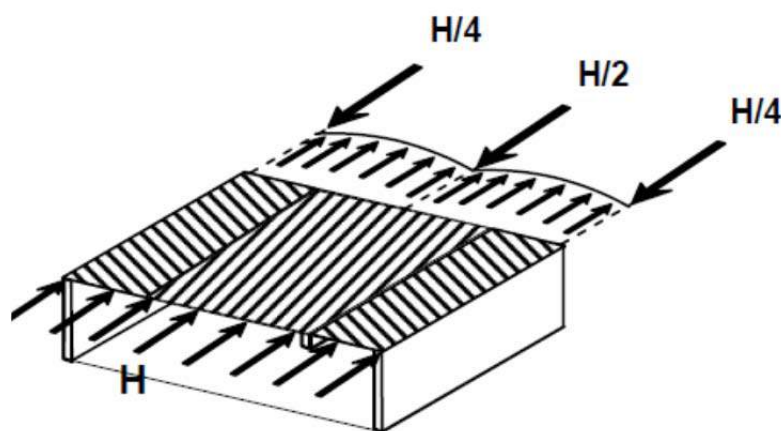


Figure 5-2 Proportional distribution of horizontal storey level force to individual walls for flexible diaphragm at floor level

5.4 CHECKS

5.4.1 CHECK FOR SHEAR (IN-PLANE LOADING)

Shear wall strength

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

$$V_a = v_a D t$$

where:

D = In plane length of masonry wall (mm)

t = thickness of wall (mm)

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

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

where:

v_{te} = Average bed-joint shear strength (MPa)

determined from in-place shear test and not to exceed 0.6 MPa

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

A_n = Area of net mortared/grouted section (mm²)

or,

$$v_a = 0.1 + \left(\frac{1}{6} \right) \times \left(\frac{P_{CE}}{A_n} \right)$$

to a maximum of 0.5 N/mm² when in place shear test data is not available

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

For walls without openings:

$$V_r = (0.50P_D + 0.25P_w) \frac{D}{H}$$

For wall with openings:

$$V_r = 0.5P_D \left(\frac{D}{H} \right)$$

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

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

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

$$V_{wx} < \Sigma V_r$$

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

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

$$V_p < V_a$$

$$V_p < V_r$$

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

5.4.2 CHECK FOR DIAPHRAGM DISPLACEMENT

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

Permissible deflection of diaphragm shall be the deflection which will permit the attached element to maintain its structural integrity under the individual loading and continue to support the prescribed loads without endangering the occupants of the building. The deflection can be checked using the approach described in Annex 2.

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

5.4.3 ROCKING STRENGTH:

The maximum horizontal shear which can be resisted by a rocking pier failing under static-in-plane is given by equation 4-1 and 4-2⁵.

$$V_r = (0.50P_D + 0.25P_w) \frac{D}{H'} \dots\dots\dots(4-1)$$

For walls with openings

$$V_r = 0.5P_D \frac{D}{H'} \dots\dots\dots(4-2)$$

Where,

D is the pier width

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

P_w is weight of wall

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

The detailed calculation is shown in Annex Section 7.1.

⁵ *Seismic Evaluation and Strengthening of Existing Buildings, Dr. Durgesh C. Rai, IIT Kanpur*

6 ANALYSIS METHODS

6.1 INTRODUCTION

FEMA 356 specifies four procedures that can be used to analyze an existing building. These are:⁶

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

6.1.1 ELASTIC ANALYSES

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

6.1.2 LINEAR STATIC PROCEDURE (LSP)

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

$$T = 0.0254(0.078\Delta_d)^{0.5} \dots\dots\dots(7 - 13)$$

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

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

$$V = C_1 C_2 C_3 C_m S_a W \dots\dots\dots(7 - 14)$$

Where:

- V = Pseudo lateral load
- C_1 = Modification factor relating expected inelastic displacements to the calculated elastic response.
- C_2 = Modification factor for stiffness degradation and strength deterioration (1.0 for LSP)
- C_3 = Modification factor to account for increased displacements due to P-Delta effects
- C_m = Effective mass factor to account for higher mode mass participation (1.0 for URM)
- S_a = Response spectrum acceleration at fundamental period and damping ratio of building

(estimated at 5%)

W = Effective weight of the building

For URM buildings with flexible diaphragms and a fundamental period estimated from equation 7-13, the pseudo-lateral load is calculated for each span of the building and for each floor. It is

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

then distributed to the vertical seismic-resisting elements (walls) according to tributary area. Forces in the diaphragm can then be calculated using these results.

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

$$KQ_{CL} \geq Q_{UF} \dots\dots\dots(7 - 15)$$

Where:

- K = Knowledge factor
- Q_{CL} = Lower-bound strength of component
- Q_{UF} = Force-controlled design action

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

Table 6.1 Knowledge factor according to acquired data (FEMA 356-2000)

Date	Level of knowledge							
	Minimum		Usual				Comprehensive	
Rehabilitation Objective	BSO or Lower		BSO or Lower		Enhanced		Enhanced	
Analysis Procedures	LSP/LDP		All		All		All	
Testing	No Tests		Usual Testing		Usual Testing		Comprehensive Testing	
Drawing	Design Drawings	Or Equivalent	Design Drawings	Or Equivalent	Design Drawings	Or Equivalent	Construction Documents	or Equivalent
Condition Assessment	Visual	Comprehensive	Visual	Comprehensive	Visual	Comprehensive	Visual	Comprehensive
Material Properties	From Drawings or Default Values	From Default Values	From Drawing and Tests	From Usual Tests	From Drawings and Tests	From Usual Tests	From Documents and Tests	From Comprehensive Test
Knowledge Factor (K)	0.75	0.75	1.00	1.00	0.75	0.75	1.00	1.00

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

$$mKQ_{CE} \geq Q_{UD} \dots\dots\dots(7 - 16)$$

Where:

- m = Modification factor to account for expected ductility of failure mode
- Q_{CE} = Expected strength of component
- Q_{UD} = Deformation-controlled design action

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

Table 6.2 *m*-factor for URM walls (FEMA 356- 2000)

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

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

6.1.3 LINEAR DYNAMIC PROCEDURE (LDP)

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

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

6.1.4 INELASTIC ANALYSES

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

6.1.5 NONLINEAR STATIC PROCEDURE (NSP)

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

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

The next step for the NSP is to generate nonlinear force-deformation relationships for each of the pier elements. A generalized force-deformation relationship is given in the standard and can be seen in

Figure 6- 1. These relationships are then used to develop a global force-displacement relationship for the building. An idealized bilinear curve is then fit over the actual building curve with the slope of the first section equal to an effective lateral stiffness, which is taken as the secant stiffness at 60% of the effective yield strength of the structure. This portion lasts until the effective yield strength of the building is reached. The second line has a slope of is a fraction of the effective lateral stiffness. This line ends when a target displacement is reached (FEMA 356-2000).

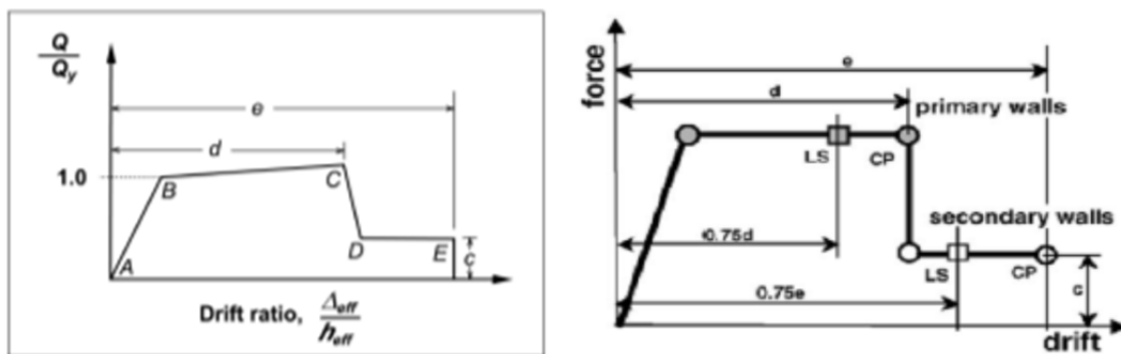


Figure 6- 1: Generalized Force-Deformation Relationship for Deformation Controlled Masonry Elements or Components (FEMA 356-2000)

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

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

Where:

- T_i = Elastic fundamental period calculated by elastic dynamic analysis
- K_i = Elastic lateral stiffness
- K_e = Effective lateral stiffness

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

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

Where:

- C_o = Factor to relate spectral displacement of equivalent SDOF system to the control node of the actual MDOF building
- C_1, C_2, C_3 = Same factors as LSP
- T_e = Effective fundamental period
- S_a = Response spectrum acceleration
- g = Acceleration of gravity

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

6.1.6 APPROACH FOR NON-LINEAR ANALYSIS

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

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

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

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

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

Table 6.3 Acceptance criteria for nonlinear static analysis of masonry (FEMA356)

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

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

Table 6.4 Structural performance level for unreinforced masonry building as define in FEMA356

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

6.1.7 CALCULATION OF STIFFNESS OF MASONRY WALLS⁷

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

The masonry assemblage of units, mortar, and grout shall be considered to be a homogeneous medium for stiffness computations with an expected elastic modulus in compression as given in the table below⁸.

Property	Masonry Condition ¹		
	Good	Fair	Poor
Compressive Strength (f_m^c)	900 psi	600 psi	300 psi
Elastic Modulus in Compression	$500f_m^c$	$500f_m^c$	$500f_m^c$
Flexural Tensile Strength ²	20 psi	10 psi	0
Shear Strength ³			
Masonry with a running bond lay-up	27 psi	20 psi	13 psi
Fully grouted masonry with a lay-up other than running bond	27 psi	20 psi	13 psi
Partially grouted or ungrouted masonry with a lay-up other than running bond.	11 psi	6 psi	5 psi

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

Table 6.5 Factors to Translate Lower-Bound Masonry Properties to Expected Strength Masonry Properties

Property	Factor
Compressive Strength (f_m^c)	1.3
Elastic Modulus in Compression ²	-
Flexural Tensile Strength	1.3
Shear Strength	1.3

1. See Chapter 6 for properties of reinforcing steel.

2. The expected elastic modulus in compression shall be taken as $550f_m^c$ where f_m^c is the expected masonry compressive strength.

⁷ FEMA 356

⁸ Adapted from FEMA 356

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

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

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

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

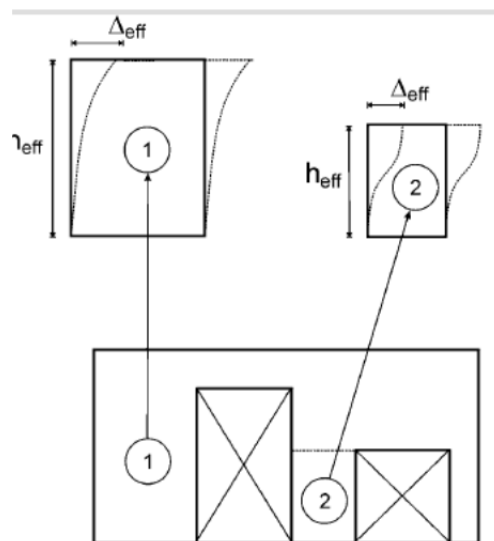
where:

- h_{eff} = wall height
- A_v = Shear area
- I_g = Moment of inertia for the gross section representing uncracked behavior
- E_m = Masonry elastic modulus
- G_m = Masonry shear modulus

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

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

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



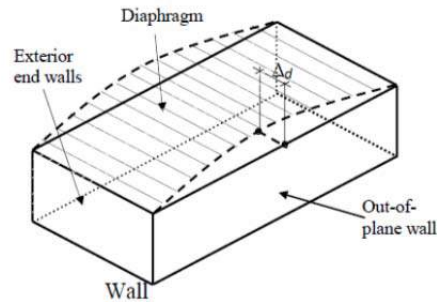
- h_{eff} = The effective height of the component under consideration.
- Δ = The differential displacement between the top and bottom of the component.

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

6.1.8 DEFLECTION CHECK OF DIAPHRAGM⁹

(a) Calculation of Diaphragm Deflection

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



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

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

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

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

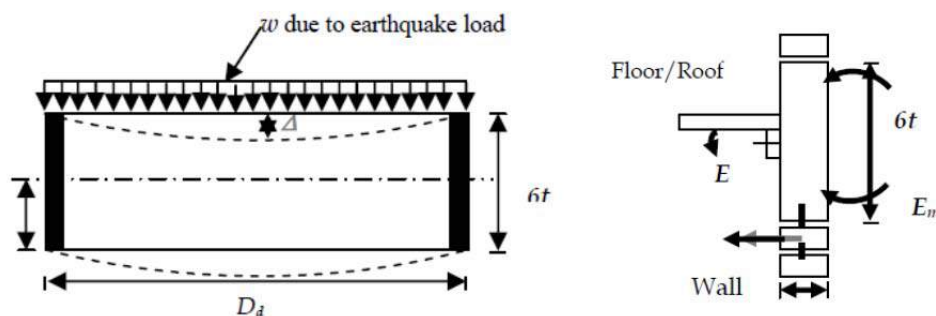


Figure C34(c) Diaphragm Deflection-I beam approximation

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

⁹ Adapted from IITK guideline (Rai, 2005)

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

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

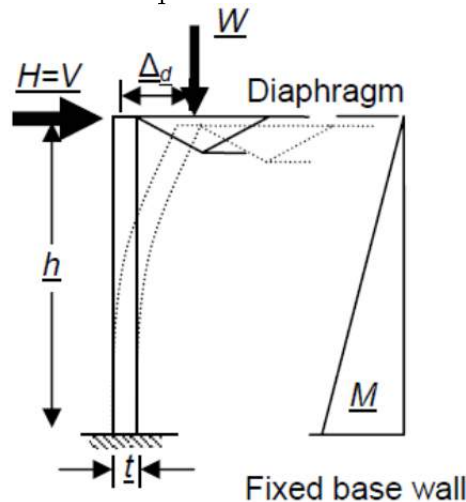
Where E_d and E_m are modulus of elasticity of diaphragm and masonry materials respectively.

(b) Control of Diaphragm Deflection:

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

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

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



Moment of the base of wall is calculated using equation.

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

and the resulting stress are shown in Figure (C39).

Hence,

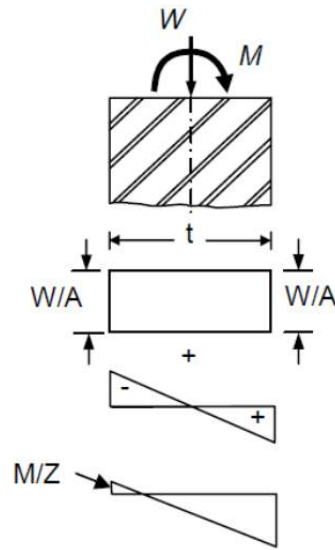
$$\frac{M}{Z} - \frac{W}{A} = f_t \quad \text{(Tensile)}$$

$$-\left(\frac{M}{Z} + \frac{W}{A} \right) = f_c \quad \text{(Compressive)}$$

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

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

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



Hence,

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

$$-\left(\frac{M}{Z} + \frac{W}{A} \right) = f_c \quad (\text{Compressive})$$

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

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

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

Stresses at the base of wall

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

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

6.1.9 ADDITION OF ELEMENTS

i. Addition of cross wall

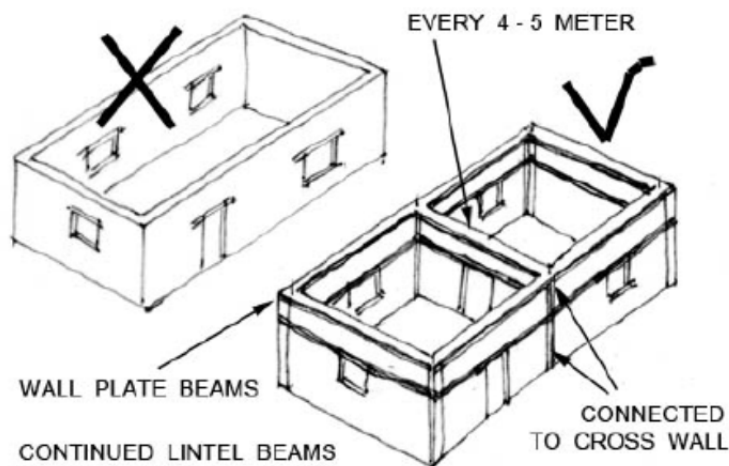


Figure 6- 2: Addition of cross-wall (Advies, 2012)

7 ALTERNATIVE APPROACH FOR ANALYSIS

7.1 PERFORMANCE BASED BEHAVIOR OF MASONRY¹⁰

7.1.1 Building Performance Levels

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

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

Target Building Performance Levels

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

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

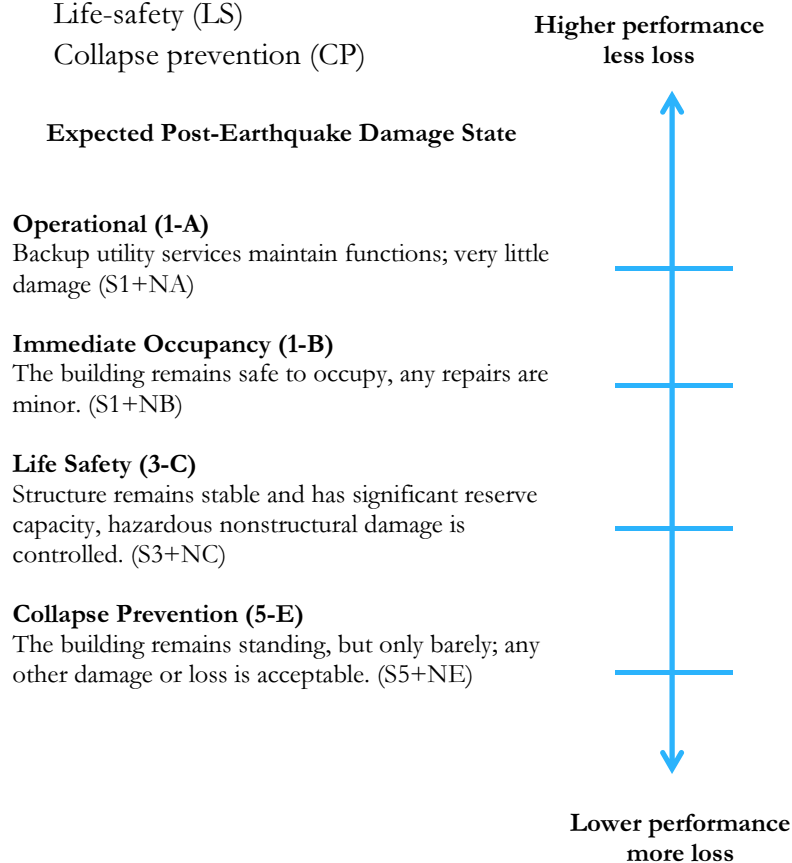


Figure 7- 1: Range of performance levels (FEMA 356-2000)

These performance levels are fairly self-explanatory and based on the desired condition of structural and architectural components in the building after an earthquake. Figure 7- 1 shows the range of performance levels and the expected damage state after the seismic event. These performance levels are combined with the earthquake hazard level of the site to obtain the rehabilitation objective for the project.

¹⁰ Adapted from FEMA 356 - 2000

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

7.2 PROPERTIES OF MASONRY WALLS

7.2.1 IN PLANE PROPERTIES OF URM WALL

7.2.1.1 Masonry Shear Strength

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

$$v_{me} = \frac{0.75 \left(0.75 v_{te} + \frac{P_{CE}}{A_n} \right)}{1.5} \dots\dots\dots (7-1b)$$

where,

- P_{CE} = Expected gravity compressive force applied to a wall or pier component considering load combinations of gravity load and earthquake load
- A_n = Area of net mortared/grouted section of a wall or pier
- v_{te} = Average bed-joint shear strength, v_{to} , given in Equation (7-1b)

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

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

where,

- V_{test} = Test load at first movement of a masonry unit
- A_b = Sum of net mortared area of bed joints above and below the test unit
- P_{D+L} = Stress due to gravity loads at the test location

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

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

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

$$Q_{CE} = V_{bjs} = v_{me} A_n \dots\dots\dots (7-2)$$

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

where,

- A_n = Area of net mortared/grouted section
- h_{eff} = Height to resultant of lateral force
- L = Length of wall or pier
- P_E = Expected axial compressive force due to gravity loads
- v_{me} = Expected bed-joint sliding shear strength
- V_{bjs} = Expected shear strength of wall or pier based on bed-joint sliding shear Strength
- V_r = Strength of wall or pier based on rocking
- α = Factor equal to 0.5 for fixed-free cantilever wall, or equal to 1.0 for fixed- fixed pier

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

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

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

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

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

- f_a = Axial compressive stress due to gravity loads
- f'_{dt} = Lower bound masonry diagonal tension strength
- f'_m = Lower bound masonry compressive strength
- P_L = Lower bound axial compressive force due to gravity loads
- V_{dt} = Lower bound shear strength based on diagonal tension stress for wall or pier
- V_{tc} = Lower bound shear strength based on toe compressive stress for wall or pier

7.2.2 OUT OF PLANE PROPERTIES OF URM WALL

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

7.2.2.1 Out-of-Plane Anchorage to Diaphragms

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

element and the distribution of out-of-plane forces to wall anchors and diaphragm ties shall consider the stiffening effect and accumulation of forces at these elements. Wall anchor connections shall be considered force-controlled.

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

where,

- F_p = Design force for anchorage of walls to diaphragms
- χ = Factor from Table 7-1 for the selected Structural Performance Level. Increased values of χ shall be used when anchoring to flexible diaphragms
- S_{xs} = Spectral response acceleration parameter at short periods for the selected hazard level and damping adjusted for site class
- W = Weight of the wall tributary to the anchor

Exceptions:

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

Table 7.1 Coefficient χ for Calculation of Out of Plane Wall Forces

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

7.2.2.2 Out-of-Plane Strength

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

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

where,

- F_p = Out-of-plane force per unit area for design of a wall spanning between two out-of-plane supports
- χ = Factor from Table 7-1 for the selected performance level. Values of χ for flexible diaphragms need not be applied to out-of-plane strength of wall components
- S_{xs} = Spectral response acceleration at short periods for the selected hazard level and damping adjusted for site class
- W = Weight of the wall per unit area

i. Stiffness

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

ii. Strength

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

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

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

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

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

iii. Acceptance Criteria

For the Immediate Occupancy Structural Performance Level, flexural cracking in URM walls due to out-of-plane inertial loading shall not be permitted as limited by the tensile stress requirements of Section 7.2.1.2. For the Life Safety and Collapse Prevention Structural Performance Levels, flexural cracking in URM walls due to out-of-plane inertial loading shall be permitted provided that cracked wall segments will remain stable during dynamic excitation. Stability shall be checked using analytical time-step integration models considering acceleration time histories at the top and base of a wall panel. For the Life Safety and Collapse Prevention Structural Performance Levels, stability need not be checked for walls spanning vertically with a height-to-thickness (h/t) ratio less than that given in Table below:

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

Wall Types	$S_{x1} \leq 0.24g$	$0.24g < S_{x1} \leq 0.37g$	$S_{x1} > 0.37g$
Walls of one-story buildings	20	16	13
First-story wall of multistory building	20	18	15
Walls in top story of multistory building	14	14	9
All other walls	20	16	13

7.2.3 IN PLANE PROPERTIES OF REINFORCED MASONRY WALL

i. Stiffness

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

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

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

ii. Strength

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

iii. Flexural Strength of Walls and Piers

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

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

iv. Shear strength of walls and piers

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

For M/Vd_v less than 0.25:

$$V_{CL} \leq 6\sqrt{f'_m A_n} \dots\dots\dots (7-8)$$

For M/Vd_v greater than or equal to 1.00:

$$V_{CL} \leq 4\sqrt{f'_m A_n} \dots\dots\dots (7-9)$$

where:

- A_n = Area of net mortared/grouted section
- f'_m = Lower bound compressive strength of masonry
- M = Moment on the masonry section
- V = Shear on the masonry section
- d_v = Wall length in direction of shear force

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

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

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

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

$$V_{sL} = 0.5 \left(\frac{A_v}{s} \right) f_y d_v \dots\dots\dots (7-11)$$

where:

- A_v = Area of shear reinforcement
- s = Spacing of shear reinforcement
- f_y = Lower bound yield strength of shear reinforcement

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

v. Strength considerations for flanged walls

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

1. The face shells of hollow masonry units are removed and the intersection is fully grouted.
2. Solid units are laid in running bond, and 50% of the masonry units at the intersection are interlocked.
3. Reinforcement from one intersecting wall continues past the intersection a distance not less than 40 bar diameters or 24 inches.

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

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

vi. Vertical compressive strength of walls and piers

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

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

where:

- f'_m = Lower bound masonry compressive strength
- f_y = Lower bound reinforcement yield strength

vii. Acceptance Criteria

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

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

viii. Default Properties

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

Table 7.3 Default Lower-Bound Masonry Properties

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

Table 7.4 Factors to Translate Lower-Bound Masonry Properties to Expected Strength Masonry Properties

Property	Factor
Compressive Strength (f_{mc})	1.3
Elastic Modulus in Compression ²	-
Flexural Tensile Strength	1.3

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

8 RETROFITTING OF DIFFERENT ELEMENTS

8.1 GENERAL

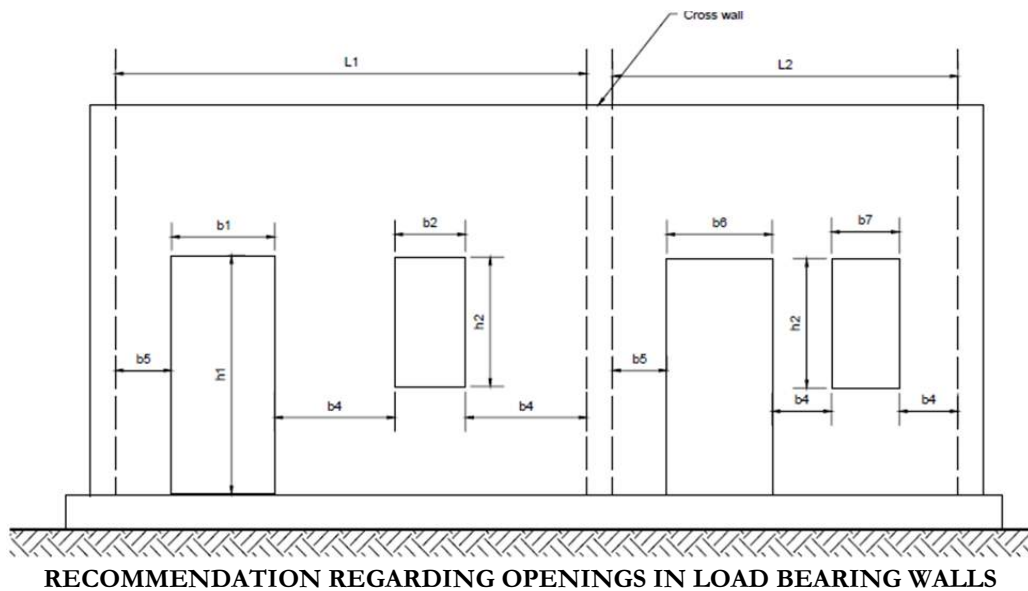
This guideline focuses on seismic strengthening (Retrofitting) of unreinforced Masonry Structures. Seismic retrofitting may require intervention at element level but the required performance shall be achieved at global level. The extent of the modifications must be determined by the general principles 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:¹¹

- 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 8-1.
- vi. Openings in load bearing walls should be restricted as shown in Figure 8.2

¹¹ 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>W $\geq 2W$</p>	

Figure 8-1 Recommended forms of buildings (Adapted from NBC 203)



Note:

- $b_1 + b_2 < 0.3 L_1$ for one story, $0.25 L_1$ for one plus attic storeyed.
- $b_6 + b_7 < 0.3 L_2$ for one storey, $0.25 L_2$ for one plus attic storeyed, three storeyed
- $b_4 \geq 0.5h_2$ but not less than 600 mm
- $b_5 \geq 0.25 h_1$ but not less than 420 mm.

Figure 8.2 Recommendation regarding openings in load bearing walls

8.2 STRENGTHENING OF FLOOR/ROOF

8.2.1 GENERAL

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

8.2.2 DIAPHRAGMS

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

Diaphragms and their connections to vertical elements providing lateral support shall comply with the following requirements¹².

i. RCC slabs

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

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

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

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

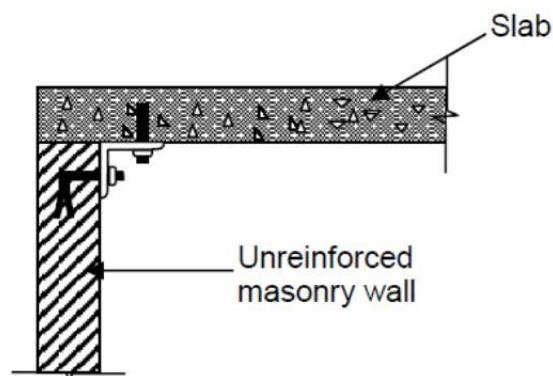


Figure 8-3 Anchorage of RCC slab with masonry wall

ii. Timber floors/roofs

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

¹² FEMA 356 page 2-21

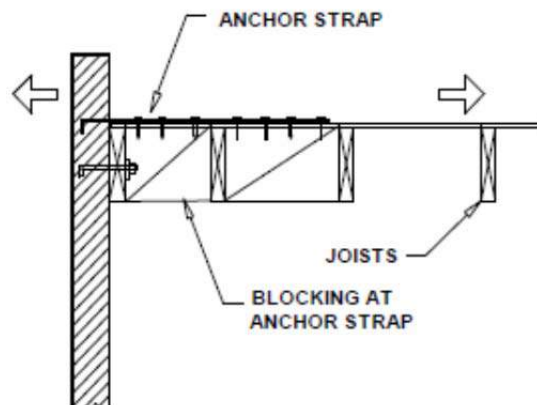


Figure 8-4 Wall anchorage

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

The connections between the walls and the diaphragm shall not induce cross-grain bending or tension in the wood members (see Figure 8-4). Connections that rely on cross-grain bending in wood members induce tension perpendicular to grain. Failure of such connections is sudden and non-ductile resulting in loss of bearing support and partial or complete collapse of the floors and roof.

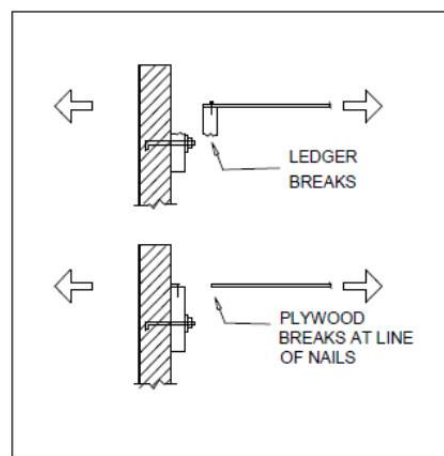


Figure 8-5 Connection relying in cross-grain of timber members induce cross-grain tension causing failure of the connection

Anchors shall be capable of development the maximum of¹³
 $2.5 S_{D1}$ times the weight of the wall
 KN per meter acting normal to the wall at the level of the floor or roof

¹³ FEMA 310 4.2.6.6

8.2.3 STIFFENING THE SLOPING ROOF SURFACE¹⁴

Most of the sloping roof are usually made of rafters, purlins with covering of burnt clay tiles or corrugated galvanized iron (CGI) sheets or asbestos – cement (AC) sheets on top. Sometimes sloping roofs on reinforced concrete slabs are also used. Such roofs push the walls outward during earthquakes. For stiffening such roofs, the rafters should be tied with the seismic belt as in Note 1 below, and the opposite rafters, on both sides of the ridge need to be connected near about mid-height of the roof through cross ties nailed to the rafters (See Figure 8.5). The important point in retrofitting is the provision of seismic belts just below eave level and the gable level.

Note 1:

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

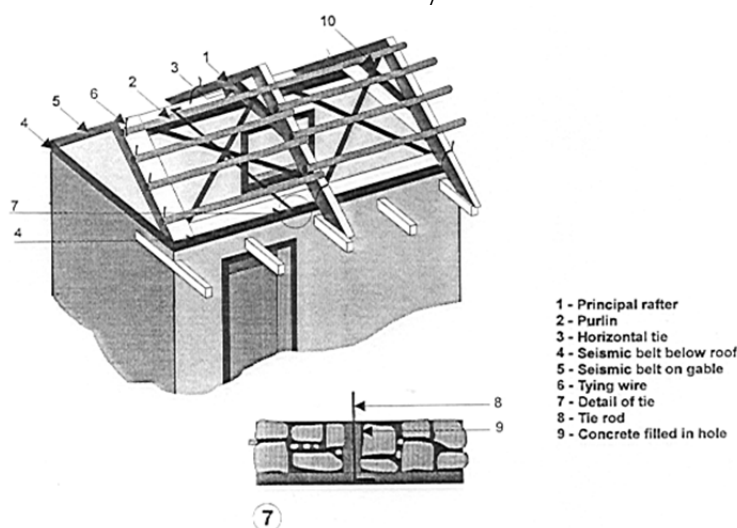


Figure 8-6 Stiffening of sloping roof structure

2. Using galvanized binding wire, tie up the roof rafters with the nails of the eave level belt before applying the plaster over the mesh.
3. In brick and Bela stone walls, it will be easy to drill or chisel out holes of 75mm 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.

8.2.4 REINFORCEMENT AT DIAPHRAGM OPENINGS

There shall be reinforcement around all diaphragm openings greater than 50% of the building width in either major plan dimension as shown in Figure 8-7.

¹⁴ Guidelines for Repair, Restoration and Seismic Retrofitting Of Masonry Buildings, Dr. Anand S. Arya, FNA, FNAE, March 2003

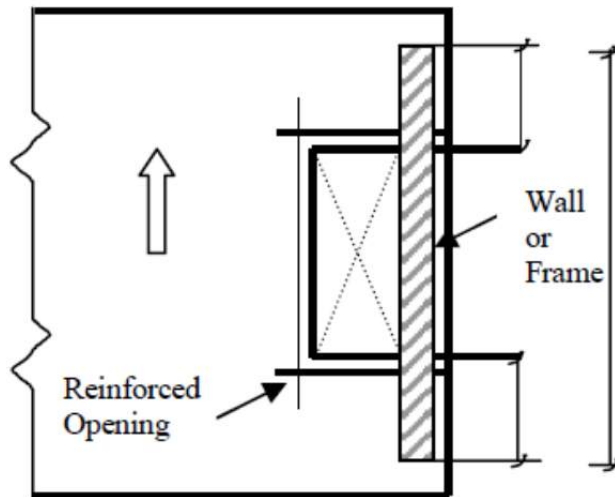


Figure 8-7 Opening adjacent to the masonry walls

8.3 STRENGTHENING OF WALL SECTIONS

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

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

- Steel wire mesh with plaster on both faces of the wall
- PP Band with cement or mud plaster on both faces of the wall
- Gabion wire net with or without plaster on both faces of the wall
- The retrofitted walls must be safe against worst combination of lateral forces and designers shall check it before starting the construction.
- A sample calculation for strengthening the wall by using GI wire is shown in Annex.
- A step by step approach for application of wire mesh and plaster in masonry building is given in Annex.

8.4 WALL OPENINGS

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

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

8.4.1 CONTROL ON DOOR AND WINDOW OPENINGS IN MASONRY WALLS

8.4.1.1 INFILL OPENINGS¹⁵

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.

¹⁵A Performance Based Approach to Retrofitting Unreinforced Masonry Structures for Seismic Loads by Keith Bouchard, 2006

8.4.1.2 SEISMIC BELTS AROUND DOOR / WINDOW OPENING¹⁶

The jambs and piers between window and door openings require vertical reinforcement as in Table No. 8.1.

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 8.1 Mesh and reinforcement for covering the jamb area

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

* N = Number of longitudinal wires in the mesh.

**B = Width of the micro concrete belt, half on each all meeting at the corner of T-junction.

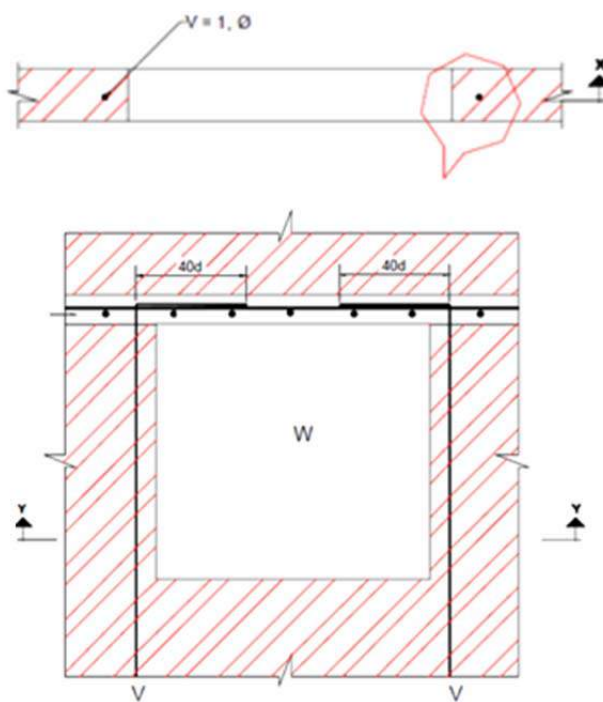
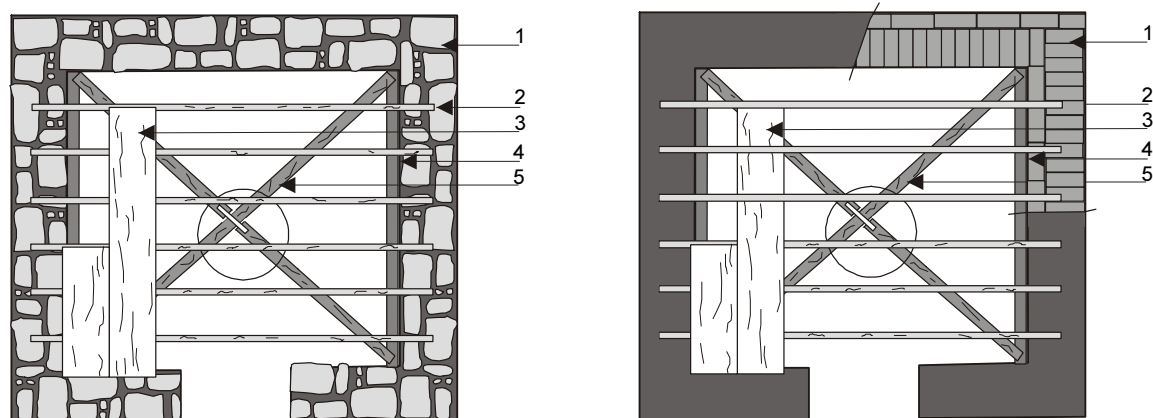


Figure 8-8 Reinforcement around opening in wall (Adapted from NBC 202)

¹⁶ Guidelines for Repair, Restoration and Retrofitting of Masonry Buildings in Kachhbh Earthquake affected areas of Gujarat,, Gujarat State Disaster Management Authority Government of Gujarat, March - 2002

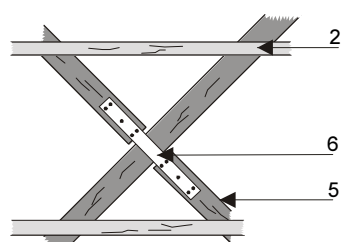
8.5 STIFFENING THE FLAT WOODEN FLOOR / ROOF¹⁷

Many of the damaged houses have flat floor or roof made of wood logs or timber joists covered with wooden planks and earth. For making such roof/floor rigid, long planks 100mm wide and 25 mm thick should be nailed at both ends of the logs/joists from below. Additionally, similar planks or galvanized metal strips 1.5 mm thick 50 mm wide should be nailed diagonally also (Figure 8-8).



(a) Stone building

(b) Brick building



- 1 - Wall
- 2 - Wood joist
- 3 - Wood plank
- 4 - Tie plank under ends of joist
- 5 - Diagonal ties
- 6 - Joint by nailing through 3 mm flat iron

Figure 8-9 Stiffening flat wooden floor/ roof

8.6 SHEAR WALLS¹⁸

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

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

¹⁷Guidelines For Repair, Restoration and Seismic Retrofitting of Masonry Buildings, Dr. Anand S. Arya, FNA, FNAE March 2003

¹⁸Canadian Journal on Environmental, Construction and Civil Engineering Vol. 2, No. 8, November 2011, An Investigation into the Interaction of Concrete Shear walls and Masonry Structures in the Seismic Performance of Concrete Shear Walls, M. Kheirollahi, B. Rafiezy

8.7 STRENGTHENING OF FOUNDATION

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

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

Foundation rehabilitation schemes shall be evaluated in conjunction with any rehabilitation of the superstructure and according to the general principles and requirements of this standard to assure that the complete rehabilitation achieves the selected building performance level for the selected earthquake hazard level. When new rehabilitation elements are used in conjunction with existing elements, the effects of differential foundation stiffness on the modified structure shall be demonstrated to meet the acceptance criteria¹⁹.

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

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

¹⁹ FEMA 356

9 ADVANCED APPROACHES FOR RETROFITTING OF MASONRY STRUCTURES

9.1 ENERGY DISSIPATION DEVICES FOR EARTHQUAKE RESISTANCE (DAMPER)²⁰

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

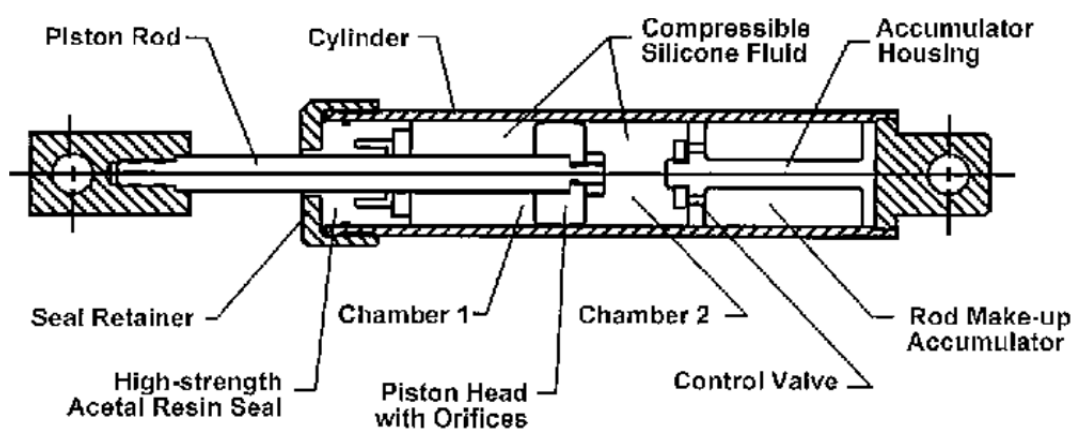


Figure 9-1 Typical Damper

9.1.1 Commonly used types of seismic dampers include:

- I) **Viscous Dampers** (energy is absorbed by silicone-based fluid passing between piston cylinder arrangement)
- II) **Friction Dampers** (energy is absorbed by surfaces with friction between them rubbing against each other)
- III) **Yielding Dampers** (energy is absorbed by metallic components that yield)
- IV) **Viscoelastic dampers** (energy is absorbed by utilizing the controlled shearing of solids)

9.1.2 BASE ISOLATION²¹

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

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

²⁰Two Activities- Base Isolation for Earthquake Resistance, TOTLE.

²¹Two Activities- Base Isolation for Earthquake Resistance, TOTLE.

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

- i) **Lead-rubber bearings** are frequently used for base isolation. A lead rubber bearing is made from layers of rubber sandwiched together with layers of steel. The bearing is very stiff and strong in the vertical direction, but flexible in the horizontal direction.
- ii) **Spherical sliding isolation** uses bearing pads that have a curved surface and low-friction materials similar to Teflon. During an earthquake the building is free to slide both horizontally and vertically on the curved surfaces and will return to its original position after the ground shaking stops. The forces needed to move the building upwards limit the horizontal or lateral forces that would otherwise cause building deformations.
- iii) **Working Principle** To get a basic idea of how base isolation works, first examine the diagrams above that illustrate traditional earthquake mitigation methods. When an earthquake vibrates a building with a fixed foundation, the ground vibration is transmitted to the building. The building's displacement in the direction opposite the ground motion is actually due to inertia. In addition to displacing in a direction opposite to ground motion, the un-isolated building is deformed. If the deformation exceeds the constraints of the building design, the structure of the building will fail. This failure often occurs in the ground floor because most of the building's mass is above that level. Also many buildings have "soft" ground floors with many windows or unreinforced spaces for parking or lobbies.

9.1.3 NON-METALLIC FIBRE COMPOSITES / FIBRE REINFORCED COMPOSITES (FRC)²²

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

Fibre Reinforced Polymer (FRP) composites comprise fibres of high tensile strength within a polymer matrix such as vinylester or epoxy. FRP composites have emerged from being exotic materials used only in niche applications following the Second World War, to common engineering materials used in a diverse range of applications such as aircraft, helicopters, spacecraft, satellites, ships, submarines, automobiles, chemical processing equipment, sporting goods and civil infrastructure. The role of FRP for strengthening of existing or new reinforced concrete structures is growing at an extremely rapid pace owing mainly to the ease and speed of construction, and the possibility of application without disturbing the existing functionality of the structure. FRP composites have proved to be extremely useful for strengthening of RCC structures against both normal and seismic loads. The FRP used for strengthening of RC structures can be mainly categorized as:

²²New and emerging technologies for retrofitting and repairs, DrGopalRai, CEO, R & M International Group of companies

(i) Laminates, for flexural strengthening

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

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



*Figure 9-2 Use of Laminates in for strengthening of slabs in a bridge and a building
(Source: Hari Darshan Shrestha)*

(ii) Fibre wraps, for shear and axial strengthening

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

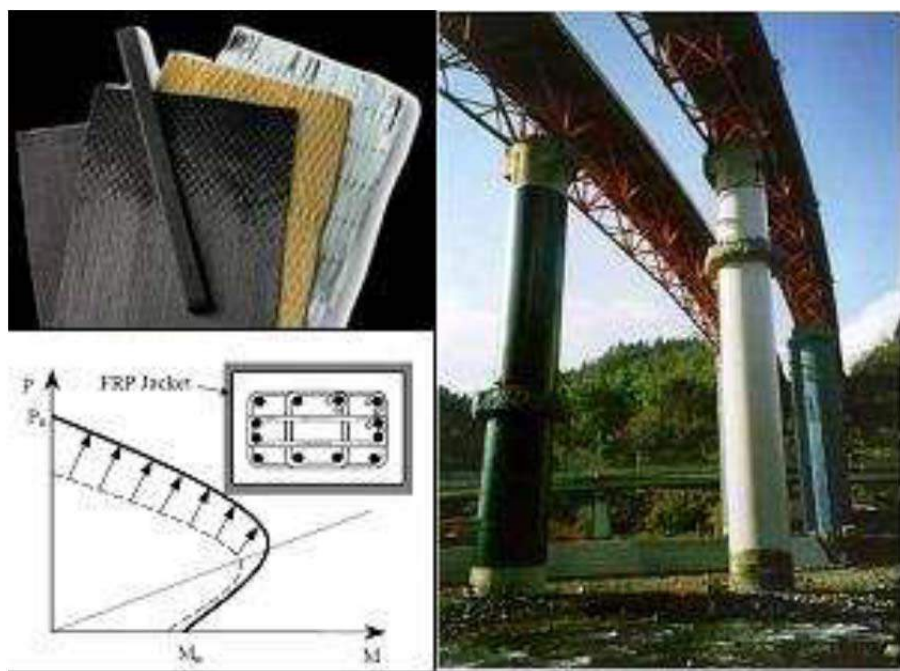


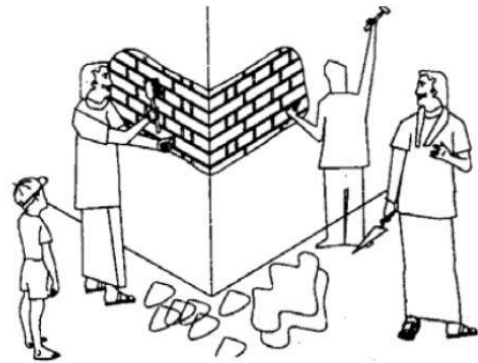
Figure 9-3 Different types and uses of Fiber wraps

10 DIFFERENT TECHNIQUES FOR STRENGTHENING

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

To strengthen a half brick thick load bearing wall

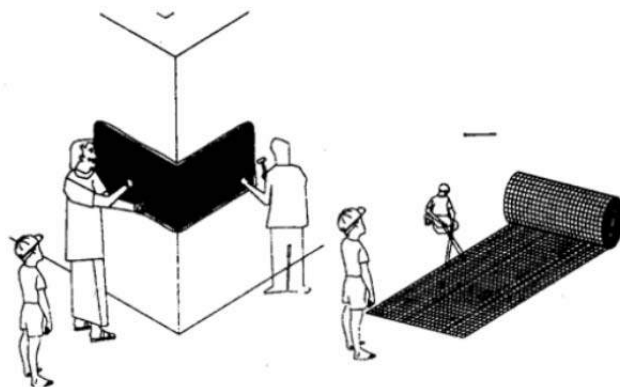
The welded wire mesh may be of 14 gauge wires @ 35 to 40 mm apart both ways. Provision of mesh on external or internal faces with an overlap of 30 cm at the corners will suffice for upto 3 m long walls. For longer walls, ferro-cement planting be provided on both faces.



Details of retrofitting elements

Ferro-Cement Planting:

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



- (i) Mark the height or width of the desired planting based on the weld mesh number of longitudinal wires and the mesh size.
- (ii) Cut the existing plaster at the edge by a mechanical cutter for neatness, and remove the plaster.
- (iii) Rake the exposed joints to a depth of 20 mm. Clean the joints with water jet.
- (iv) Apply neat cement slurry and plaster the wall with 1:3 cement-coarse and mix by filling all raked joints fully and covering the wall with a thickness of 15 mm. Make the surface rough for better bond with the second layer of plaster.
- (v) Fix the mesh to the plastered surface through 15 cm long nails driven into the wall at a spacing of 45 cm tying the mesh to the nails by binding wire.
- (vi) Now apply the second layer of plaster with a thickness of 15 mm above the mesh. Good bonding will be achieved with the first layer of plaster and mesh if neat cement slurry is applied by a brush to the wall and the mesh just in advance of the second layer of plaster.

10.2 GROUTING

Grouting is defined as the injection of fluid mortars or adhesives to fill discontinuities and cracks and reintegrate detached wall sections, is seen as a more promising solution to the problem²³.

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

10.2.1 METHODOLOGY FOR GROUTING OF CRACKS²⁴

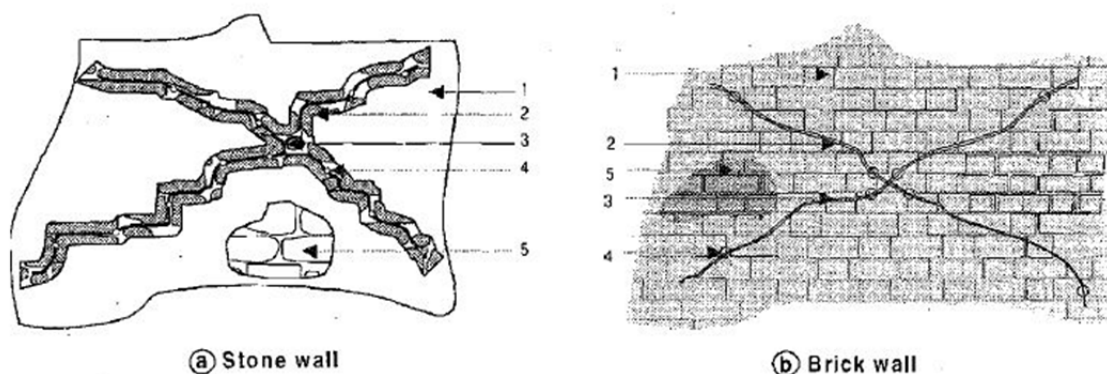
10.2.1.1 Minor and medium cracks (crack width 0.5 mm to 5.0mm)

Material / Equipment required

- i) Plastic/ Aluminum nipples of 12mm dia.(30 to 40 mm long)
- ii) Non-shrink cement (shrinkomp of ACC or equivalent).
- iii) Polyester putty of 1:3 cement sand mortar for sealing of the cracks.
- iv) Compressor for injecting the slurry.

Procedure: - See Figure 10-1 and Figure 10-2.

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



1. Plaster
2. Plaster removed and cracks cleaned
3. Cracks sealed with mortar or putty
4. Grout ports
5. Plaster fallen to be done again

Figure 10-1 Filling grout in cracks

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

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

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

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

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

²³ Interdisciplinary Experts Meeting on Grouting Repairs for Large-scale Structural Cracks in Historic Earthen Buildings in Seismic Areas, The Getty Conservation Institute Pontificia Universidad Católica del Perú August 13-16, 2007

²⁴ Guidelines for Repair, Restoration & Retrofitting of Masonry Buildings in Kachhbh earthquake affected areas of Gujarat, Gujarat State Disaster Management Authority, Government of Gujarat March - 2002

- Step-7 Make cement slurry with 1:1(1-non shrink cement:1-water) and start injecting from lower most nipple till the cement slurry comes out from the next higher nipple and then move to next higher nipple.
- Step-8 After injection grouting through all the nipples is completed, replaster the finish the same.

10.2.1.2 Major Crack (Crack width more than 5.0mm)

Material / equipment required

- i) Plastic/ Aluminum nipples of 12 mm dia. (30 to 40 mm long).
- ii) Polyester putty of 1:3 cement-sand mortar for sealing of cracks.
- iii) Non-Shrink cement (shrinkomp of ACC or equivalent).
- iv) Compressor for injecting the slurry.
- v) Galvanized steel wire fabric (16 to 14 gauge i.e. 1.5 to 2.03 mm dia. wire) with 25 mm x 25mm.
- vi) Galvanized steel clamping rod of 3.15 mm dia, or 5 mm dia 150 mm long wire nails.

Procedure:-

- Step-1 Remove the plaster in the vicinity of crack exposing the cracked bare masonry.
- Step-2 Make the shape of crack in the V-Shape by chiseling out.
- Step-3 Clean the crack with compressed air.
- Step-4 Fix the grouting nipples in the V-groove in both faces of the wall at spacing of 150-200 mm c/c.
- Step-5 Clean the crack with the compressed air through nipples to ensure that the fine and loose material inside the cracked masonry has been removed.
- Step-6 Seal the crack on both the faces of the wall with polyester putty or cement mortar 1:3 (1-cement: 3-coarse sand) and allowed to gain strength.
- Step-7 Inject water starting with nipples fixed at higher level and moving down so that the dust inside the crack is washed off and masonry is saturated with water.
- Step-8 Make cement slurry with 1:2:W (1-non shrink cement: 2- fine sand: just enough water) and start injecting from lower most nipple till the slurry comes out from the next higher nipple and then move to next higher nipple.
- Step-9 After injection grouting through all the nipples is completed, replaster the surface and finish the same.

Alternative Procedure:

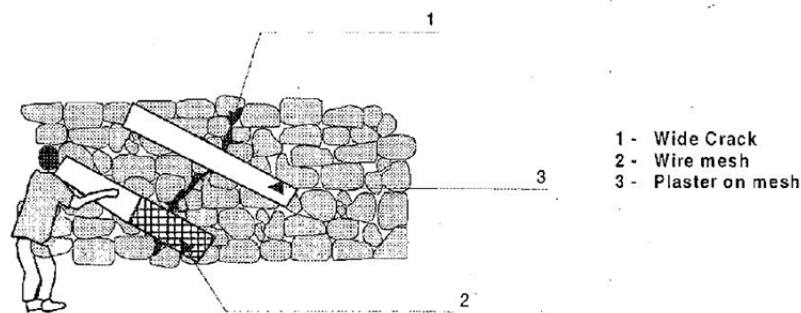


Figure 10-2 Fixing mesh across wide cracks

- Step-1 Remove the plaster in the vicinity of crack exposing the cracked bare masonry.
- Step-2 Make the shape of crack in the V-shape by chiseling out.
- Step-3 Clean the crack with compressed air.

- Step-4 Fill the crack with cement mortar 1:3 (1-non shrink cement: 3-fine sand: necessary water) from both sides as deemed as feasible.
- Step-5 Provide wire mesh on both the faces of wall after removal of plaster in the region of repair to a width of 150mm on each side of the crack.
- Step-6 Clamp the mesh with the wall using clamps or wire nails at the spacing of 300mm c/c.
- Step-7 Plaster the meshed area with cement sand mortar of 1:3, covering the mesh by a minimum of 12mm.

10.3 JACKETING

Jacketing consists of covering the wall surface with a thin layer of reinforced mortar, micro-concrete, or shotcrete overlays interconnected by means of through-wall anchors²⁵. One of the most used traditional techniques for strengthening existing residential masonry Buildings, by improving its lateral resistance and energy dissipation capacity of the system is application of reinforced-cement coating (jacket) on one or both sides of the walls. (See Figure 10-3).²⁶

The method of jacketing can be summarized as below:

1. Remove the existing plaster from the wall and in the joints between the bricks or blocks, 10 to 15mm deep, and grouting of the cracks in the wall.
2. The wall surface is cleaned, water moistened and spattered with cement milk.
3. The welded steel reinforcement mesh with 4-6 mm bars at 100-150 mm intervals in vertical and horizontal directions is placed in between two cement coatings with 10-15 mm thick cement mortar layer with compressive strength of 20-30 MPa.
4. The wire mesh is connected with steel anchors 6mm diameter bars placed in pre-drilled holes and cement or epoxied on the wall surface, with quantity of 4-6 pieces per m².

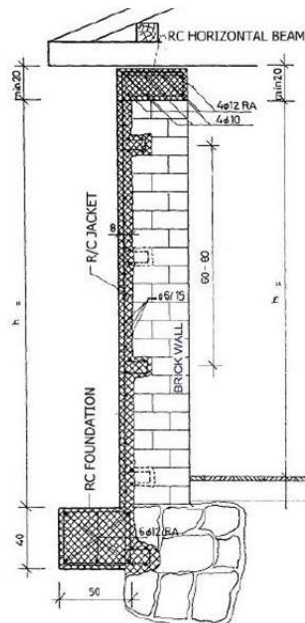


Figure 10-3 Application of RC coating

25 A TUTORIAL: Improving the Seismic Performance of Stone Masonry Buildings, Jitendra Bothara • Svetlana Brzev, First Edition, April 2011
 26 Experimental and Analytical Research of Strengthening Techniques for Masonry, Sergey Churilov, Elena Dumova-Jovanoska

5. Except for connecting and securing the coating reinforcement to the existing masonry with steel anchors, the connection can be achieved by inserting shear connectors from cage reinforcement. The total thickness of the jacket should not exceed 30 mm.
6. Ideally, jacketing should be applied to both interior and exterior wall surfaces, but this may not always be possible due to functional or financial constraints. In the case of a single- surface application, steel dowels of adequate size and spacing should be provided to ensure that the existing stone wall and the new jacket act in unison (Figure 10-4). However, it should be noted that the effectiveness of single-surface jacketing is significantly inferior to double-sided application because a single-sided jacket cannot confine the wall.

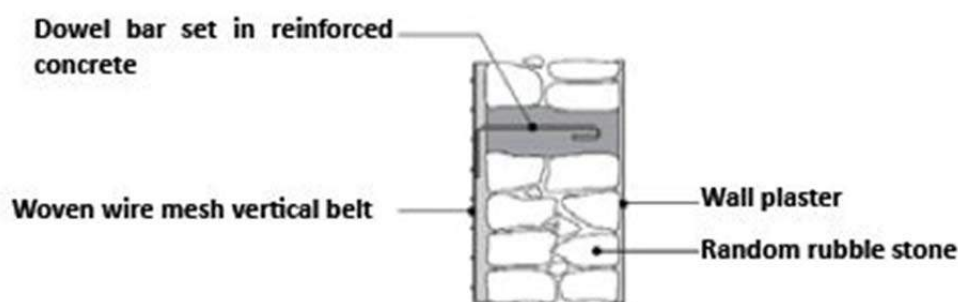


Figure 10-4 Single-sided jacketing showing steel dowels

10.4 SEISMIC BAND AND BELT

10.4.1 Seismic Bands (Ring Beams)²⁷

A seismic band is the most critical earthquake-resistant provision in a stone masonry building. Usually provided at lintel, floor, and/or roof level in a building, the band acts like a ring or belt, as shown in

Figure 10-5. Seismic bands are constructed using either reinforced concrete (RC) or timber. Proper placement and continuity of bands and proper use of materials and workmanship are essential for their effectiveness.

Seismic bands hold the walls together and ensure integral box action of an entire building. Also, a lintel band reduces the effective wall height. As a result, bending stresses in the walls due to out-of-plane earthquake effects are reduced and the chances of wall delamination are diminished. During earthquake shaking, a band undergoes bending and pulling actions, as shown in Figure 10-6. A portion of the band perpendicular to the direction of earthquake shaking is subjected to bending, while the remaining portion is in tension.

Seismic bands can be provided at plinth, lintel, floor, and roof levels (Figure 10-7). In some cases, a lintel band is combined with a floor or roof band. An RC plinth band should be provided a top the foundation when strip footings are made of unreinforced masonry and the soil is either soft or uneven in its properties.

²⁷A TUTORIAL: *Improving the Seismic Performance of Stone Masonry Buildings*, JitendraBothara• Svetlana Brzev, First Edition, April 2011



Figure 10-5 A seismic band acts like a belt (adapted from: GOM 1994)

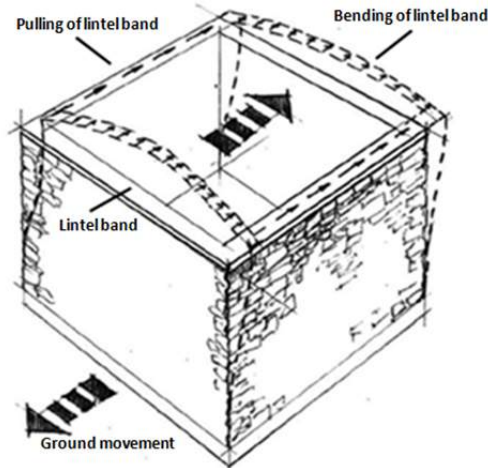


Figure 10-6 Pulling and bending of a lintel band in a stone masonry building (adapted from: Murty 2005)

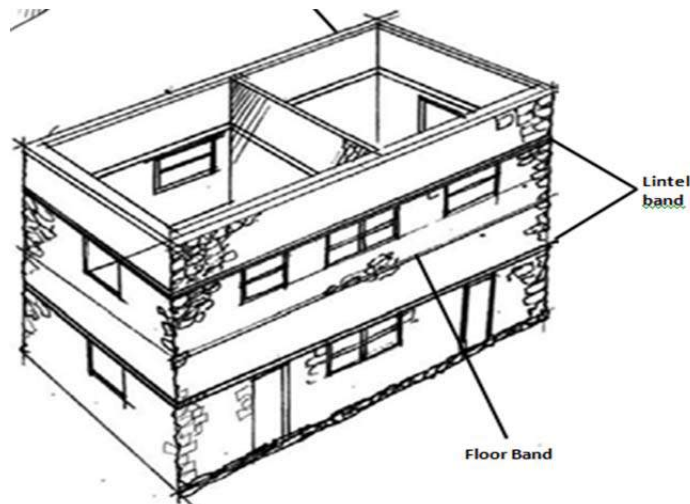


Figure 10-7 Location of seismic bands in a stone masonry building (roof omitted for clarity) (adapted from: UNCRD 2003)

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

Seismic bands must be continuous (like a loop or a belt), otherwise they are inefficient. Some examples of undesirable discontinuities in lintel band construction are illustrated in Figure 10-8 and Figure 10-9.

Lintel beams (commonly known as lintels) are required a top all the openings in a wall. However, if a band is provided at the lintel level, a lintel beam can be cast as an integral part of the lintel band to minimize construction costs, as illustrated in Figure 10-10. Details for combining a lintel and floor/roof band are shown in Figure 10-11. The band must be continuously reinforced at the wall intersections, as shown in Figure 10-12.

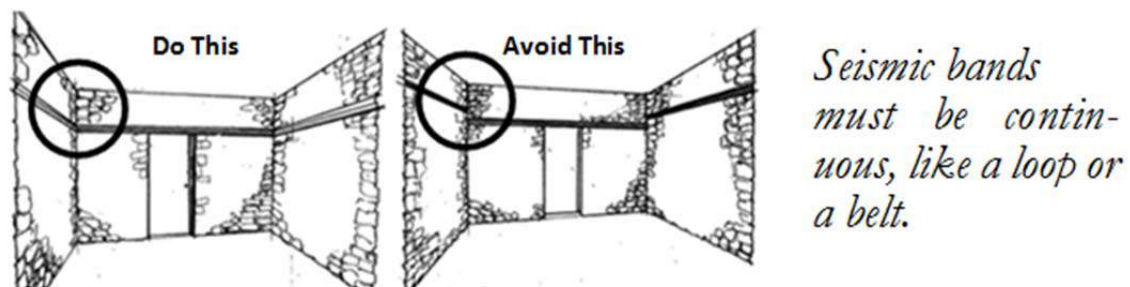


Figure 10-8 Seismic bands should always be continuous; an offset in elevation is not acceptable (adapted from: GOM 1998)

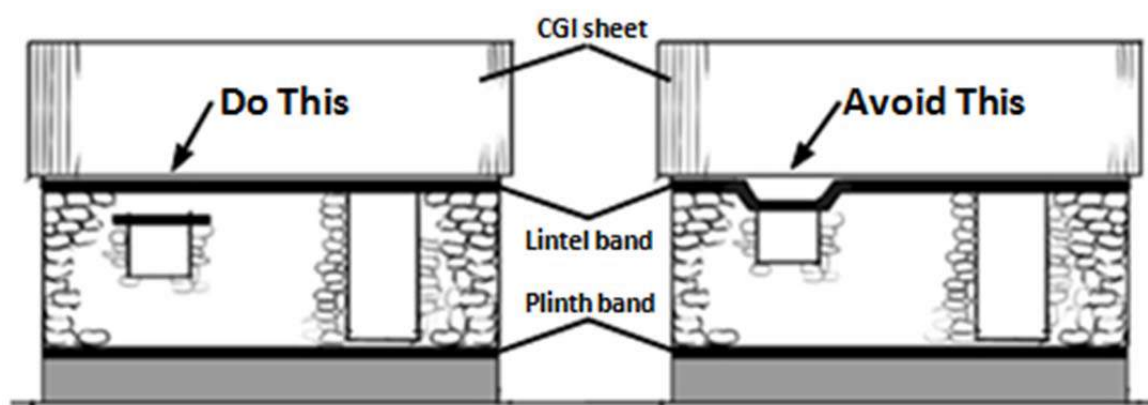


Figure 10-9 RC seismic bands should always remain level without any dips or changes in height (adapted from: GOM 1998)

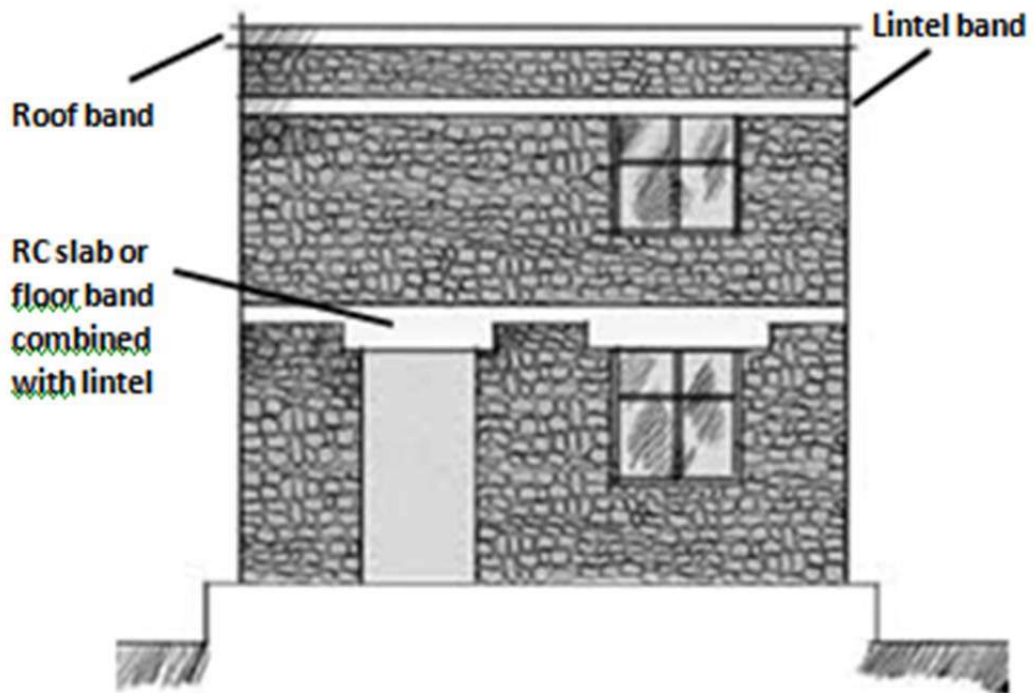


Figure 10-10 Merging RC floor and lintel bands

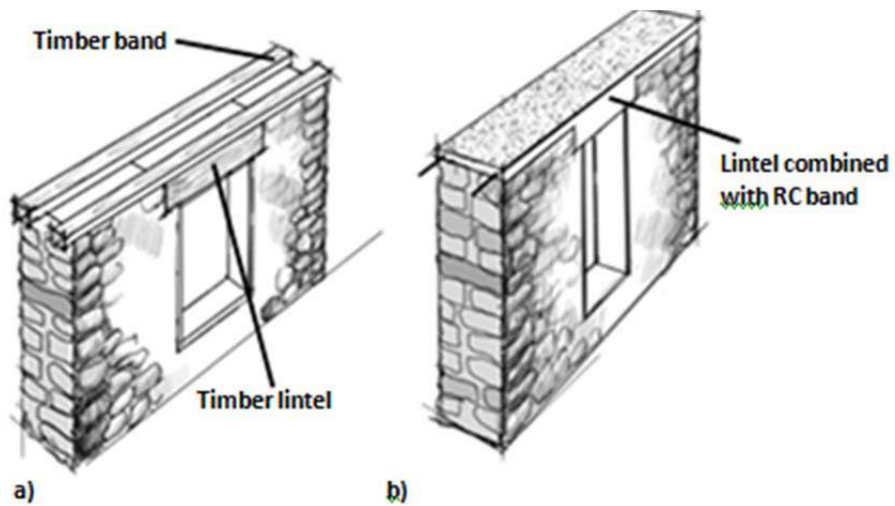


Figure 10-11 Combining floor/roof and lintel band: a) timber band, b) RC band

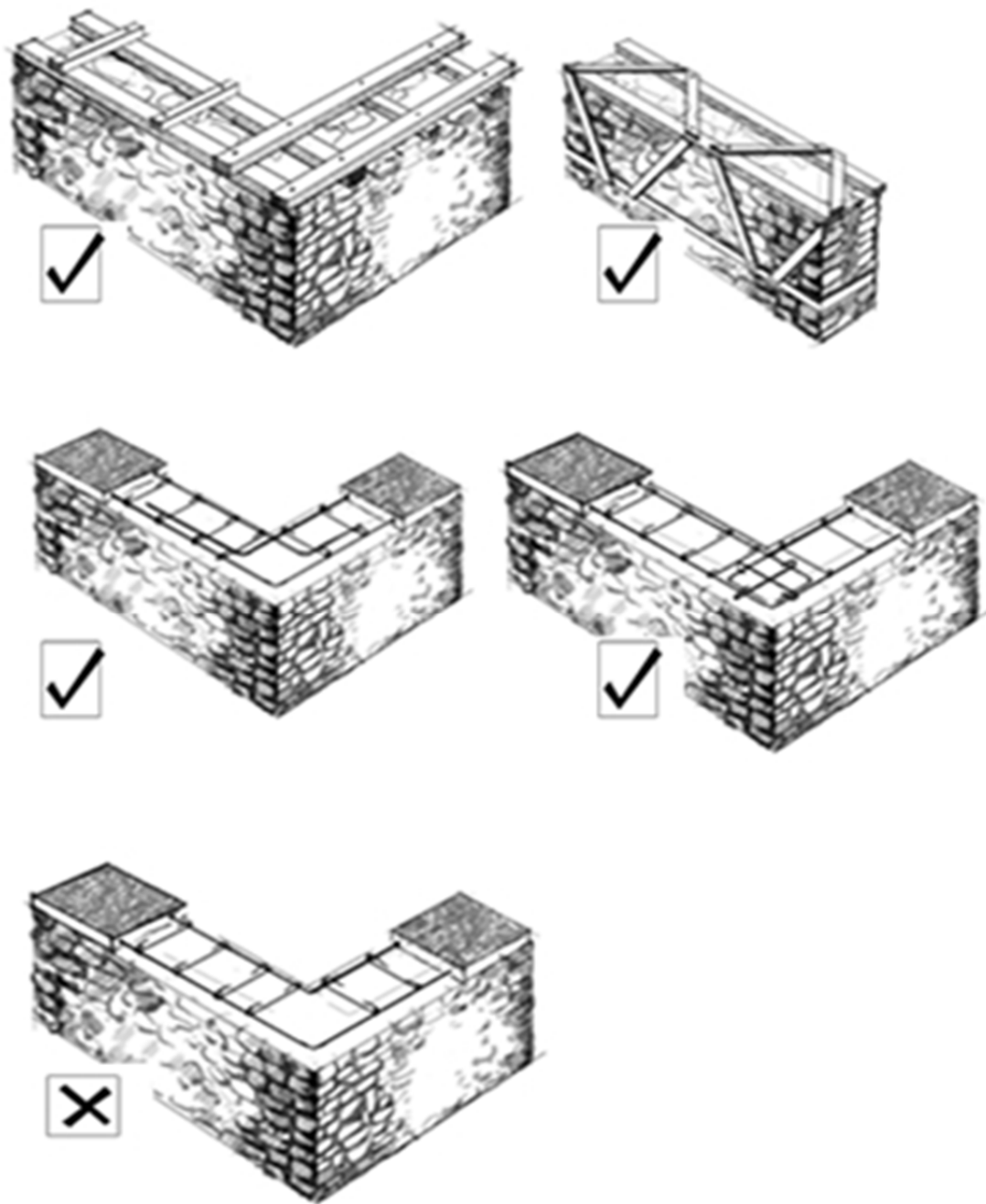


Figure 10-12 Recommended detailing of timber and RC bands (adapted from: T. Schacher and C.V.R Murty)

10.5 VERTICAL REINFORCEMENT AT CORNERS AND THE JUNCTIONS OF WALLS²⁸

The vertical reinforcement consisting of TOR bar as per Table 10-1 or equivalent shall be provided on the inside corner of room starting from 750 mm below the ground floor going up to the roof slab, passing through each middle floor through holes made in the slabs. (As in fig. shown) The reinforcement will be connected to the walls by using L shape dowels of 8 mm TOR bar, the vertical leg of 400 mm length firmly tied to the vertical reinforcement bars and the horizontal leg of minimum 150 mm length embedded in the walls through 75 mm dia. holes drilled in the wall into which the 8 mm dia. leg of the dowel will be grouted using non- shrink cement cum polymer grout. Such dowels will be provided, first one just above plinth level and then at about every 1 m distance apart. The corner reinforcement will be covered with 1:3 cement mortars or 1:1 ½:3 micro concrete fully bonded with the walls giving a minimum cover of 15 mm on the bar.

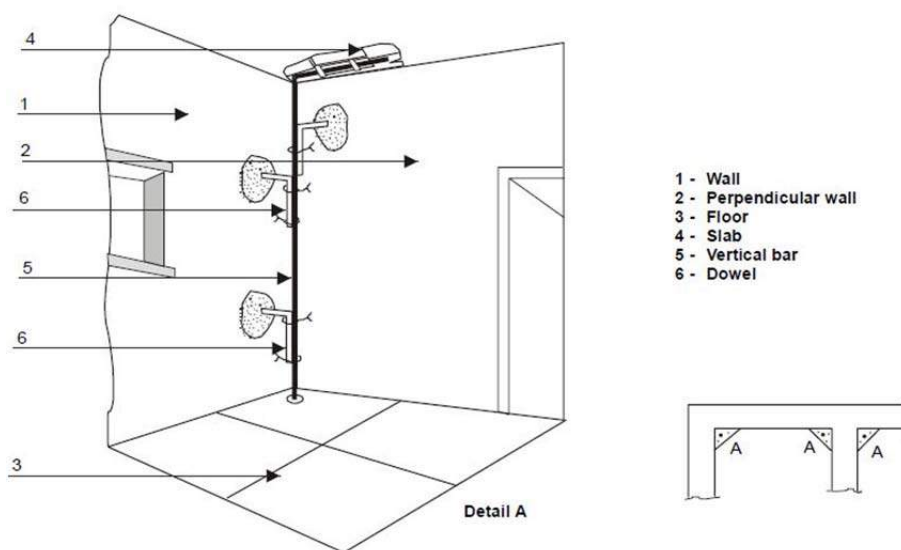


Figure 10-13 Vertical Reinforcement at corner and Junctions of Walls

Table 10-1 Vertical Bar of Mesh Reinforcement in Vertical Belt at inside Corners of Rooms

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

²⁸ Guidelines for Repair, Restoration & Retrofitting of Masonry Buildings in Kachhbh earthquake affected areas of Gujarat, Gujarat State Disaster Management Authority, Government of Gujarat March - 2002.

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

10.5.1 HOW TO INSTALL VERTICAL BAR IN A CORNER²⁹

1. Identify the inside corner for installation of vertical bar. Select appropriate location to maintain vertical continuity between storeys in case of a multi-storey structure
2. Mark the area where the bar is to be installed. Using plumb-bob, demarcate a 100 mm (4") wide patch at the corner on both walls as the limits of concreting for encasing the rod.
3. Use electric grinder if available, cut the plaster along vertical boundary of both the patches to restrict the removal of plaster.
4. Remove the plaster from the marked area and expose the walling material. Rake all the mortar joints to the depth of 12 mm (½"). Clean the surface with a wire brush.
5. Remove flooring within 300 mm x 300 mm patch at the corner and excavate to 450 mm depth.
6. Make holes for installing shear connectors in both walls, starting on one wall at 150 mm (6") from the floor, with successive holes at approximately every 600 mm (2') but in alternate walls, and the last hole 150 mm below the ceiling level or 150 mm below eave level. Clean all the holes with wire brush to remove loose material.
7. Place appropriate diameter bar in the floor excavation with the lower 150 mm (6") bent in 'L' shape. In a structure with CGI roof, the top end can be connected to one of the principal elements of the attic floor or the roof. In case of an RC slab roof, the top end can be bent into 'L' shape for connecting to the slab reinforcement. The rod will pass through each intermediate floor.
8. Place appropriately shaped 8 mm TOR bar in the holes made for shear connectors and connect them to the vertical bar making sure that the vertical bar is 35 to 50 mm (½" to 2") from each wall.
9. With vertical bar plumb and at right distance from the walls pour concrete in 1:2:4 proportion in the hole excavated in the floor, with continuous rodding, to completely encase the bottom of the steel rod in concrete.
10. Clean all the shear connector holes by splashing water and wetting the surface of the holes thoroughly. Fill up the holes with non-shrink cement cum polymer grout. Make sure that the grout completely encases the shear connector bar.
11. Once all the shear connectors are grouted, clean the exposed surfaces of the wall with wire brush and water.
12. Install centering for concreting around the vertical bar. This can be done with GI sheet or timber plank. The concreting must be done in stages with the height of each new stage not exceeding 900 mm (3'). Pour 1:1½:3 micro-concrete into the form work, with continuous rodding to prevent honeycombing. Once the concrete is set, move the formwork upwards and continue concreting. Encase the entire length of the vertical bar in this manner. The bar must have the minimum concrete cover of 15 mm. Connecting top bent end of vertical rod to slab reinforcement

²⁹ Manual for Restoration and Retrofitting of Rural Structures in Kashmir

13. Where the roof is of RC slab, in the vicinity of the vertical bar, break the bottom concrete cover to expose the slab reinforcing bars. Connect the top bent portion of the vertical bar to the exposed bars of the slab using binding wires providing a minimum of 300 mm (12") overlap. Wet the exposed surface of the slab and then apply neat cement slurry. Finally apply cement mortar in 1:4 proportions and finish the joint to match the surrounding area.
14. Cure all concrete work for 15 days.

10.6 FOUNDATION RESTRENGTHENING / REHABILITATION (BASED ON FEMA 356)

1. *Soil Material Improvements:*

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

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

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

2. *Shallow Foundation Rehabilitation:*

The following measures may be effective in the rehabilitation of shallow foundations:

- 2.1. New isolated or spread footings may be added to existing structures to support new structural elements such as shear walls or frames.
- 2.2. Existing isolated or spread footings may be enlarged to increase bearing or uplift capacity. Consideration of existing contact pressures on the strength and stiffness of the modified footing may be required unless uniform distribution is achieved by shoring and/or jacking.
- 2.3. Existing isolated or spread footings may be underpinned to increase bearing or uplift capacity. Underpinning improves bearing capacity by lowering the contact horizon of the footing. Consideration of the effects of jacking and load transfer may be required.
- 2.4. Uplift capacity may be improved by increasing the resisting soil mass above the footing.

- 2.5. Mitigation of differential lateral displacement of different portions of a building foundation may be carried out by provision of interconnection with grade beams, reinforced grade slab or ties.
3. *Deep Foundation Rehabilitation:*
The following measures may be effective in the rehabilitation of deep foundation consisting of driven piles made of steel, concrete, or wood, or cast-in-place concrete piers, or drilled shafts of concrete.
- 3.1. Shallow foundation of spread footings or mats may be provided to support new shear walls or frames or other new elements of the lateral force-resisting system, provided the effects of differential foundation stiffness on the modified structure are analyzed and meet the acceptance criteria.
- 3.2. New wood piles may be provided for an existing wood pile foundation. A positive connection should be provided to transfer the uplift forces from the pile cap or foundation above to the new wood piles. Existing wood piles should be inspected for deterioration caused by decay, insect infestation, or other signs of distress prior to undertaking evaluation of existing wood pile foundation.
- 3.3. Driven piles made of steel, concrete, or wood, or cast-in-place concrete piers or drilled shafts of concrete, may be provided to support new structural elements such as shear walls or frames.
- 3.4. Driven piles made of steel, concrete, or wood, or cast-in-place concrete piers or drilled shafts of concrete, may be provided to supplement the vertical and lateral capacities of existing pile and pier foundation groups.

10.7 ALTERNATE APPROACH FOR ROOF REPLACEMENT/ STRENGTHENING/RETROFITTING

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

- i) Complete the repair and retrofitting work of the first storey including provision of 'through' elements.
- ii) Stiffen the first floor wooden deck.
- iii) Complete the horizontal and vertical seismic belts in the first storey, keep the vertical mesh reinforcement extending beyond the first storey by 300mm and leave uncovered.
- iv) Complete the repair and retrofitting of the second storey.
- v) Now open the roof structure and remove gable portion up to eave level.
- vi) It will be preferable to use seismic bands, instead of belts in this case. Therefore, construct the vertical seismic belts and the eave level and gable bands together, taking the vertical steel in the bands. Anchor steel wires in the bands and extend out for tying down the rafters and purlins.
- vii) Now erect the rafters, tie them with bands and fix the tie to make A-frames.
- viii) Install diagonal bracing in the plane of the rafters.
- ix) Erect the purlins over the rafters, tie them with rafters and gable bands. Bolt down the AC or CGI sheets to the purlins using J or U bolts with iron and bitumen

ANNEX

ANNEX A - EXAMPLES

EXAMPLE 1

A.1. STRENGTH BASED ANALYSIS

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

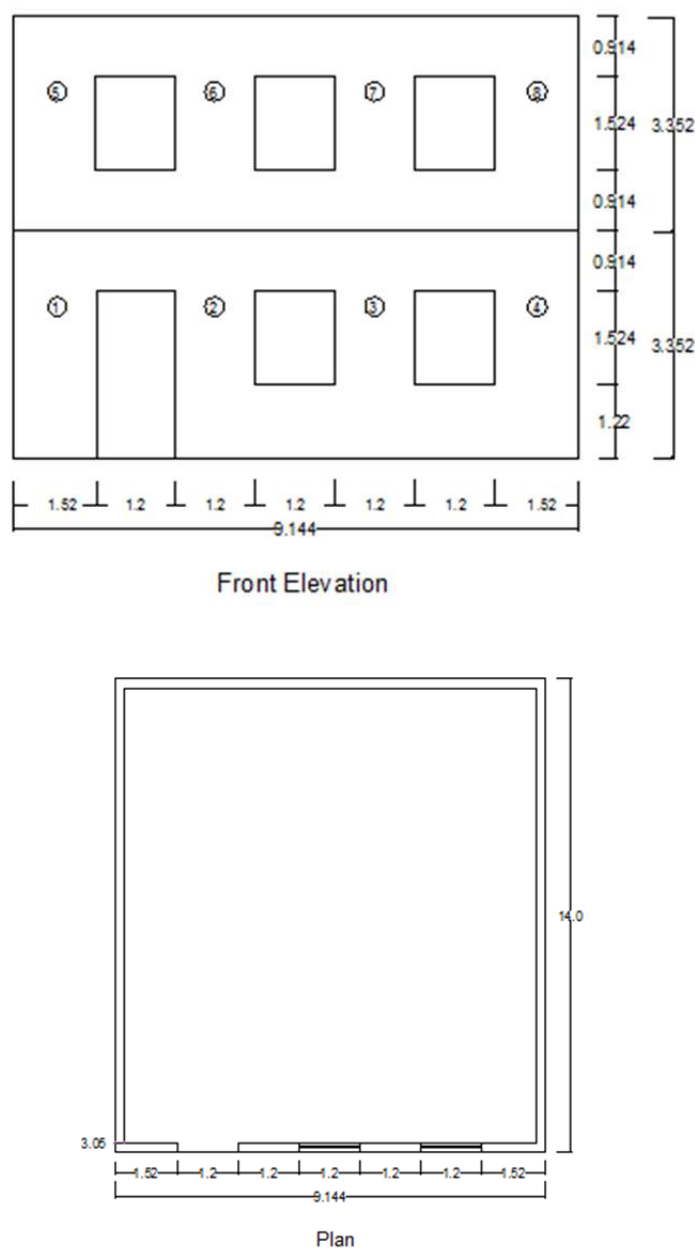


Figure A- 1: Elevation and Plan of a fictitious building

1. Detailed Evaluation

1.1. Floor and Roof Dead Loads

Floor : 1.5

Roof : 1.2

1.2. Unit Weight of Walls

Weight of the wall per m run

	Wall Thickness (m)	KN/m ²
Second	0.305	6.1
First	0.305	6.1

Opening Top from Roof	= 0.914 m
Opening Top from FF	= 0.914 m
No. of opening in SF	= 3
Opening one O1	= 1.2 × 1.524
O 2	= 1.2 × 1.524
O 3	= 1.2 × 1.524
No. of opening in FF	
O 1	= 1.2 × 2.438
O 2	= 1.2 × 1.524
O 3	= 1.2 × 1.524

Level	Height (m)	Wall Thickness
Second	3.352	0.305
First	3.352	0.305

Unit weight of Brick	= 20 KN/m ³
Building Length	= 14 m
Breadth	= 9.144 m
Roof Live Load	= 1.5 KN/m ²
Room Live Load	= 2 KN/m ²

Opening	
Top	Bottom
0.914	0.914
0.914	0.914
0.914	0.914
0.914	0.914
0.914	0.914
0.914	0.914

1.3. Seismic Weights

Roof Load

Roof Dead Load	= 153.6192 KN	
Side Walls (Length)	= 286.2608 KN	
End Walls (Width)	= 50.98146 KN	Front Side
End Walls (Width)	= 93.4846 KN	Back Side
Walls weight between opening	= 25.76962 KN	Front Side
Walls weight between opening	= 0 KN	Back Side
Seismic Live Load	= 0 KN	(No consideration of roof live load as per IS 1893:2002)
Total Seismic Weight on Roof	= 610.1157 KN	

First Floor Load

Floor Dead Load	= 192.024 KN	
Side Walls (Length)	= 572.521 KN	
End Walls (Width)	= 101.9629 KN	Front Side
End Walls (Width)	= 186.9692 KN	Back Side
Walls weight between opening	= 51.53924 KN	Front Side
Walls weight between opening	= 0 KN	Back Side

Seismic Live Load = 64.008 KN (25% of live load as per IS 1893:2002)

Total Seismic Weight of FF = 1169.025 KN

Total Seismic Weight of the building = 1779.1241 KN

1.4 Calculation of A_{hm}

$A_{hm} = A, U$	From IS 15988:2013
$Z = 0.36$	From IS 1893:2002
$I = 1$	From IS 1893:2002
$R = 1.5$	From IS 1893:2002

Time Period of Building

Along-x= $0.09 \cdot h / \sqrt{d_x}$	=	0.19953	From IS 1893:2002
Along-y= $0.09 \cdot h / \sqrt{d_y}$	=	0.161255	From IS 1893:2002
For this Time Period, $S_a/g = S_a/g_x = S_a/g_y$	=	2.5	From IS 1893:2002
From IS 1893:2002, $A_h = Z/2 \cdot I/R \cdot S_a/g$	=	0.300	
U(assumed)	=	0.7	From IS15988:2013 Sec 5.4
A_{hm}	=	0.21	

1.5 Calculation of DCR Values

1.5.1 Roof Diaphragm

WD	= Roof Load + Load on Side Walls
WD	= 439.88 KN
v_u	= 3.6KN/m From IITK GSDMA Guideline: Table 4

In- plane width dimension of masonry, D_d	= 9.144m	
$\sum v_u D_d$ = Sum of Diaphragm shear capacities	= 65.8368	From IITK GSDMA Guideline: EQ.A.2
K = Knowledge factor	= 0.8	From IS15988 Table1
DCR = Demand capacity Ratio	= $2.5 \cdot A_{hm} W_d / \{K \cdot \sum (v_u D_d)\}$	= 4.38
Since this ($L=14$, $DCR=4.38464885899679$) fall in region3 of the Figure 5 of the IITK GSDMA Guideline, the roof diaphragm does not need cross wall.		

1.5.2 First Floor Diaphragm

W_d	= 828.5536 KN	
v_u	= 7.3KN/m	From IITK GSDMA Guideline: Table 4
In- plane width dimension of masonry, D_d	= 9.144 m	
$\sum v_u D_d$ = Sum of Diaphragm shear capacities	= 133.5024	From IITK GSDMA Guideline:EQ.A.2
k = Knowledge factor	= 0.8	From IS15988 Table1
DCR = Demand Capacity Ratio	= $2.5 \cdot A_{hm} W_d / \{K \cdot \sum (v_u D_d)\}$	= 4.07
		From IITK GSDMA Guideline:EQ.A.2
Since this ($L=14$, $DCR=4.07287284722971$) fall in region3 of the figure 5 of the IITK GSDMA Guideline, the roof diaphragm does not need cross wall.		

Table 1.1 Checking acceptability of diaphragm Span (Special Procedure)

Level	D,m	$\sum v_u D_d$	DCR=2.5* $A_{hm} W_d / \{K * \sum (v_u D_d)\}$	Region from Figure 2 of Draft Code for Coordinates (L,DCR)
Roof	9.144	65.8368	4.38	Region 3 (14,4.38464885899679)
1	9.144	133.5024	4.07	Region 3(14,4.07287284722971)

1.6.1. Design Seismic Base Shear

$V_B = A_{hm} W = 373.6195 \text{ KN}$ From IITK GSDMA Guideline: EQ.A.1.

1.6.2. Distribution of Base Shear to Floor Levels

$Q_i = V_s * W_i h_i^2 / \text{Summation } (W_j H_j^2, j=1, n)$ From IS 1893:2002

Table 1.2 Distribution of Base Shear to

Level	$W_j \text{ (KN)}$	$h_i \text{ (m)}$	$W_i * h_i^2$	$Q_i \text{ (KN)}$
Roof	610.1157	6.704	27420.8	252.6133
1	1169.025	3.352	131351.1	121.0062
			40555.86	

1.6.3. Strength of Diaphragm

$F_{px} = (\sum Q_i / \sum W_i) * w_{px}$ From IITK GSDMA Guideline Clause 4.6

Table 1.3. Checking Strength of the Diaphragm

Level	$Q_i = V_B * W_i h_i^2 / \text{Summation } (W_j h_j^2)$	$\sum Q_i \text{ KN}$	$W_i, \text{ KN}$	$\sum W_i \text{ KN}$	$w_{px}, \text{ KN}$	0.35 $Z_i w_{px}$	0.75 $Z_i w_{px}$	$F_{px} = (\sum Q_i / \sum W_i) * w_{px}$	F_{px} / w_{px}
Roof	252.6132851	252.6133	610.1157	610.11568	439.88	55.42488	118.7676	182.1286292	0.41
1	121.0062473	373.6195	1169.025	1779.1406	828.5536	104.3978	223.7095	173.996256	.21

The Diaphragm Force, F_{px} shall not be more than $0.75 Z_i w_{px}$ and less than $0.35 Z_i w_{px}$
 $w_{px} = W_d =$ weight of roof of floor diaphragm

1.7 Actual h/t Ratio for walls

	Height, h	Thickness, t	h/t	Check (From IS 15988:2013 Table 3)
Top Storey:	3.352	0.305	10.99	> 9 Not Satisfied
First Storey:	3.352	0.305	10.99	<15 OK

Top Storey Need Bracing

1st Storey is ok and out of plane Stability

1.8 Diaphragm Shear Transfer

The Design connection between the diaphragm and the masonry walls is to be lesser of:

$V_d = 1.5 * A_{hm} C_p W_d$ or From IITK GSDMA Guideline: EQ.A.6

$V_d = V_u D_d$ From IITK GSDMA Guideline: EQ.A.7

$C_p =$ horizontal force factor = Coefficient given in Table 5 of IITK GSDMA Guideline

1.8.1. Roof Level

$C_p = 0.5$

$A_{hm} = 0.21$

$$\begin{aligned}
W_d &= 439.88 \\
V_u &= 3.6 \\
D_d &= 9.144 \\
V_d &= 1.5 * AhmCpW_d \text{ or } 69.2811 \text{ KN} \\
\text{or } V_d &= v_u D_d = 32.9184 \text{ KN}
\end{aligned}$$

Therefore Adopt, $V_d = 32.9184 \text{ KN}$

1.8.2. First Floor

$$\begin{aligned}
C_p &= 0.5 \\
Ahm &= 0.21 \\
W_d &= 828.5536 \\
V_u &= 7.3 \\
D_d &= 9.144 \\
V_d &= 1.5 * AhmCpW_d \text{ or } 130.4972 \text{ KN} \\
\text{or } V_d &= V_u D_d = 66.7512 \text{ KN}
\end{aligned}$$

Therefore Adopt, $V_d = 66.7512 \text{ KN}$

1.9. In-plane Shear for Masonry Wall

1.9.1. Design in-plane Shears

$$F_{wx} = Ahm (W_{wx} + 0.5 * W_d) \quad \text{From IITK GSDMA Guideline: EQ. A.8}$$

But should not exceed

$$\text{Force applied to a wall at level } x = F_{wx} = Ahm W_{wx} + v_u * D_d$$

From IITK GSDMA Guideline: EQ. A.9

W_{wx} = Dead load of an unreinforced masonry wall assigned to level under consideration.

1.9.1.1. Root Level

Side (Long) Walls

$$\begin{aligned}
Ahm &= 0.21 \\
W_{wx} &= 143.1304 \text{ KN} \\
W_d &= 439.88 \text{ KN} \\
v_u &= 3.6 \\
D_d &= 9.144 \\
F_{wx} &= 76.24 \text{ KN} \quad \text{This should be less than } = 62.98
\end{aligned}$$

Adopt,

$$F_{wx} = 62.98 \text{ KN}$$

End (Short Walls)

$$\begin{aligned}
Ahm &= 0.21 \\
W_{wx} &= 85.11784 \text{ KN} \\
W_d &= 439.88 \text{ KN} \\
v_u &= 3.6 \\
D_d &= 9.144 \\
F_{wx} &= 64.05 \text{ KN} \quad \text{This should be less then } = 50.79
\end{aligned}$$

Adopt,

$$F_{wx} = 50.79 \text{ KN}$$

1.9.1.2. First Floor Level

Side (Long) Walls

$$Ahm = 0.21$$

$$\begin{aligned}
 W_{wx} &= 286.2608 \text{ KN} \\
 W_d &= 828.5536 \text{ KN} \\
 V_u &= 7.3 \\
 D_d &= 9.144 \\
 F_{wx} &= 147.11 \text{ KN} \quad \text{This should be less than} = 126.87
 \end{aligned}$$

Adopt,

$$\begin{aligned}
 F_{wx} &= 126.87 \text{ KN} \\
 \text{End (Short Walls)} \\
 A_{hm} &= 0.21 \\
 W_{wx} &= 170.2357 \text{ KN} \\
 W_d &= 828.5536 \text{ KN} \\
 v_u &= 7.3 \\
 D_d &= 9.144 \\
 F_{wx} &= 122.75 \text{ KN} \quad \text{This should be less than} = 102.50
 \end{aligned}$$

Adopt,

$$F_{wx} = 102.50 \text{ KN}$$

Table 1.4 Summary of Storey Shear Force

Wall	Level	Storey Force F_{wx} KN	Wall Storey Shear Force KN $\sum F_{wx}$
Long walls	Roof	62.98	62.98
	FF	126.87	189.84
Short Walls	Roof	50.79	50.79
	FF	102.50	153.29

1.10 Check for in plane shear strength of End (Short) Masonry Walls

Shear Wall strength

$$V_a = v_a D_t \quad \text{GSDMA Guideline Clause 4.10}$$

$$v_a = 0.1 v_{te} + 0.15 P_{CE} / A_n \quad \text{GSDMA Guideline Clause 4.10}$$

Where,

P_{CE} = Expected gravity compressive force applied to a wall or pier.

v_a = Expected masonry shear strength

v_{te} = average bed joint shear strength = 0.4 MPa for typical old masonry buildings

A_n = Area of net mortared grouted section

Rocking Shear Strength

$$V_r = 0.5 P_o D / H \quad \text{GSDMA Guideline Clause 4.10}$$

Where,

V_r = Rocking shear capacity of an unreinforced masonry wall or wall pier

P_o = Superimposed dead load at the top of the pier under consideration

H = Total Height of the building

If $V_a > V_r$ = Rocking controlled mode

If $V_a < V_r$ = Shear controlled mode

Width of wall = 0.305 m

Length of pier 1,4,5,8 = 1.572 m $A_n = 0.47946 \text{ m}^2$

Length of pier 2,3,6,7 = 1.2 m $A_n = 0.366 \text{ m}^2$

Roof Load Calculation on pier

$$\text{Total Roof Load} = 153.6192 \text{ KN}$$

Roof Load on Short Wall = 5.4864 KN/m
 Effective Length of Pier 5 and 8 = 2.172 m
 Effective Length of Pier 6 and 7 = 2.4 m

Axial load on
 Pier 5 and 8 = 11.91646 Superimposed Dead
 Pier 6 and 7 = 13.16736 Load

First Floor Load calculation on pier

Total Floor Load = 192.024 KN
 Roof Load on Short wall = 6.858 KN/m
 Effective Length of Pier 1 and 4 = 2.172 m
 Effective Length of Pier 2 and 3 = 2.4 m
 Wall load above Ff = 16.7872 KN/m

Axial load on
 Pier 1 and 4 = 63.27384 Superimposed Dead
 Pier 2 and 3 = 69.91584 Load

Table 1.5 Summary of Pier Analysis

Story	Pier	Pd, KN	D,m	H,m	t,m	Va,KN	Vr,KN	Mode
2	5 and 8	11.91646	1.572	1.524	0.305	19.501823	6.15	Rocking
	6 and 7	13.16736	1.2	1.524	0.305	15.133776	5.18	Rocking
1	1 and 4	63.27384	1.572	1.524	0.305	20.895708	32.63	Shear
	2 and 3	69.91584	1.2	1.524	0.305	17.261844	27.53	Shear

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

A.2. NONLINEAR DYNAMIC PROCEDURE (NDP)

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

A.2.1. CALCULATION EXAMPLE OF PERFORMANCE BASED APPROACH FOR THE FICTITIOUS TWO-STOREY BRICK URM BUILDING UNDER CONSIDERATION:

The same building used for analysis of strength based analysis in Annex 1 Section A.1 is analyzed by performance based approach. Calculation is shown below:

i. Without Retrofitting

Wall Thickness	= 0.3048 m
Masonry Unit Wt	= 18.8505 KN/m ²
Modal Degree of freedom	= 2
Number of Stories	= 2
Masonry Condition	= fair with running bond lay up
From Fema 356 Table 7-1	
Poisson Ratio	= 0.25

Property	Lower (KN/m ²)	Expected (KN/m ²)
Compressive Strength	41308.2	5379.66
Elastic Modulus	2276010	2958813
Gm	910404	1183525
Flexural tensile strength	68.97107	89.66239
Shear Strength of		
Masonry with a running bond lay up	137.94214	179.3248

Wall Stiffness

Wall thickness	= 0.3048 m
Masonry Unit Wt	= 18.8505 KN/m ³

FIRST STORE			SECOND STORY		
Store Ht	=	3.6576	Store Ht	=	3.6576
No. of opening	=	3	No. of opening	=	3
No. of pier	=	4	No. of pier	=	4
Pier 1			Pier 4		
Pier Length	=	1.524 m	Pier Length	=	1.524 m
Effective height, hef	=	2.7432	Effective height, hef	=	1.524
Io	=	0.089906 m ⁴	Io	=	0.089906 m ⁴
Av	=	0.464515 m ²	Av	=	0.464515 m ²
	LB	Ex		LB	Ex
k (KN/m ²)	67143.617	87286.7	k (KN/m ²)	198208	257670.3

FIRST STOREY			SECOND STOREY		
Pier 2			Pier 2		
Pier Length	=	1.2192 m	Pier Length	=	1.2192 m
Effective height, hef	=	1.524	Effective height, hef	=	1.524
Io	=	0.046032 m ⁴	Io	=	0.046032 m ⁴
Av	=	0.3716125 m ²	Av	=	0.371612 m ²
	LB	Ex		LB	Ex
k (KN/m ²)	136611.02	177594.3	k (KN/m ²)	136611	177594.3
Pier 3			Pier 3		
Pier Length	=	1.2192 m	Pier Length	=	1.2192 m
Effective height, hef	=	1.524	Effective height, hef	=	1.524
Io	=	0.046032 m ⁴	Io	=	0.046032 m ⁴
Av	=	0.371612 m ²	Av	=	0.371612 m ²
	LB	Ex		LB	Ex
k (KN/m ²)	136611.02	177594.3	k (KN/m ²)	136611	177594.3
Pier 4			Pier 4		
Pier Length	=	1.524 m	Pier Length	=	1.524 m
Effective height, hef	=	1.524	Effective height, hef	=	1.524
Io	=	0.089906 m ⁴	Io	=	0.089906 m ⁴
Av	=	0.464515 m ²	Av	=	0.464515 m ²
	LB	Ex		LB	Ex
k (KN/m ²)	198207.96	2576703	k (KN/m ²)	136611	177594.3
Total First Storey Stiffness			Total Second Storey Stiffness		
	LB	Ex		LB	Ex
	538573.6	700145.7		669638.0	870529.3

	Property	Lower (KN/m ²)	Expected (KN/m ²)
Wall Thickness = 0.3048			
Masonry Unit Wt = 18.8505	Compressive Strength	4138.2	5379.66
Length = 9.144 m	Elastic Modulus	2276010	29588.13
Width = 9.144 m	GM	910404	1183525
Dead Load = 1.2 KN/m ²	Flexural tensile strength	68.97107	89.66239
Live Load = 1.2 KN/m ²	Shear strength of		
	Masonry with a running bond lay-up	137.9421	179.3248

Gravity Loads (Fema 356 Section 3.2.8)

Storey 1		Storey 2	
Ht. of masonry above (m)	4.572	Ht. of masonry above (m)	1.2192
Dead Load from Masonry KN/m	26.26903	Dead Load from Masonry KN/m	7.005075
Dead Load from Building KN/m	5.4864	Dead Load from Building KN/m	5.4864
Live Load from building (KN/m)	5.4864	Live Load from building (KN/m)	5.4864
QG (factored additive) =1.1 (Total Dead Load+Live)	53.03609	QG (factored additive) =1.1 (Total Dead Load+Live)	19.77566
QG (factored counteractive) =0.9 (Total Dead Load)	33.51765	QG (factored counteractive) =0.9 (Total Dead Load)	11.24233

Wall Strength

Expected Strength					
Pier 1			Pier 5		
Length (m) =	1.524		Length (m) =	1.524	
heff (m) =	2.7432		heff (m) =	2.7432	
An(m ²) =	0.464515		An(m ²) =	0.464515	
vt (KN/m ²) =	179.32		vt (KN/m ²) =	179.32	
PCE (KN) =	80.83		PCE (KN) =	30.14	
vme (KN/m ²) =	154.2497		vme (KN/m ²) =	99.68743	
6 =	1		6 =	1	
QCE:			QCE:		
Vbjs (KN) =	71.65132		Vbjs (KN) =	46.30632	
Vr (KN) =	40.415	Controls	Vr (KN) =	27.126	Controls
<i>Reference: Fema 356 Clause 7.4.2.2.1</i>					

Pier 2			Pier 6		
Length (m) =	1.2192		Length (m) =	1.2192	
heff (m) =	1.524		heff (m) =	1.524	
An(m ²) =	0.371612		An(m ²) =	0.371612	
vt (KN/m ²) =	179.32		vt (KN/m ²) =	179.32	
PCE (KN) =	64.66		PCE (KN) =	24.11	
vme (KN/m ²) =	154.2443		vme (KN/m ²) =	99.68474	
6 =	1		6 =	1	
QCE:			QCE:		
Vbjs (KN) =	57.31906		Vbjs (KN) =	37.04406	
Vr (KN) =	46.5552	Controls	Vr (KN) =	17.3592	Controls
<i>Reference: Fema 356 Clause 7.4.2.2.1</i>					

Pier 3			Pier 7		
Length (m) =	1.2192		Length (m) =	1.2192	
heff (m) =	1.524		heff (m) =	1.524	
An(m ²) =	0.371612		An(m ²) =	0.371612	
vt (KN/m ²) =	179.32		vt (KN/m ²) =	179.32	
PCE (KN) =	64.66		PCE (KN) =	24.11	
vme (KN/m ²) =	154.2443		vme (KN/m ²) =	99.68474	
6 =	1		6 =	1	
QCE:			QCE:		
Vbjs (KN) =	57.31906		Vbjs (KN) =	37.04406	
Vr (KN) =	46.5552	Controls	Vr (KN) =	17.3592	Controls
<i>Reference: Fema 356 Clause 7.4.2.2.1</i>					

Pier 4			Pier 8			
Length (m) =	1.524		Length (m) =	1.524		<i>Reference: Fema 356 Clause 7.4.2.2.1</i>
heff (m) =	1.524		heff (m) =	1.524		
An(m ²) =	0.464515		An(m ²) =	0.464515		
vt (KN/m ²) =	179.32		vt (KN/m ²) =	179.32		
PCE (KN) =	80.83		PCE (KN) =	30.14		
vme (KN/m ²) =	154.2497		vme (KN/m ²) =	99.68743		
6 =	1		6 =	1		
QCE:			QCE:			
Vbjs (KN) =	71.65132		Vbjs (KN) =	46.30632		
Vr (KN) =	72.747	Controls	Vr (KN) =	27.126	Controls	
Expected Storey Vbjs (KN) =	257.9408		Expected Storey Vbjs (KN) =	166.708		
Strength (QCE)=	206.2724	Controls	Strength (QCE)=	88.9704	Controls	

Lower Bound Strength						
Pier 1			Pier 5			
Length (m) =	1.524		Length (m) =	1.524		<i>Reference: Fema 356 Clause 7.4.2.2.2</i>
heff (m) =	2.7432		heff (m) =	1.524		
An(m ²) =	0.464515		An(m ²) =	0.464515		
f _a (KN/m ²) =	109.9641		f _a (KN/m ²) =	36.87716		
PCL (KN) =	51.08		PCL (KN) =	17.13		
f _{dt} (KN/m ²) =	154.2497		f _{dt} (KN/m ²) =	99.68743		
f _m (KN/m ²) =	4138.2		f _m (KN/m ²) =	4138.2		
6 =	1		6 =	1		
QCE:			QCE:			
V _{dt} (KN) =	168.7961		V _{dt} (KN) =	54.19874		
V _{tc} (KN) =	48.6082	Controls	V _{tc} (KN) =	23.46686	Controls	
PCL (KN)	1307.135	Eq-7.7	PCL (KN)	1307.135	Eq-7.7	

Pier 2			Pier 6			
Length (m) =	1.2192		Length (m) =	1.2192		<i>Reference: Fema 356 Clause 7.4.2.2.2</i>
heff (m) =	1.524		heff (m) =	1.524		
An(m ²) =	0.371612		An(m ²) =	0.371612		
f _a (KN/m ²) =	109.9533		f _a (KN/m ²) =	36.8933		
PCL (KN) =	40.86		PCL (KN) =	13.71		
f _{dt} (KN/m ²) =	154.2443		f _{dt} (KN/m ²) =	99.68474		
f _m (KN/m ²) =	4138.2		f _m (KN/m ²) =	4138.2		
6 =	1		6 =	1		
QCE:			QCE:			
V _{dt} (KN) =	93.77109		V _{dt} (KN) =	54.20067		
V _{tc} (KN) =	55.9897	Controls	V _{tc} (KN) =	1502725	Controls	
PCL (KN)	1045.708	Eq-7.7	PCL (KN)	1045.708	Eq-7.7	

Pier 3			Pier 7			
Length (m) =	1.2192		Length (m) =	1.2192		Reference: Fema 356 Clause 7.4.2.2.2
heff (m) =	1.524		heff (m) =	1.524		
An(m ²) =	0.371612		An(m ²) =	0.371612		
f _a (KN/m ²) =	109.9533		f _a (KN/m ²) =	36.8933		
PCL (KN) =	40.86		PCL (KN) =	13.71		
f _{dt} (KN/m ²) =	154.2443		f _{dt} (KN/m ²) =	99.68474		
f _m (KN/m ²) =	4138.2		f _m (KN/m ²) =	4138.2		
6 =	1		6 =	1		
QCE:			QCE:			
V _{dt} (KN) =	93.77109		V _{dt} (KN) =	54.20067		
V _{tc} (KN) =	55.9897	Controls	V _{tc} (KN) =	1502725	Controls	
PCL (KN)	1045.708	Eq-7.7	PCL (KN)	1045.708	Eq-7.7	

Pier 4			Pier 8			
Length (m) =	1.524		Length (m) =	1.524		Reference: Fema 356 Clause 7.4.2.2.2
heff (m) =	1.524		heff (m) =	1.524		
An(m ²) =	0.464515		An(m ²) =	0.464515		
f _a (KN/m ²) =	109.9641		f _a (KN/m ²) =	36.87716		
PCL (KN) =	51.08		PCL (KN) =	17.13		
f _{dt} (KN/m ²) =	154.2497		f _{dt} (KN/m ²) =	99.68743		
f _m (KN/m ²) =	4138.2		f _m (KN/m ²) =	4138.2		
6 =	1		6 =	1		
QCE:			QCE:			
V _{dt} (KN) =	93.77559		V _{dt} (KN) =	54.19874		
V _{tc} (KN) =	87.49477	Controls	V _{tc} (KN) =	23.46686	Controls	
PCL (KN)	1307.135	Eq-7.7	PCL (KN)	1307.135	Eq-7.7	
Low Bound Storey Vdt (KN)= Strength (QCL)	450.1138		Low Bound Storey Vdt (KN)= Strength (QCL)	216.7988		
Vtc (KN) =	248.0824	Controls	Vtc (KN) =	76.98824	Controls	

Linear and Non linear Analysis

Determine period			
Diaphragm Span =	9.144	m	
Diaphragm Length =	9.144	m	
Diaphragm Thickness =	0.0254	m	
Diaphragm Mod =	10345.66	KN/m ²	
Diaphragm, I=	1.618308	m ⁴	
Floor Dead Load =	1.2	KN/m ²	
Inertial Diaphragm force =	100.3353	KN	
Max Diaphragm Displacement =	0.072806	m	2.8663761 in
Approximate Period T (S) =	0.47284	Ref: Fema 356 Eq. 3-9	
Tributory Weight of Building (KN)			
Floor One :	37.4625		
Floor Two :	37.4625		

Calculation Spectral Acceleration

BSE-1		BSE-2	
Sa	0.127	Sa	0.448

Calculate Pseudo Lateral Load

BSE-1			BSE-2		
Factors					
C1	1		C1	1	
C2	1		C2	1	
C3	1		C3	1	
Cm	1		Cm	1	
Pseudo Lateral Load (KN)					
Floor one (KN)		4.757738	Floor one (KN)		16.7832
Floor two (KN)		4.757738	Floor two (KN)		16.7832

Design Forces:

BSE-1		BSE-2	
Deformation Controlled		Deformation Controlled	
Storey Shear		Storey Shear	
QUD: Floor Two (KN)	4.757738	QUD: Floor Two (KN)	16.7832
QUD: Floor One (KN)	9.515475	QUD: Floor One (KN)	33.5664
Force Controlled		Force Controlled	
Factor	1	Factor	1.6
Storey Shear		Storey Shear	
QUD: Floor Two (KN)	4.757738	QUD: Floor Two (KN)	26.85312
QUD: Floor One (KN)	9.515475	QUD: Floor One (KN)	53.70624

Acceptance Criteria

BSE-1		BSE-2	
<i>Deformation Controlled</i>		<i>Deformation Controlled</i>	
Limit State: Performance Level: Knowledge Factor (k) m factor	Rocking IO 0.75 1	Limit State: Performance Level: Knowledge Factor (k) m factor	Rocking LS 0.75 3
mk QCE Floor Two (KN) Floor One (KN)	66.7278 154.7043	mk QCE Floor Two (KN) Floor One (KN)	200.1834 464.1129
<i>Force Controlled</i>		<i>Force Controlled</i>	
Limit State Toe Crushing Knowledge factor (K)	0.75	Limit State Toe Crushing Knowledge factor (K)	0.75
KQCL Floor Two (KN) Floor One (KN)	57.74118 186.0618	KQCL Floor Two (KN) Floor One (KN)	57.74118 186.0618

A.3. COMPUTER AIDED ANALYSIS

A local building in Kathmandu was chosen for the Analysis. The plan and elevation of the building is shown in Figure A-2.

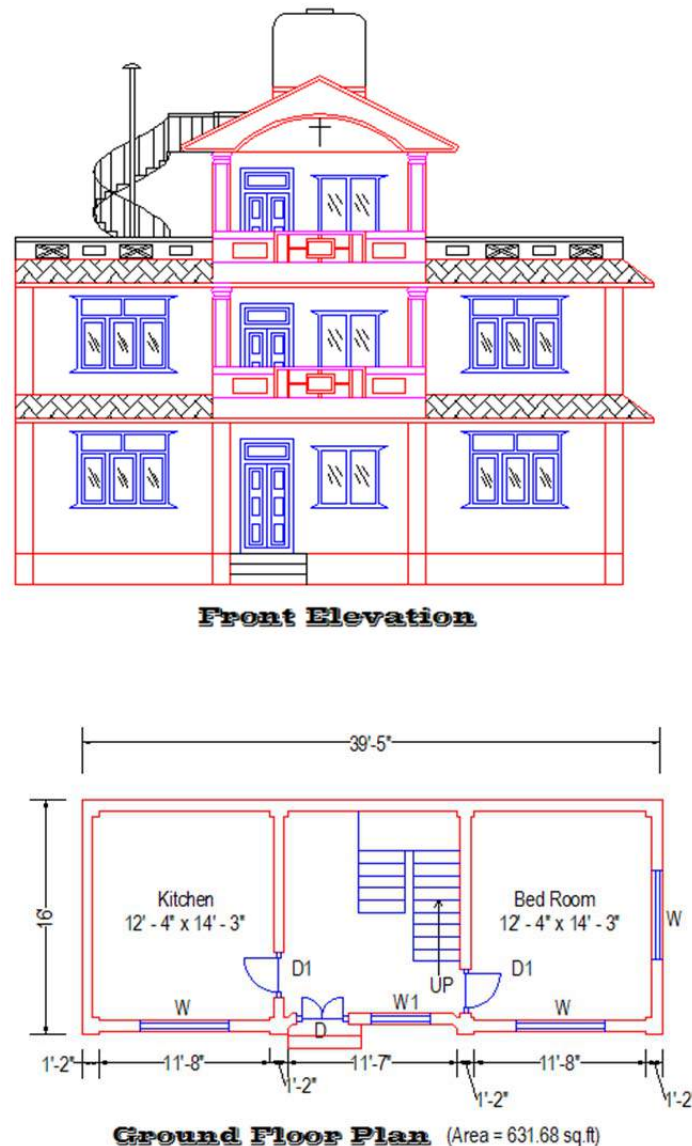


Figure A- 2: Elevation and Plan of Local Building.

The vertical structure is a single layer non retrofitted masonry made of brick masonry, while the floors are of concrete slabs. In order to reduce the computational burden of the dynamic analyses needed for the vulnerability assessment, only the facade wall was analyzed using the proposed SAP2000 v.14.0.0 model. The design values assumed for the mechanical properties are based on the mean values measured in brick masonry from different researches conducted in Pulchowk Campus, for brick masonry, $f_d=1.82\text{N/mm}^2$, $E=509\text{N/mm}^2$, and $G=203.6\text{N/mm}^2$. Only the in-plane seismic performance of the wall was investigated, assuming that the wall was effectively connected to the floors.

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

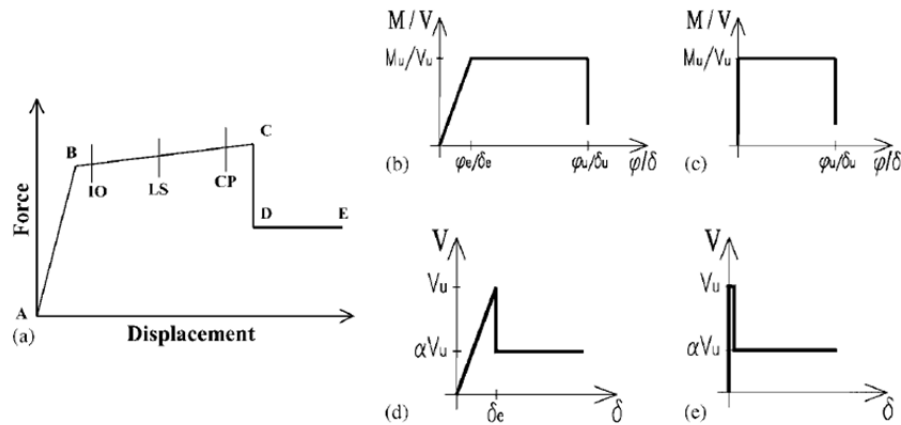


Figure A- 3: (a) Standard shape of the force vs displacement curve in SAP2000® v.10 for the plastic hinge element ; (b) and (c): behavior assumed, respectively, for the entire pier and the correspondent plastic hinge; (d) and (e): behavior assumed, respectively, for the entire spandrel beam and the correspondent plastic hinge.

The masonry pier was modeled as elasto-plastic. Hinges were determined according to the failure mechanisms of masonry. The standard force-displacement curve that can be implemented in the SAP 2000 plastic hinges is depicted in Figure A-3(a) . The masonry piers were modeled as elastoplastic with final brittle failure (Figure A- 3(b) by introducing two 'rocking hinges' at the end of the deformable parts and one 'shear hinge' at mid-height . A rigid- perfectly plastic behavior with final brittle failure was assumed for all these plastic hinges (Figure A- 3(c)).

A.3.1. Pushover Analysis

i. Without retrofitting

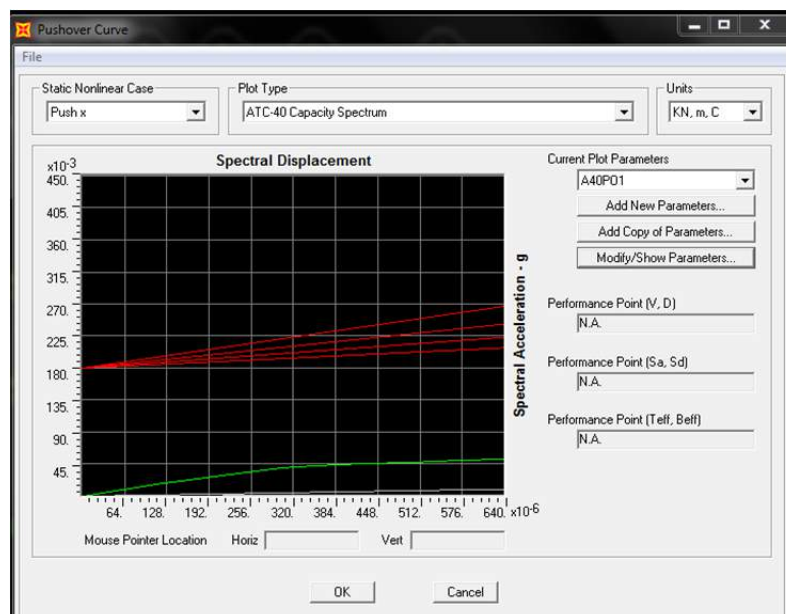


Figure A-4: Push Over Curve of Local Building without retrofitting, from SAP 2000 v 14.0.0 Analysis

As the model is assumed to be without lintel, both the shear hinges (at the middle) and flexural hinges (at the ends) are assigned in the spandrels. Base Shear calculated here for the building is 88.29 KN, but the structure seems to be collapsed at 25.553 KN base shear. No performance point is found for DBE ($C_a = 0.18$, $C_v = 0.3$). Pushover curve is shown in Figure A-4.

ii. Retrofitting with lintel

Only shear hinge is assigned in spandrels at the mid span while piers are modeled with two flexural hinges at the ends and one shear hinge at the mid span. Slight in base shear capacity was found but performance point was not found. Pushover curve is shown in Figure A-5.

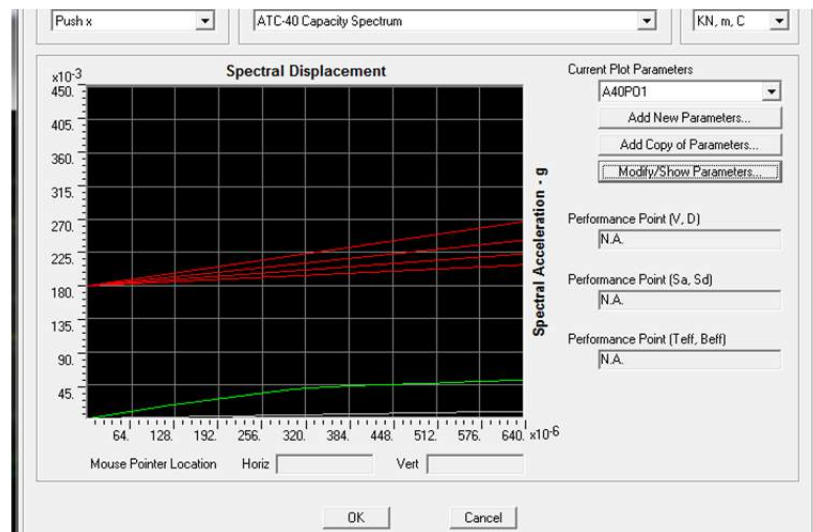


Figure A-5: Push over Curve of Local Building with lintel, from SAP 2000 v 14.0.0 Analysis

iii. Retrofitting with lintel + Rigid Diaphragm

Rigid diaphragm was assigned at the floor level but no increase in base shear capacity in found in in-plane analysis. No performance point was found. Pushover curve is shown in Figure A-6.

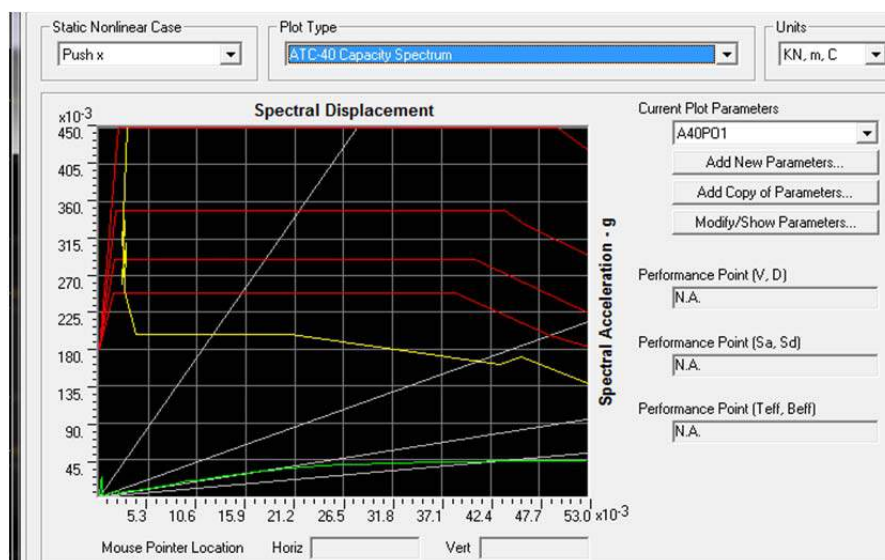


Figure A-6: Push over Curve of Local Building with lintel and rigid diaphragm, from SAP 2000 v 14.0.0 Analysis

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

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

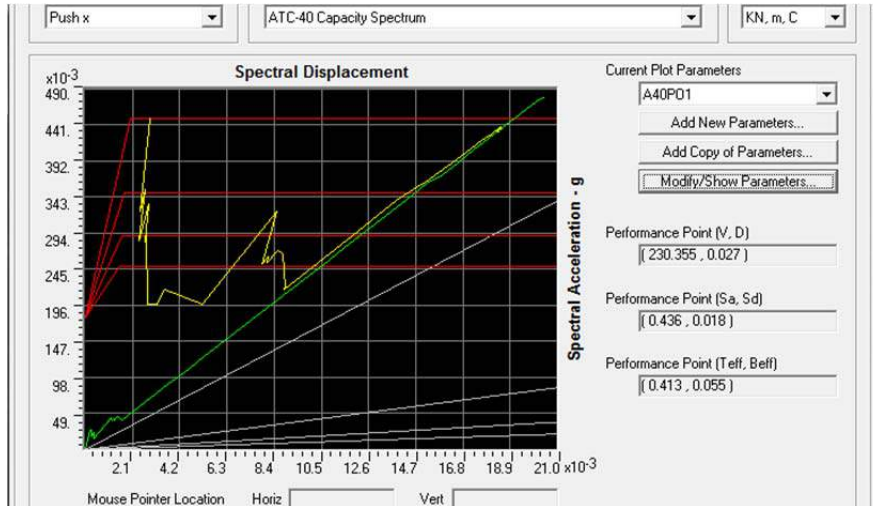


Figure A-7 Push over Curve of Local Building with lintel, rigid diaphragm and columns, from SAP 2000 v 14.0.0 Analysis

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

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

base shear = 88.289 kN, ($Z = 0.36, I = 1, S_a/g = 2.5, R = 3$)

Performance point of the structure with DBE ($C_a = 0.18, C_v = 0.3$) is (107 kN, 0.016m)

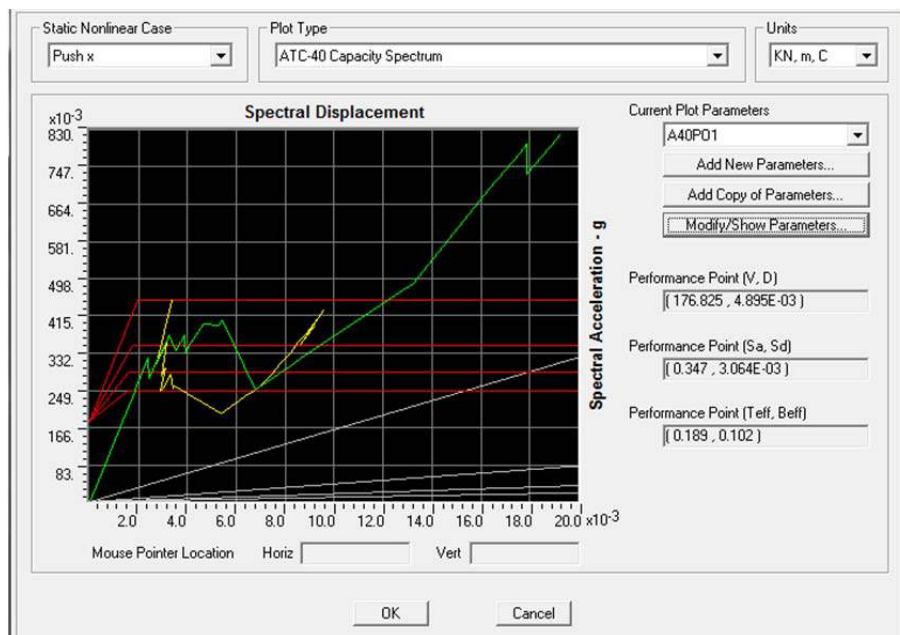
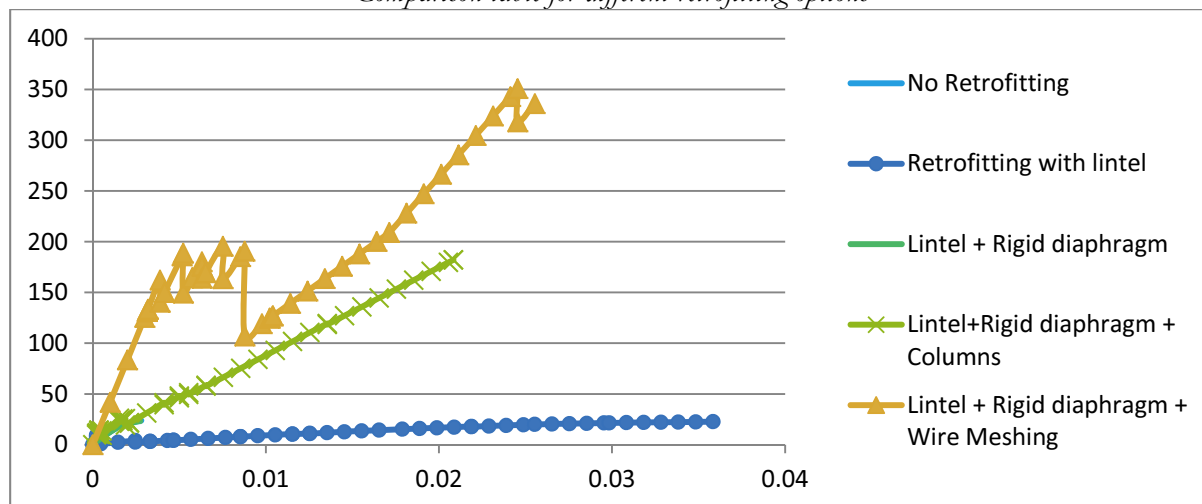


Figure A-8: Push over Curve of Local Building with lintel, rigid diaphragm and meshing, from SAP 2000 v 14.0.0 Analysis

Although the base shear capacity of the structure is lesser than while assigning RC columns, this option seems to be more reliable as in this case the masonry structure is covered with wire mesh which enable the masonry members to remain stable without the formation of the hinges approximately up to the performance point. Pushover curve is shown in Figure A-8.

Comparison table for different retrofitting options



A.3.2. CONCLUSION FROM IN PLANE ANALYSIS

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

A.4. STRENGTHENING OF WALLS USING GI WIRE (SAMPLE CALCULATION)

Gabion wire to strengthen masonry walls

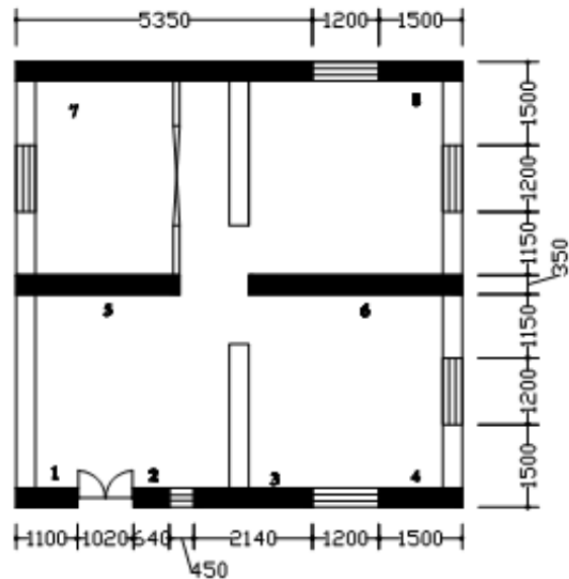


Figure A- 4: A sample masonry building

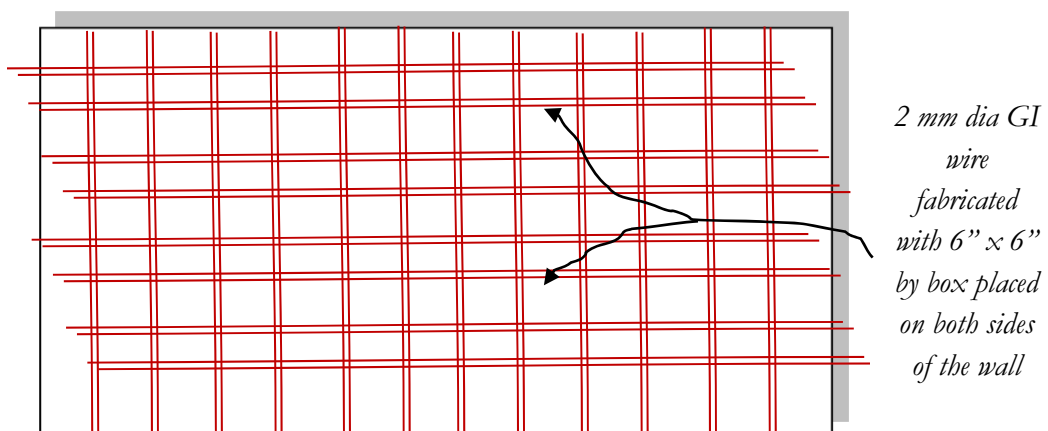
For a two storey building

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

If the outer wall is supposed to carry 1/3rd of the 1ststorey shear then shear in the wall is 100 KN. If the shear strength of the wall is assumed to be zero, the shear force has to be taken by GI wire placed on both sides of the wall.

Supposing 2.0 mm GI wire with x-sectional area of 3.1 mm² with 0.45KN/mm² strength³⁰, total number of wires required

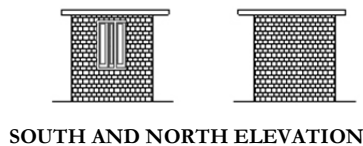
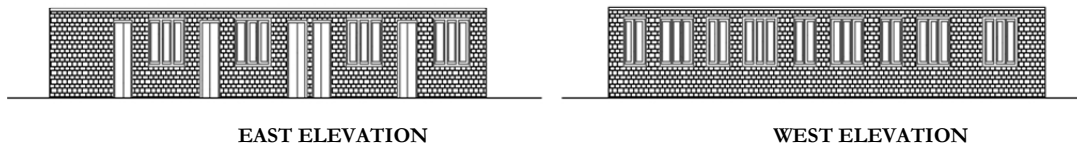
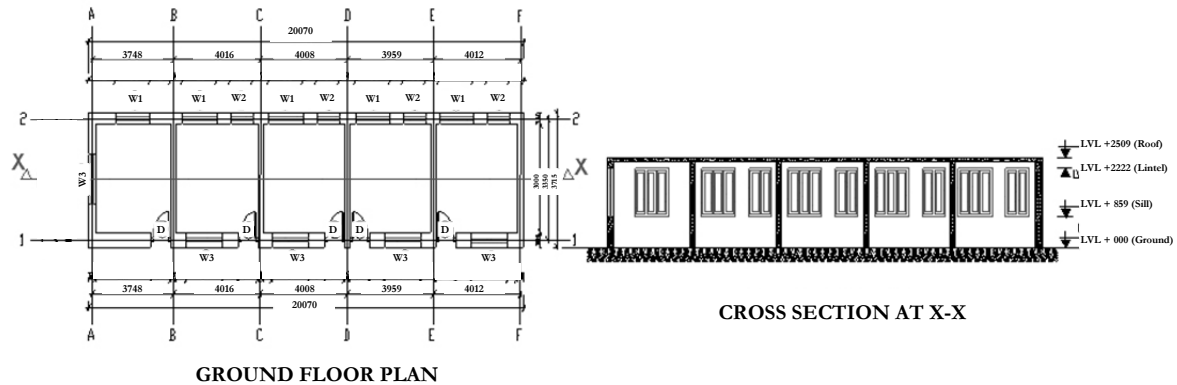
- = $100/3.1*0.45*0.75$ [25% strength reduction for knotting of the GI wire]
- = 100 wires to be placed equally on both faces of the wall i.e. 50 wires are required in each face of the wall which has to be placed in the length of 8000 mm
- = 6" x 6" GI wire mesh on both sides will provide sufficient shear capacity for the wall



³⁰ British standard wire gauge

A.5. RETROFIT DESIGN OF MASONRY BUILDING

Measured drawing of the building



OPENING SCHEDULE			
SN	DESCRIPTION	SYM	SIZE
1	WINDOW	W1	1575×1372
2	WINDOW	W2	1043×1372
3	WINDOW	W3	1677×1363
4	DOOR	D	906×1363

1. PARAMETERS

Seismic zone V	Z =	0.36	
Importance Factor	(I) =	1.5	
Soil type		III	
Response reduction factor	(R) =	1.5	
height of the building	h =	2.51	m
Dimension of the building along X	D _x =	19.74	m
Dimension of the building along Y	D _y =	3.37	m
Time Period along X	T _x =	0.051	Sec
Time Period along Y	T _y =	0.123	Sec
	(S _a /g) =	1.762	
	(S _a /g) =	2.500	
	(A _h) _x =	0.317	
	(A _h) _y =	0.450	

2. MASONRY PROPERTIES

Compressive Strength of brick	10	N/mm ²
Mortar Type	M1	C:S = 1:5
Basic compressive strength for masonry after 28 days (IS 1905:1987)	0.96	N/mm ²
Compressive strength of masonry (f_m)	3.84	N/mm ²
Young's modulus ($550 \cdot f_m$)	2112	N/mm ²
Poisson's ratio	0.15	
Unit Weight of Masonry	19	KN/m ³

Permissible Compressive Stress Calculation:

Permissible Compressive Stress

Compressive strength of masonry units =

Mortar type M1 corresponding to cement sand ratio of 1:5

Basic compressive strength of wall (f_b) = 0.96

Permissible compressive stress (f_c) = $f_b \times k_s \times k_a \times k_p$

$$\begin{array}{r} \text{wall thickness} = \\ \hline 230 \\ \hline 2510 \\ \hline \text{slenderness ratio} = \\ \hline 8.2 \end{array}$$

Stress reduction factor (k_s) = 0.92

Area reduction factor (k_a) = $0.7 + 1.5 A$, A being the area of section in m²

Area reduction factor (k_a), takes into consideration smallness of the sectional area of the elements and is applicable when sectional area of the element is less than 0.2 m².

width of the brick = 110 mm

height of the brick = 55 mm

Sectional Area (A) = $0.11 \cdot 0.055 = 0.00605 \text{ m}^2$

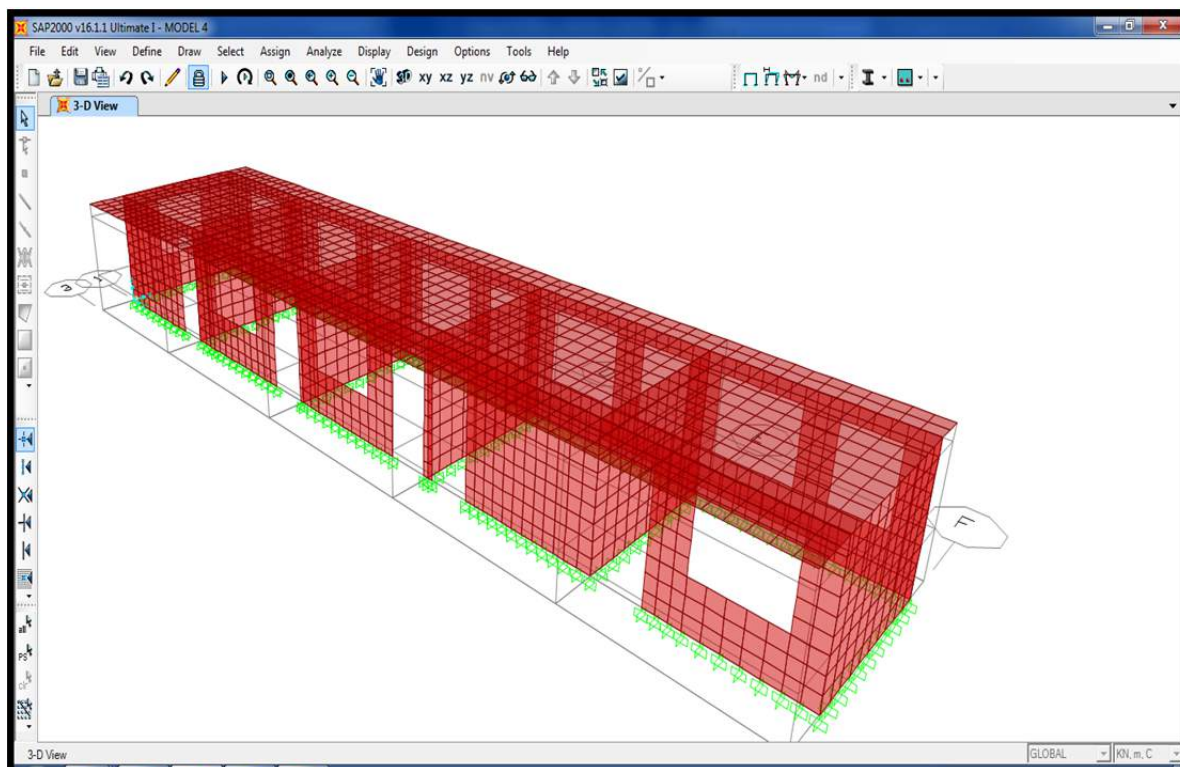
$k_a = 0.7 + 1.5 \cdot 0.00605 = 0.71$

Shape modification ratio (k_p) = 1

Hence, permissible compressive stress in Masonry (f_c) = 0.63 N/mm^2

3. 3D-FINITE ELEMENT MODEL OF THE BUILDING

Wall elements modeled as shell elements of specified thickness in Measured Drawing



4. ANALYSIS OUTPUT

The analysis was carried accordingly and the base reactions are summarized as below:

Base Shear

The output result from the analysis for Base Shear is illustrated in table below.

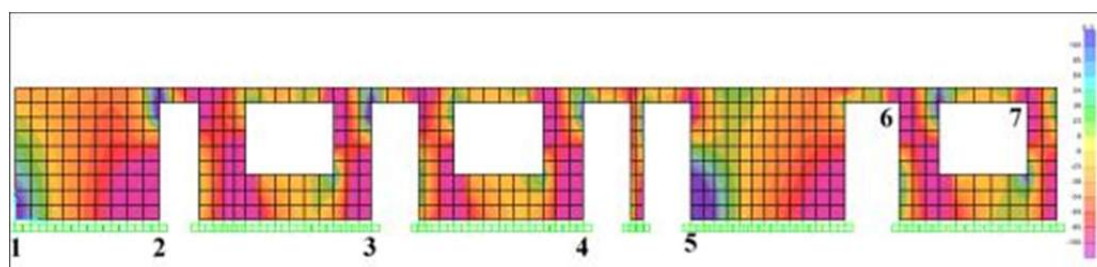
Output Case	Case Type	Global FX	Global FY	Global FZ			
Text	Text	KN	KN	KN			
LL	Lin Static	0	0	186.702			
EQx	Lin Static	-919.707	0	0			
EQy	Lin Static	0	-919.707	0			
TLL	Lin Static	0	0	85.481			
DL-ALL	Combination	0	0	1801.335	<i>(Including floor finishes etc)</i>		
seismic weight =		1916.0563	KN				
Base shear=		919.707	KN				

In Plane and Out of Plane Stresses

The results of in plane stresses, out plane moments and shear to be carried by each of the walls as illustrated and tabulated below:

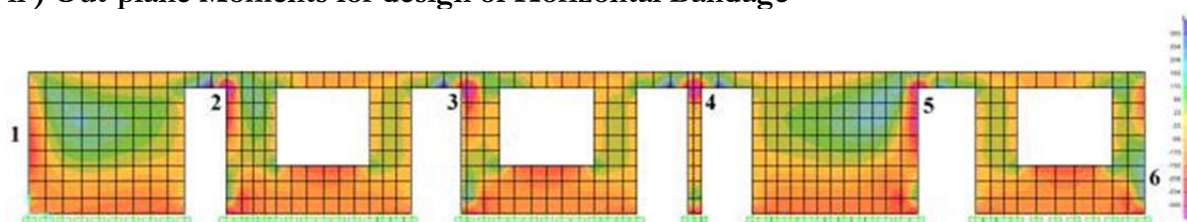
For Wall 1

i) In-plane stresses for design of Vertical Splints



Load Combination	Point ID	Stress(S22) Mpa	Remarks
0.7DL+EQ _x	1	0.125	T
	2	-0.422	C
	3	-0.276	C
	4	-0.293	C
	5	0.322	T
	6	-0.237	C
	7	-0.2	C

ii) Out-plane Moments for design of Horizontal Bandage



Load Combination	Point ID	Moment(M11) KN-m
0.7DL+EQ _y	1	0.26
	2	0.778
	3	0.594
	4	0.779
	5	0.572
	6	0.254

The total shear carried by the wall to resist force along X-direction, for load combination EQ_x is equal to 168.45 KN (From SAP).

1. DESIGN OF RETROFITTING MEASURES

Wire-mesh Retrofitting Options-Capacity Calculation

Option 1

(Using gauge 15 wire for retrofitting)

out-of-plane bending resistance of the wall with GI wire mesh retrofitting	1,829 mm@	20 mm c/c
wire mesh grade=	Fe	250
No. of horizontal wires per m strip =		51
Area of each wire=		2,627 mm ²
Total area of wires=		133,99465 mm ²
Yield Strength of wires=		250 N/mm ²
Allowable Strength of wires=		140 N/mm ²
Permissible increase in strength of wires=		33%
Total allowable tensile force= $1.33 \times 140 \times 133.994653013614 / 1000$	=	24.95 KN
Allowable compressive strength in masonry =		0.630 N/mm ²

Capacity of the 9" wall with GI wire mesh jacketing for out-of-plane bending per m strip wall thickness	= 9 inch
thickness of jacketing=	25 mm
overall depth= 25+230+25 =	280 mm
Leaving 10 mm clear cover on either side	
Effective depth= $280 - 2 \times 10 =$	= 260 mm

Applying condition for the equilibrium

Neutral axis for the triangular distribution of the stress from one face=	= $24.95 \times 1000 / (0.5 \times 1000 \times 0.63)$	= 79.210 mm
Lever arm= $260 - 79.21 / 3$		= 233.60 mm

Moment of Resistance per m strip	= $24.95 \times 233.6 / 1000$	5.828 KN-m
Moment of Resistance per 0.4 m strip=		2.331 KN-m
Moment of Resistance per 0.6 m strip=		3.497 KN-m
Moment of Resistance per 0.8 m strip=		4.663 KN-m

Summary

Allowable Tensile Strength of the wall of all wires per m strip=	24.95 KN
For(9") 230 mm wall, Wall Area per m strip=	230000 mm ²
Allowable tensile Stress in 230 mm wall=	0.11 N/mm ²

Option 2

out-of-plane bending resistance of the wall with Tor steel retrofitting	4.75 mm@	150 mm c/c
wire mesh grade=	Fe	415
No. of horizontal wires per m strip =		8
Area of each wire=		17,721 mm ²
Total area of wires=		141,77 mm ²
Yield Strength of wires=		415 N/mm ²
Allowable Strength of wires=		232.4 N/mm ²
Permissible increase in strength of wires=		33%

$$\begin{aligned} \text{Total allowable tensile force} &= 1.33 \times 232.4 \times 141.77 / 1000 = 43.82 \text{ KN} \\ \text{Allowable compressive strength in masonry} &= 0.630 \text{ N/mm}^2 \\ \text{Capacity of the 9" wall with GI wire mesh jacketing for out-of-plane} & \\ \text{bending per m strip wall thickness} &= 9 \text{ inch} \\ \text{thickness of jacketing} &= 25 \text{ mm} \\ \text{overall depth} &= 25 + 230 + 25 = 280 \text{ mm} \\ \text{Leaving 10 mm clear cover on either side} & \\ \text{Effective depth} &= 280 - 2 \times 10 = 260 \text{ mm} \end{aligned}$$

Applying condition for the equilibrium

$$\begin{aligned} \text{Neutral axis for the triangular distribution of the stress from one face} &= \\ &= 43.82 \times 1000 / (0.5 \times 1000 \times 0.63) = 139.120 \text{ mm} \\ \text{Lever arm} &= 260 - 139.12 / 3 = 213.630 \text{ mm} \end{aligned}$$

$$\begin{aligned} \text{Moment of Resistance per m strip} &= 24.95 \times 213.63 / 1000 = 9.361 \text{ KN-m} \\ \text{Moment of Resistance per } 0.4 \text{ m strip} &= 3.745 \text{ KN-m} \\ \text{Moment of Resistance per } 0.6 \text{ m strip} &= 5.617 \text{ KN-m} \\ \text{Moment of Resistance per } 0.8 \text{ m strip} &= 7.489 \text{ KN-m} \end{aligned}$$

Summary

$$\begin{aligned} \text{Allowable Tensile Strength of the wall of all wires per m strip} &= 43.82 \text{ KN} \\ \text{For 230 mm wall, Wall Area per m strip} &= 230000 \text{ mm}^2 \\ \text{Allowable tensile Stress in 230 mm wall} &= 0.19 \text{ N/mm}^2 \end{aligned}$$

Summary

$$\begin{aligned} \text{Option 1} & \quad 1.829 \text{ mm@} \quad 20 \text{ mm c/c} \quad \text{Fe 250} \\ \text{Option 2} & \quad 4.75 \text{ mm@} \quad 150 \text{ mm c/c} \quad \text{Fe415} \end{aligned}$$

Mesh option	Jacketing Strip width(m)	Moment of Resistance(KN-m) Wall Thickness(mm)
OPTION 1		230
	1	5.828
	0.4	2.331
	0.6	3.497
	0.8	4.663
OPTION 2	1	9.361
	0.4	3.745
	0.6	5.617
	0.8	7.489

Mesh Option	Tensile Strength per face N/mm ² Wall thickness(mm)	Shear Strength per face KN/m Wall thickness(mm)
	230	230
Option 1	0.108	24.95
Option 2	0.191	43.82

i) Design for Vertical bands

Wall ID 1

Vertical Tensile and compressive stress, S22, 0.7DL+Eq_x loading

Maximum Tension due to pier action= 0.322 N/mm²

Maximum Compression due to pier action= 0.422 N/mm²

From SAP Analysis

Length of Tensile Stress Zone in Pier = 800 mm

Thickness of the wall= 230 mm

Therefore, Average tensile Strength= $0.322/2 = 0.161$ N/mm²

Total tensile force = $0.161*800*230/1000 = 29.624$ KN

Considering Option 1

1.829mm@20mm c/c, fy=250 Mpa wire mesh in vertical bands

from both the inner and outer face of the wall

Vertical bands are provided for pier ends and openings on both the sides.

Tensile Strength per m strip on both the faces= $0.109*2 = 0.217$ N/mm²

Tensile Strength of 1m wide strip with both faces having vertical bands=

$1000*0.217*230/1000 = 49.91$ KN

Required width of the vertical band= $29.624/49.91$ m =593.548387 mm

Therefore, use 1.829mm@20mm c/c, fy=250 Mpa wire mesh in vertical bands of width 590 mm at both the inner and outer faces of the wall

Considering Option 2

4.75mm@150mm c/c, fy=415 Mpa tor-steel in vertical bands from both the inner and outer face of the wall. Vertical bands are provided for pier ends and openings on both the sides.

Tensile Strength per m strip on both the faces= $0.191*2 = 0.381$ N/mm²

Tensile Strength of 1m wide strip with both faces having vertical bands=

$1000*0.381*230/1000 = 87.63$ KN

Required width of the vertical band= $29.624/87.63$ m =338.057743 mm

Therefore,

use 4.75mm@150mm c/c, fy=415 Mpa tor-steel in vertical bands of width 350 mm at both the inner and outer faces of the wall.

Proposed Nos. of 4.75mm tor-steel required at junction = 4

Use 4- 4.75mm dia Tor steel in 150mm vertical bands at both the inner and outer face of the wall.

Equivalent Number of 10mm dia bars = 2

Use 2-Nos. of 10-mm dia bars in 150mm vertical band width of band= 150

ii) Out-of-plane bending Moment distribution for Horizontal Strip

Wall ID 1 for load combination 0.7DL+Eq_y

Maximum bending moment intensity in wall= 0.779 KN-m/m

Distance between maximum and minimum intensity= 0.8 m

Bending Moment= $0.779*0.8 = 0.6232$ KN-m

Considering Option 1

1.829mm@20mm c/c, fy=250 Mpa wire mesh in horizontal bands on both inner and outer faces of the wall.

Moment of Resistance of the 1m band strip = 5.828 KN-m/m

$$\begin{aligned} \text{Width of the inside band for resisting bending moment} &= 0.6232 \text{ KN-m/m} \\ &= 110 \text{ mm} \\ \text{Thus band size adopted} &= 150 \text{ mm} \end{aligned}$$

Thus, use 1.829 mm@20mm c/c, $f_y=250$ wire mesh of 150 mm horizontal band on both inner and outer faces of the wall.

Considering Option 2

4.75mm@150mm c/c, $f_y=415$ Mpa wire mesh in horizontal bands on both inner and outer faces of the wall.

$$\begin{aligned} \text{Moment of Resistance of the 1m band strip} &= 9.361 \text{ KN-m/m} \\ \text{Width of the inside band for resisting bending moment} &= 0.6232 \text{ KN-m} = 70 \text{ mm} \\ \text{Thus band size adopted} &= 100 \text{ mm} \end{aligned}$$

Thus, use 4.75mm@150mm c/c, $f_y=415$ wire mesh of 100 mm horizontal band on both inner and outer faces of the wall.

$$\begin{aligned} \text{No. of 4.75mm dia bar in the band} &= 2 \\ \text{Equivalent Nos. of 10mm-dia bars in horizontal band} &= 2 \text{ (Nominal)} \\ \text{Use 2 -10 mm dia bars in 150mm horizontal band in both inner and outer faces of the wall.} \\ \text{width of band} &= 150 \end{aligned}$$

iii) Check for Shear Stress

Wall ID 1 for load combination EQ_x

$$\begin{aligned} \text{Total shear force in the wall} &= 168.45 \text{ KN} \\ \text{Length of the wall} &= 19.47 \text{ m} \\ \text{Thickness of the wall} &= 230 \text{ mm} \\ \text{Shear stress in the wall} &= 0.0376164 \text{ N/mm}^2 \\ \text{Assuming total shear force to be resisted by the wire mesh in horizontal and vertical bands.} \end{aligned}$$

Considering Option 1

$$\begin{aligned} &1.829\text{mm@}20\text{mm c/c, } f_y=250 \text{ wire mesh vertical band in both inner and outer faces of the wall.} \\ \text{shear strength per meter of the vertical bands} &= 24.95 \text{ KN/m} \\ \text{shear strength of 590 mm splint band in inner face} &= 14.7205 \text{ KN} \\ \text{Number of vertical bands along the inner face of the wall} &= 6 \\ \text{Total shear strength of outer band and inner vertical bands} &= 19.47*24.95+14.7205*6 \\ &= 574.0995 \\ \text{Minimum expected shear strength of masonry} &= 0.1 \text{ N/mm}^2 \\ \text{Total shear strength of masonry unit} &= 447.81 \text{ KN} \\ \text{Now, ratio of shear strength to shear force} &= (574.0995+447.81)/168.45 \\ &= 6.07 \quad \text{ok} \end{aligned}$$

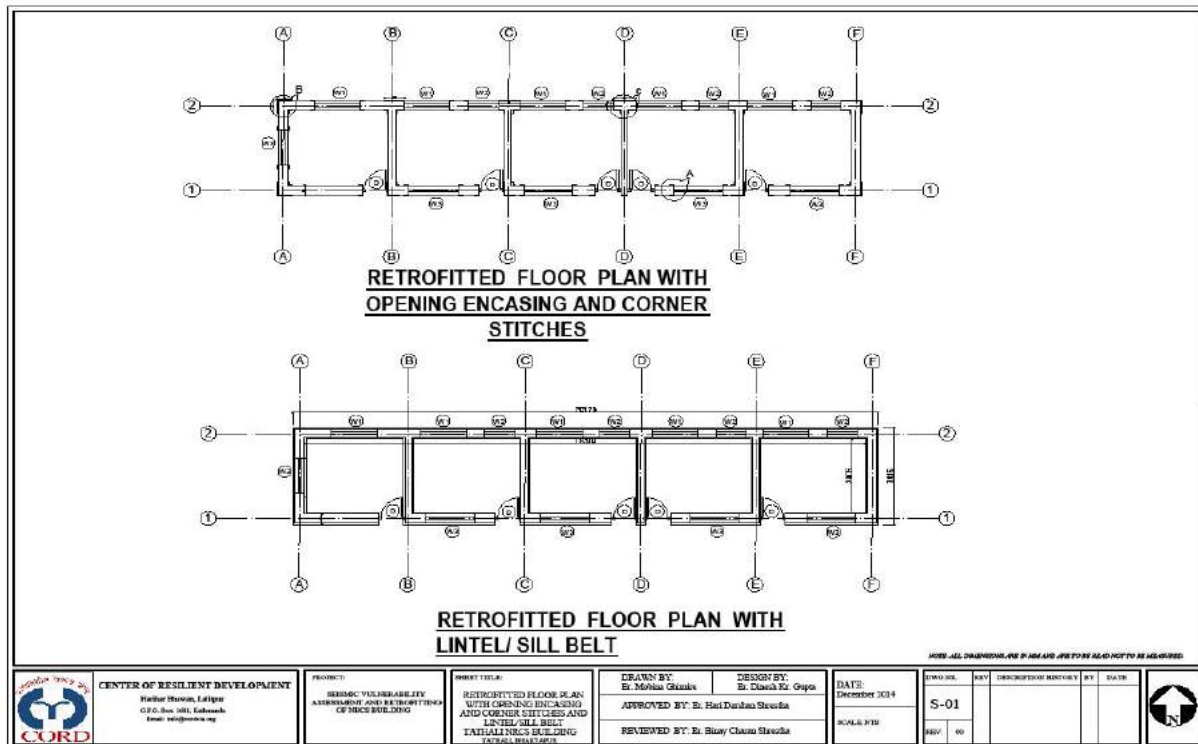
Considering Option 2

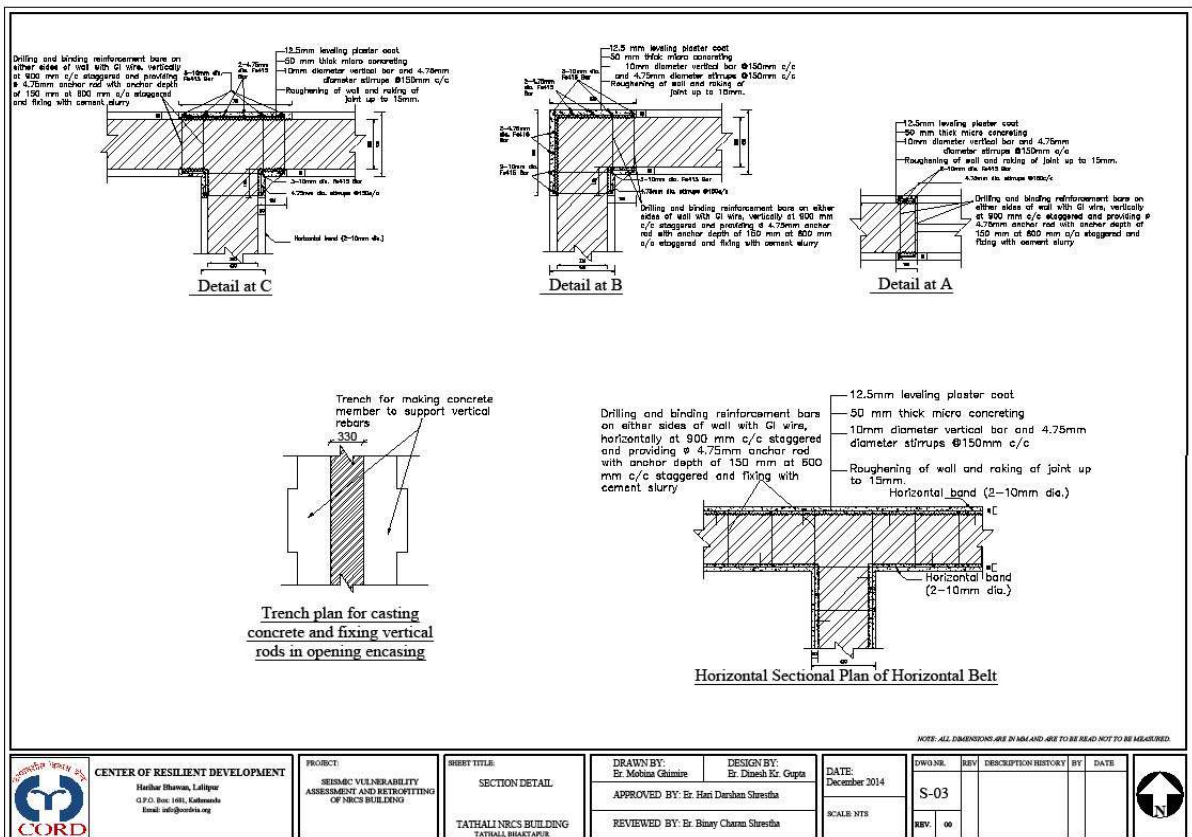
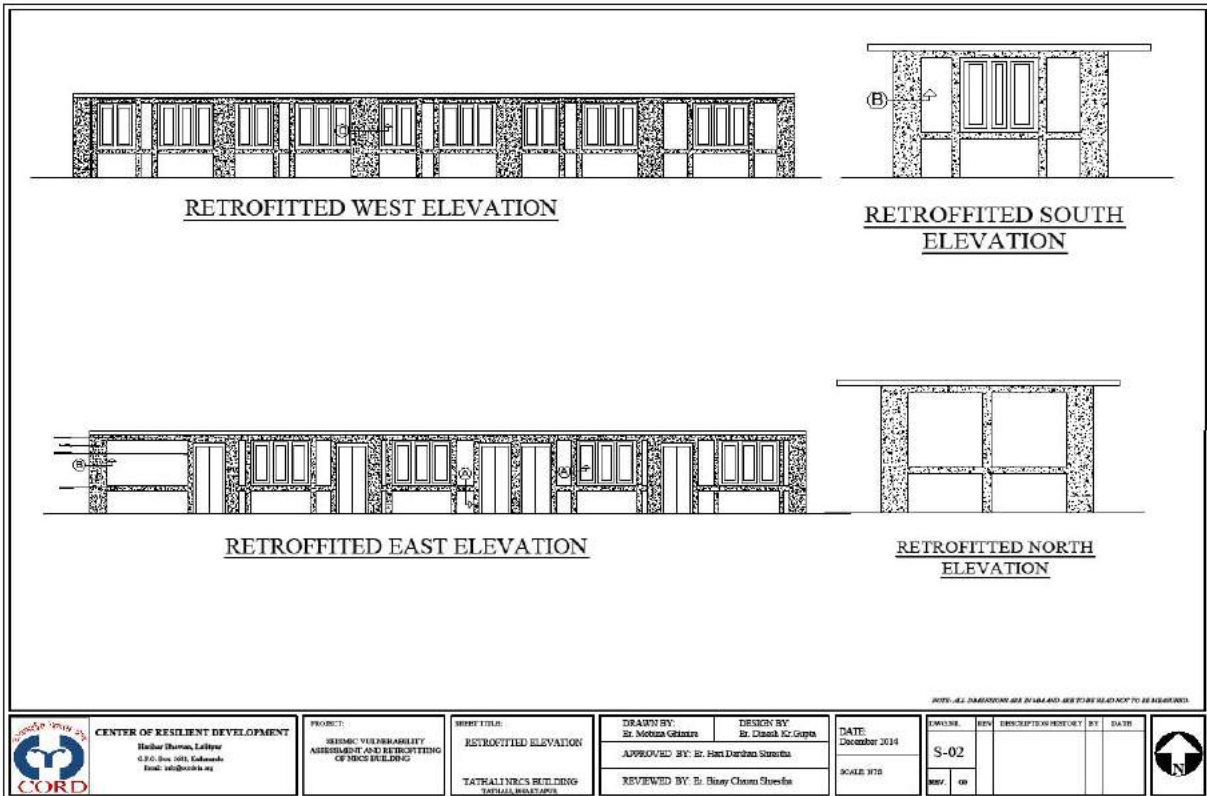
$$\begin{aligned} &4.75\text{mm@}150\text{mm c/c, } f_y=415 \text{ wire mesh vertical band in both inner and outer faces of the wall.} \\ \text{shear strength per meter of the vertical bands} &= 43.82 \text{ KN/m} \\ \text{shear strength of 350 mm splint band in inner face} &= 15.337 \text{ KN} \\ \text{Number of vertical bands along the inner face of the wall} &= 10 \\ \text{Total shear strength of outer band and inner vertical bands} &= 19.47*43.82+15.337*10 \\ &= 1006.5454 \\ \text{Minimum expected shear strength of masonry} &= 0.1 \text{ N/mm}^2 \\ \text{Total shear strength of masonry unit} &= 447.81 \text{ KN} \\ \text{Now, ratio of shear strength to shear force} &= (1006.5454+447.81)/168.45 \\ &= 8.63 \quad \text{ok} \end{aligned}$$

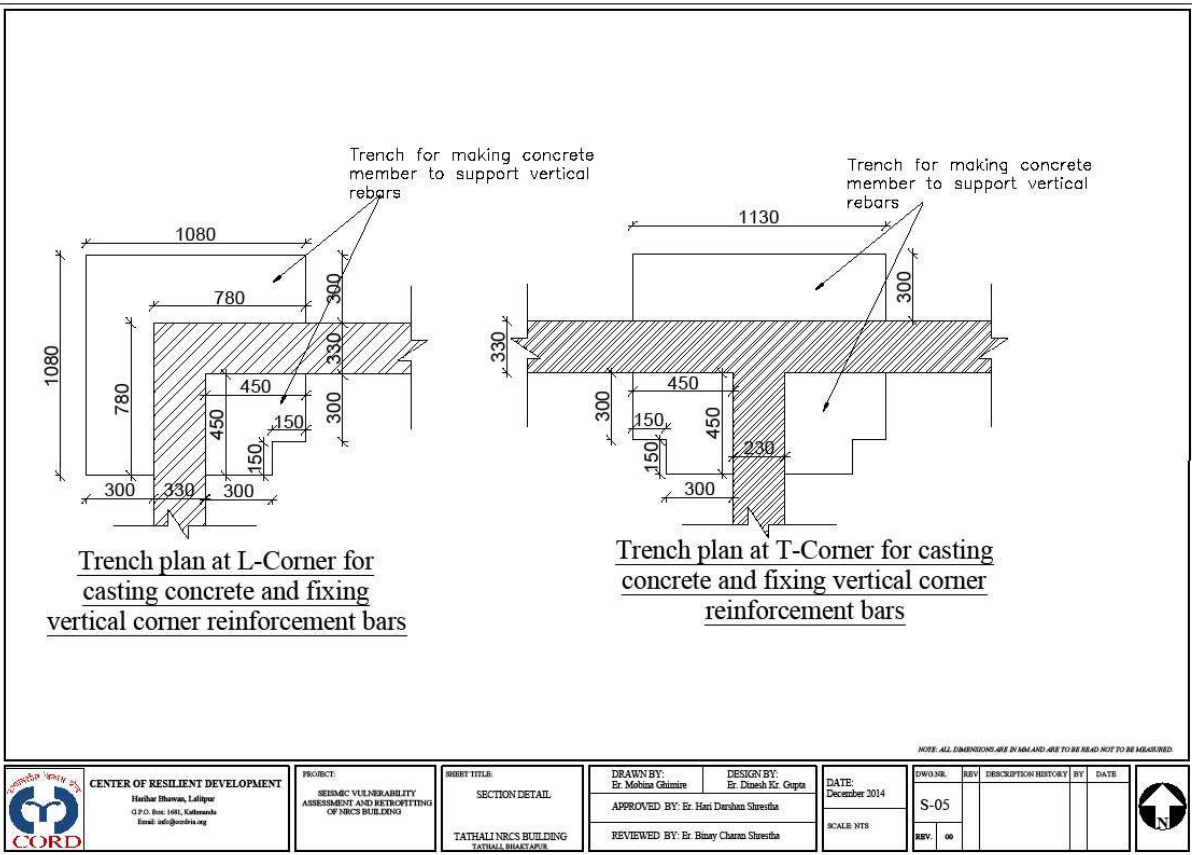
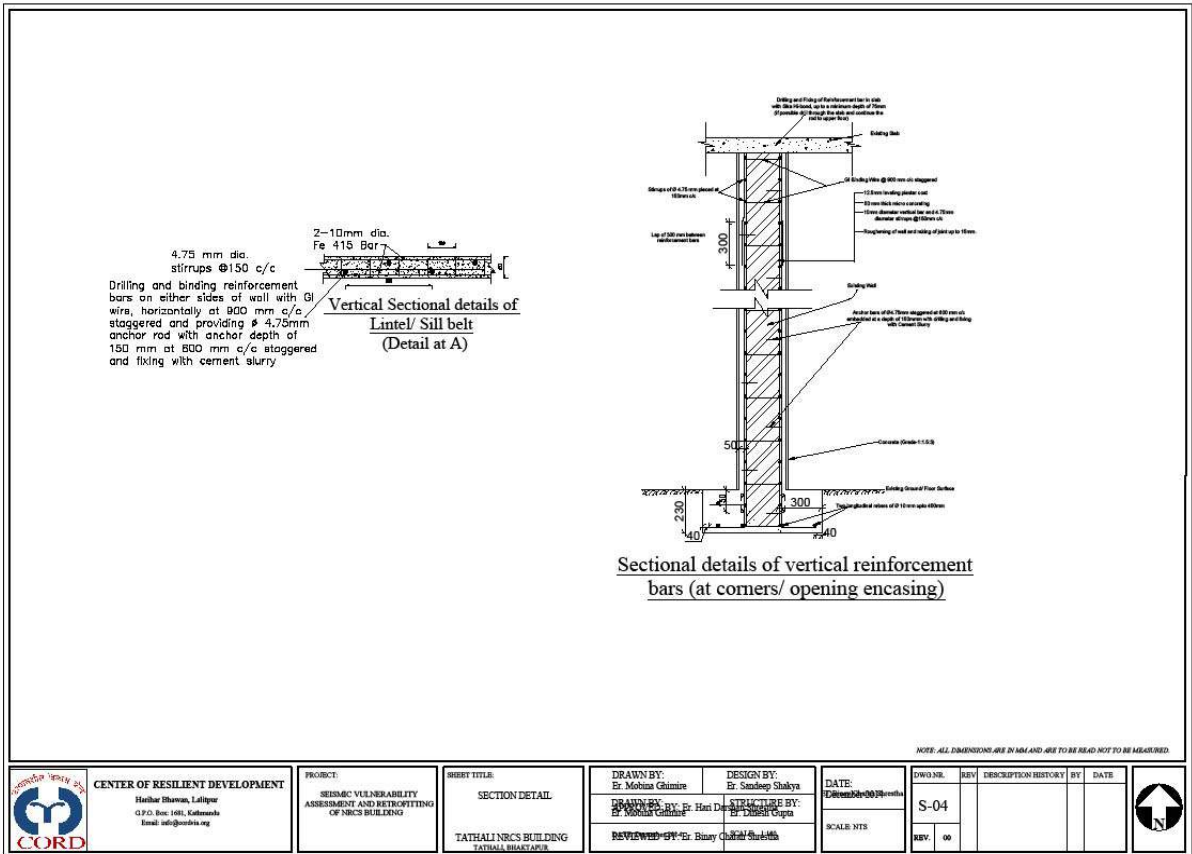
2. DESIGN OUTPUT SUMMARY

Wall ID	Option 1		Option 2	
	Vertical Band	Horizontal Band	Vertical Band	Horizontal Band
1	size=590 mm	size=150 mm	size=150 mm	size=150 mm
	1.829mm@20mm c/c, fy=250	1.829mm@20mm c/c, fy=250	Bar: 2 Nos. -10 mm dia	Bar: 2 Nos. -10 mm dia
2	size=300 mm	size=100 mm	size=150	size=150
	1.829mm@20mm c/c, fy=250	1.829mm@20mm c/c, fy=250	Bar: 2 Nos. -10 mm dia	Bar: 2 Nos. -10 mm dia
A	size=390 mm	size=100 mm	size=150	size=150
	1.829mm@20mm c/c, fy=250	1.829mm@20mm c/c, fy=250	Bar: 2 Nos. -10 mm dia	Bar: 2 Nos. -10 mm dia
B	size=240 mm	size=100 mm	size=150	size=150
	1.829mm@20mm c/c, fy=250	1.829mm@20mm c/c, fy=250	Bar: 2 Nos. -10 mm dia	Bar: 2 Nos. -10 mm dia
C	size=190 mm	size=100 mm	size=150	size=150
	1.829mm@20mm c/c, fy=250	1.829mm@20mm c/c, fy=250	Bar: 2 Nos. -10 mm dia	Bar: 2 Nos. -10 mm dia
D	size=310 mm	size=100 mm	size=150	size=150
	1.829mm@20mm c/c, fy=250	1.829mm@20mm c/c, fy=250	Bar: 2 Nos. -10 mm dia	Bar: 2 Nos. -10 mm dia
E	size=170 mm	size=100 mm	size=150	size=150
	1.829mm@20mm c/c, fy=250	1.829mm@20mm c/c, fy=250	Bar: 2 Nos. -10 mm dia	Bar: 2 Nos. -10 mm dia
F	size=210 mm	size=100 mm	size=150	size=150
	1.829mm@20mm c/c, fy=250	1.829mm@20mm c/c, fy=250	Bar: 2 Nos. -10 mm dia	Bar: 2 Nos. -10 mm dia

3. RETROFIT DRAWING





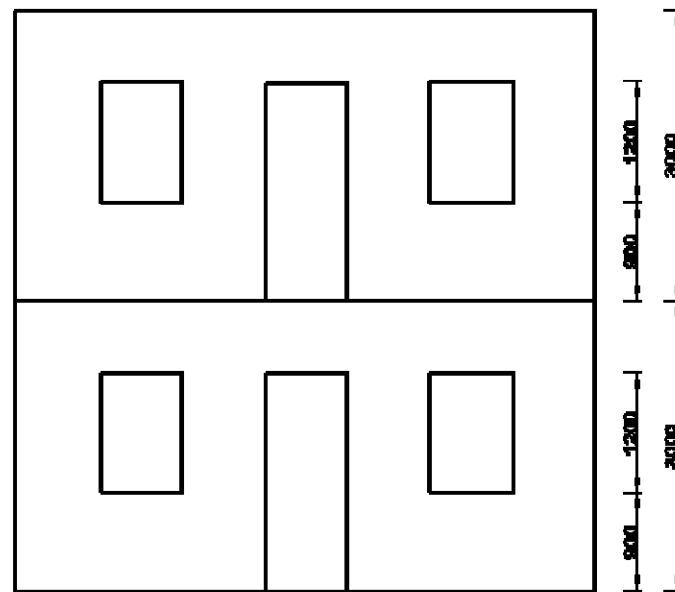


ANNEX B – CASE STUDIES

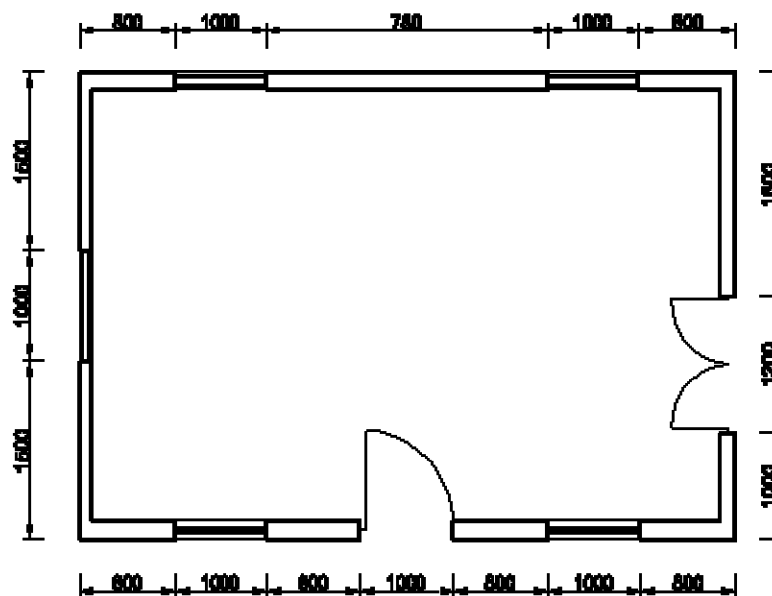
CASE STUDY - 1

B.1 ANALYSIS AND DESIGN OF A RESIDENTIAL BUILDING - STRENGTH BASED METHOD

For the case study a building was chosen which has been already retrofitted. The strength based calculation steps and procedure are shown below:



FRONT ELEVATION



PLAN

B.1.1 BUILDING DESCRIPTION

No. of stories	=	2	
Size	L=	6200 mm	6.2 m
	B=	4000 mm	4 m
	H=	6000 mm	6 m
Wall material=		Brick in cement sand mortar (1:6)	
Thickness of wall =		230 mm	0.23 m
Roof material =		R.C.C	
Thickness of roof=		125 mm	0.125 m
Earthquake Zone=		V	
Building Type=		School	
Window:	B=	1000 mm	1 m
	H=	1200 mm	1.2 m
No. of windows=		5	Per floor
Door:			
D1	B=	1200 mm	1.2 m
	H=	2100 mm	2.1 m
D2	B=	1000 mm	1 m
	H=	2100 mm	2.1 m

Dead load and live loads:

Dead Load

Unit wt. of R.C.C=	25 KN/m ³
Unit wt. of brick masonry=	19 KN/m ³
Floor Finishing=	1 KN/m ²

Live Load

Live Load for floor=	3 KN/m ²
Live load for roof=	0 KN/m ²

Earthquake Loads

Summary of lumped load calculation:

As per IS 1893 (Part 1):2002, cl 7.3, table 8:

For imposed uniformly distributed floor loads up to 3KN/m², % of imposed load =25%

Storey	Dead Load, DL (KN)	Live Load, LL (KN)	Seismic Weight, W _i =DL+25%LL (KN)	Story Height	Storey Level, h _i (m)
2	192.3436	0	192.3	3	6
1	298.5346	74.4	317.1	3	3
∑	490.8782		509.5		

Calculation Of base shear:

IS 1893:2002, Cl. 7.5.3 Design Seismic Base shear

Seismic Zone		Cl. 6.4.2, Table 2	V
Seismic Zone factor	Z	Cl. 6.4.2, Table 2	0.36
Structure type		Cl. 6.4.2, Table 6	School Building
Importance factor	I	Cl. 6.4.2, Table 6	1.5
Lateral load resisting system		Cl. 6.4.2, Table 7	Retrofitted masonry
Response reduction factor	R	Cl. 6.4.2, Table 7	2.5
Height of the building	h	Refer drawing	6 m
Dimension of the building Along X	D_x	Refer drawing	6.2 m
Dimension of the building Along Y	D_y	Refer drawing	4 m
Time period of the building along X,	T_x	Cl. 7.6.2, $T_x = \frac{0.09h}{\sqrt{d_x}}$	0.217 Sec.
Time period of the building along Y	T_y	Cl. 7.6.2, $T_y = \frac{0.09h}{\sqrt{d_y}}$	0.2700 Sec.
Soil type			Medium Soil
Average Response acceleration coefficients along X	$\left(\frac{S_a}{g}\right)_x$	Cl. 6.4.5, fig. 2	2.5
Average Response acceleration coefficients along Y	$\left(\frac{S_a}{g}\right)_y$	Cl. 6.4.5, fig. 2	2.5
Design Horizontal Seismic Coefficient	A_h	Cl. 6.4.2, $A_h = \frac{ZIS_a}{2Rg}$	0.27
Seismic Wt of the Building	W	Cl. 7.4.1	509.5 KN
Base Shear	V_B	Cl. 7.5.3, $V_B = A_h W$	137.6 KN

Distribution of Lateral Forces at different storey:
on Indian Seismic Code IS 1893(Part 1), Cl. 7.7.1

$$\text{Design lateral force at floor } i \ (Q_i) = V_B \left(\frac{W_i h_i^k}{\sum W_i h_i^k} \right) = V_B \left(\frac{w_i h_j^k}{\sum w_i h_j^k} \right)$$

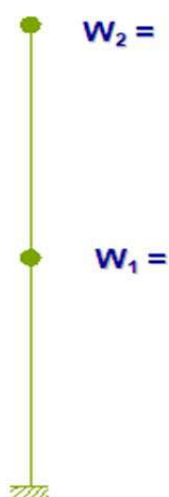
$$T = 0.216869219 \text{ Sec.}$$

$$= 0.216869219$$

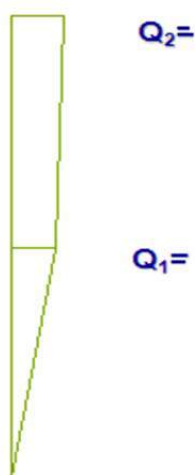
$$\text{Hence, } K = 1$$

Storey	Seismic wt. W_i (kN)	Storey level h_i (m)	$W_i h_i^k$ KNm	Design Lateral Force Q_i (KN)	Storey Shear V_j (KN)
2	192.3436	6	1154	75.4	75.4
1	317.1346	3	951	62.159	137.6
			2105	137.56	

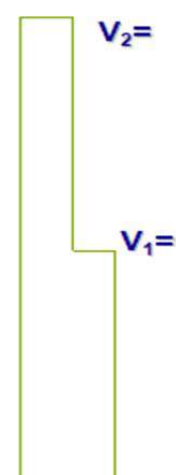
Lateral Coefficients:



Lumped Weights



Floor Level Force



Shear Force

Storey	Seismic wt. W_i (kN)	Design Lateral Force Q_i (KN)	Lateral Coefficient —	Remarks
2	192.3436	75.39981004	0.39	$C_i > A_h$
1	317.1346	62.15930396	0.20	$C_i > A_h$

Out of plane analysis and design of bandage(Lintel Band)

Effective length of wall:

Length of wall, $L = 5.97 \text{ m}$

Load Carried by bandage

$$\begin{aligned} \text{Wt. of tributary volume} &= (y \times t \times \text{th} \times 1) \\ q &= C \times (\text{wt. of tributary wall of unit length}) = 1.713 \text{ KN/m} \\ M &= \frac{ql^2}{10} = 6.106 \text{ KN/m} \end{aligned}$$

Design of bandage

Assumed size of band:

$d = 250 \text{ mm}$

$t = 50 \text{ mm}$

Lever arm = $z = 225 \text{ mm}$

f_s of steel = $0.56 \times 1.25 \times f_y = 290.5 \text{ N/mm}^2$

$$A_{st} = \frac{M}{(f_s \times z)} = 9E-05 \text{ m}^2 = 93.41$$

Dia of rods used = 8

No. of rods reqd. = 1.8575877

= 2

A provided = 100.5714286

% of reinforced in single band = $(A_{st}/(d \times t)) \times 100 = 0.805\%$

Check for shear

Shear force in band = $v = q \times L/2 + (M_1 + M_2)/l = 7.159 \text{ KN}$

Considering all shear carried by band,

Induced shear stress = $\frac{v}{2td} = 0.286356016 \text{ N/mm}^2$

From Table 19 of IS 456:2000

Permissible shear stress in concrete (M20) = 0.36 Mpa

Remarks: *Chosen section is safe in shear, Hence Ok*

Check for anchorage

Area of steel one band, $A_{st} = 100.5714286 \text{ mm}^2$

Interface bonding force = $T = C = A_{st} \times 0.56 \times f_v = 23372.8 \text{ N}$

Wall length, $l = 5970 \text{ mm}$

Band depth, $d = 250 \text{ mm}$

Induced shear stress in band and wall interface = $T / (\text{wall length} \times \text{Band depth}) = 0.016 \text{ N/mm}^2$

Assume minimum bond stress between concrete band and brick masonry = 0.1 N/mm²

So, the induced shear stress is less than the minimum bond stress

However, bond failure is a brittle kind of failure which is not desirable in earthquake resistant construction.

So, Provided dia of anchor = 4.75 mm from band to wall

Shearing area of the anchor = Area of provided bar = $A = 17.73 \text{ mm}^2$

Allowable shearing stress, $f_s = 0.4f_v = 166 \text{ N/mm}^2$
 Shear resistance per anchor, $F = A \times f_s = 2942.794643 \text{ N}$
 No. of anchor rods reqd., $N = T/F = 7.942382271 \approx 8$
 Spacing between anchors, $S = \text{Length of band}/(N-1) = 852.9 \text{ mm}$
 So, use 4.75 mm dia at a spacing of 852.857 c/c

Check for vertical bending below lintel band:

Lateral Load:

Considering $b = 1 \text{ m}$ width of wall.
 Lateral Load, $w = C \times (\text{Wt. of wall of "b" height}) = 1.71306542 \text{ KN/m}$
 Height of wall below lintel band = 1.2 m

$$M = \frac{wl^2}{12} = 0.20556785 \text{ KN/m}$$

$$Z = \frac{bt^2}{6} = 8816666.667$$

$$\text{Bending stress, } f_b = \frac{M}{Z} = 0.023 \text{ N/mm}^2$$

Vertical load on wall at mid height of wall below lintel band:

$$\text{Trapezoidal load on long wall} = (L \times B/2 - B^2/4) = \left(L \times \frac{B}{2} - \frac{B^2}{4} \right) = 8.40 \text{ m}^2$$

$$\text{Triangular load on short wall} = (0.5 \times B^2/4) = 0.5 \times \frac{B^2}{4} = 4.00 \text{ m}^2$$

$$\text{Vertical load, } P = \text{Wt. of wall} + \text{Slab} + \text{Finishing} = 75.291 \text{ KN}$$

$$\text{Vertical Stress, } f_a = P/A = 0.052798738 \text{ N/mm}^2$$

Check of combined stress:

$$\text{Combined Stress} = f_a + f_b = 0.076 \text{ N/mm}^2$$

$$f_a - f_b = -0.029 \text{ N/mm}^2$$

$$\text{Permissible tensile bending stress} = -0.07 \text{ Mpa}$$

Remark: *No, tension reinforcement required*

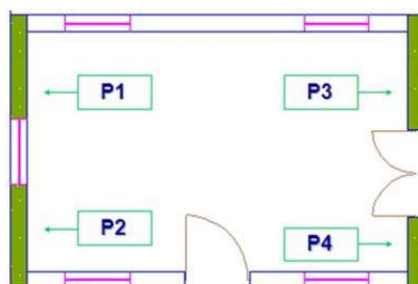
Design of stitches:

Lateral Load carried by stitch, $w = C \times (\text{Wt. of triangular portion of wall}) = 3.854 \text{ KN}$
 Count lintel and sill also as stitch band; therefore total number of stitch considered = 3

$$A_{st} = \frac{w}{(\text{No. of stitches} \times 0.56 \times f_y)} = 5.528 \text{ mm}^2$$

Since, for detailing requirement, we shall use 2 numbers of 8mm dia bars
 (One on outside face and another in the inside face of the wall).

In-Plane analysis and design of splint (Vertical Band):



Width of P ₁ =	1.5 m
Width of P ₂ =	1.5 m
Width of P ₃ =	1.8 m
Width of P ₄ =	1 m
Width of opening in grid 2, between P ₃ and P ₄ =	1.2 m
Width of opening in grid 1, between P ₁ and P ₂ =	1 m

Pier analysis:

Let us assume P₁ and P₂ along Grid 1 and P₃ and P₄ along grid 2
Analyzing piers P₃ and P₄ of first floor:

Lateral load carried by piers:

We have, V_B= 137.559114 KN (From calculation of base shear)

Lateral load on each wall (grid 1 and grid 2) $V_i = \frac{V_B}{\text{Number of walls}} = 68.78 \text{ KN}$

Masonry, E = 2400000 KN/m²

Pier	Height h (m)	Depth d (m)	Width b (m)	I- $\frac{bd^3}{12}$ (m ⁴)
P ₄	2.1	1.00	0.23	0.01917
P ₃	2.1	1.8	0.23	0.11178
				$\Sigma K =$

Stiffness (KN/m)	Proportion of lateral load carried by pier	Lateral load carried by pier
$K = \frac{12EI}{\left[1 + 2.4 \left(\frac{d}{h}\right)^2\right] h^3}$	$P = \frac{K}{\Sigma K}$	$F_i = V_i \times P$
38598.699	0.235	16.15
125798.692	0.765	52.63
164397.391		68.78

Bending stress in pier(fb):

Pier	Moment M = F×h/2 (KNm)	Z = bd ² /6 m ³	f _b = M/Z MPa
P ₄	16.96	0.038333333	0.44
P ₃	55.26	0.1242	0.44

Overturning stress (f_0):

Lateral load $Q_1 = 62.15930396$ KN

$Q_2 = 75.39981004$ KN

Overturning moment $= M_0 = Q_2/2 * (h_1 + h_2) + Q_1/2 * h_1 = 319.44$ KNm

Centroid of pier $P_3 \& P_4 = \frac{(A_3 X_3 + A_4 X_4)}{(A_3 + A_4)} = 1.914$ m

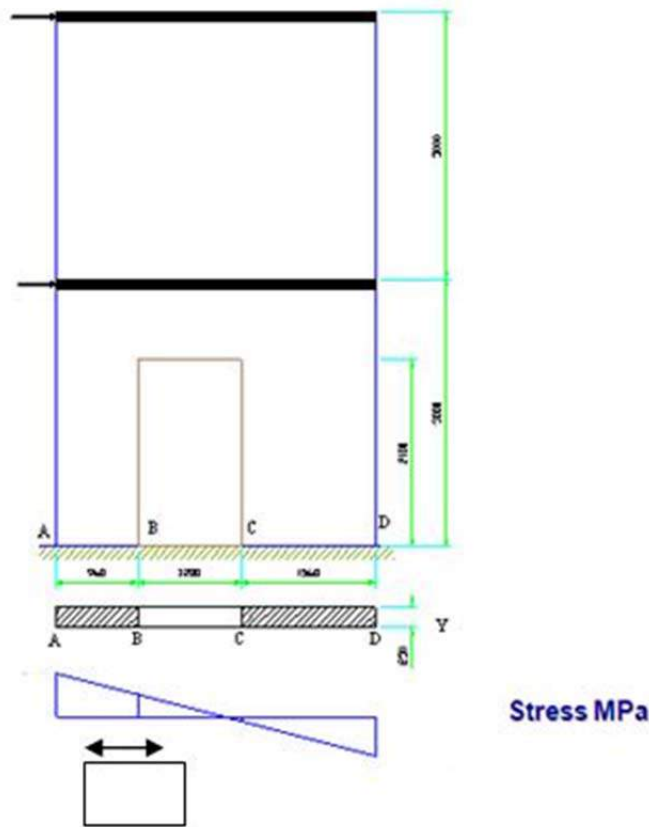
$h_3 = 1.186$ m

$h_4 = -0.914$ m

where h_3 is the distance between the centroid of P3 and centroid of the grid 2.

M.O.I about centroid, $I_c = I_3 + A_3 h_3^2 + I_4 + A_4 h_4^2 = 9.05258E+11$ mm⁴

Overturning stress at different piers



Points	y (mm)	$f_0 = M_0 y / I_c$ (MPa)
A	1914.29	0.68
B	914.2857143	0.32
C	285.7142857	0.10
D	2085.714286	0.74

Note: $y =$ distance of the points from centroid

Vertical stress (f_a):

Roof Slab = Triangular load \times (Thickness of slab \times unit wt of rcc + Floor finish) = 165KN

Ground Floor slab,

(Triangular area \times ((thickness of slab \times unit wt. of rcc + Floor finish) + LL) = 28.5KN

Wall = Length of the wall \times H \times thickness \times γ -h \times height of wall below lintel \times thickness \times γ = 93.8676KN

Total vertical load = 138.868 KN

Area = 0.644 m²

$$\text{Vertical Stress, } f_a = \frac{\text{Vertical load}}{\text{Area}} = 215.633 \text{ KN/m}^2$$

$$= 0.21563 \text{ N/mm}^2$$

Points	Bending stress f_b (MPa)	Overturning stress f_0 (MPa)	Vertical stress f_a (MPa)	Net Stress F_n (MPa)
A	0.44	0.68	-0.216	0.90
B	-0.44	0.32	-0.216	-0.34
C	0.44	-0.10	-0.216	0.13
D	-0.44	-0.74	-0.216	-1.40

Design of pier P4

$$\text{Distance of NA from point A, } x = 0.729024286 \text{ m}$$

$$= 729.0242864 \text{ mm}$$

$$\text{Total tensile force, } T = f_n A = 75637.9962 \text{ N}$$

$$A_{st, reqd} = \frac{T}{(0.56 \times f_y)} = 325.4646997 \text{ mm}^2$$

$$\text{Dia. of rod provided} = 10 \text{ mm}$$

$$\text{NO. of rods required} = 4.142277996 \approx 5$$

Check for Shear

$$\text{Shear force, } V = F_4 = 16.15 \text{ KN}$$

$$\text{Shear stress} = \frac{V}{A} = 0.070211668 \text{ N/mm}^2$$

Where, f_d = compressive shear stress = $f_a = 0.215632919$

$$\text{From IS 1905, 5.4.3, Permissible shear stress} = 0.1 + \frac{f_d}{6} = 0.14 \text{ N/mm}^2$$

Remarks: Safe

CASE STUDY: 2

B.2 A ANALYSIS AND RETROFITTING DESIGN OF SDN PADASUKA II SCHOOL BUILDING

B.2.1 BUILDING DESCRIPTION

A UNCRD project, with technical assistance from CDM-ITB, SDN padasuka II is located at KecamatanSoreang, Bandung County. The school has approximately 400 students. The school building consists of 2 buildings with four rooms each, and the total area of the school building is approximately 500 m². The structural system before retrofitted is reinforced concrete frames and masonry walls. The buildings were built in the early of 1990s, and still in the expected life-time.



Figure B- 1: Layouts and Existing Conditions of SDN Padasuka II Prior to Retrofitting

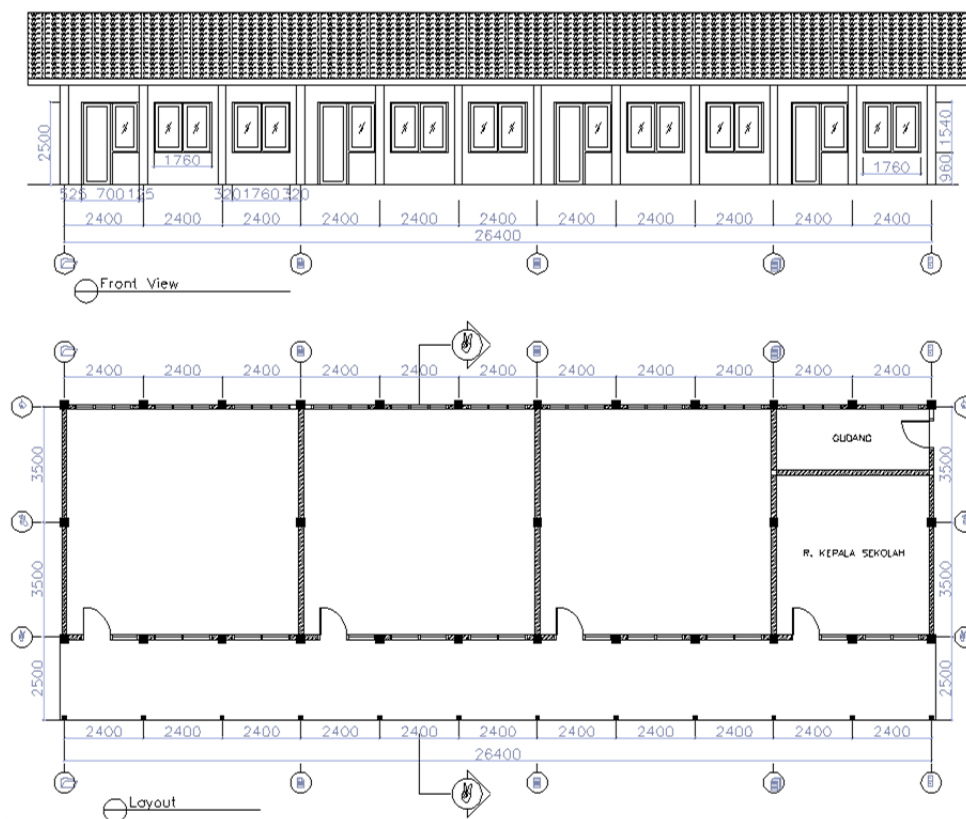


Figure B- 2: Drawings of the existing school building

B.2.2 BACKGROUND

a. Condition of Existing Structure

Many problems were found when the visual survey and structural investigation were conducted. The problems found could be listed as follows:

1. Inadequate foundation system (shallow foundation, no tie beam). The foundation system was exposed on some places and no support was provided on areas with eroded soil surface.
2. Inadequate roof framing system, poor wall-roof connection, and poor roof truss element and connection. The roof was in dire need of repair as the construction shown excessive deformation on the top of the building.
3. Damages found on walls, with cracks and gaps found on some places, due to lack of structural elements (beams and columns).
4. Poor materials and detailing on the structures.

1. Poor sanitation facilities.



Figure B- 3: Existing Condition of SDN Padasuka II

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

B.2.3. VULNERABILITY ASSESSMENT

Structural Analysis

Using the results from structural investigation, the buildings were analyzed to evaluate the structural performance under all applicable loads, including earthquakes. The structure was modeled as such that the structural elements (beams and columns) formed the structural frame, while masonry walls were modeled as plate elements. Both structural frame and plate elements provided lateral resisting system for the structure. The foundation provided support for the structural frames. The roof trusses were also modeled using truss elements. Material properties used for the analysis were based on results of structural investigation. The structural deficiencies/weaknesses were also included in developing the structural model.

The design criteria followed the performance based design, where the structure was expected to have minor/limited damage under design earthquake (elastic behavior). The seismic design level was obtained from the current building codes, with a p_g of 0.24g. The structural analysis was conducted using response spectra approach.

Results from structural analysis show that the structure did not have adequate capacity in resisting lateral loads. Checking of connection capacity also revealed unsatisfactory results. Moreover, the trusses required improvement to be able to support all applicable loads. Combined with data obtained from visual and structural investigations, results from structural analysis were then used to design appropriate retrofitting approach.

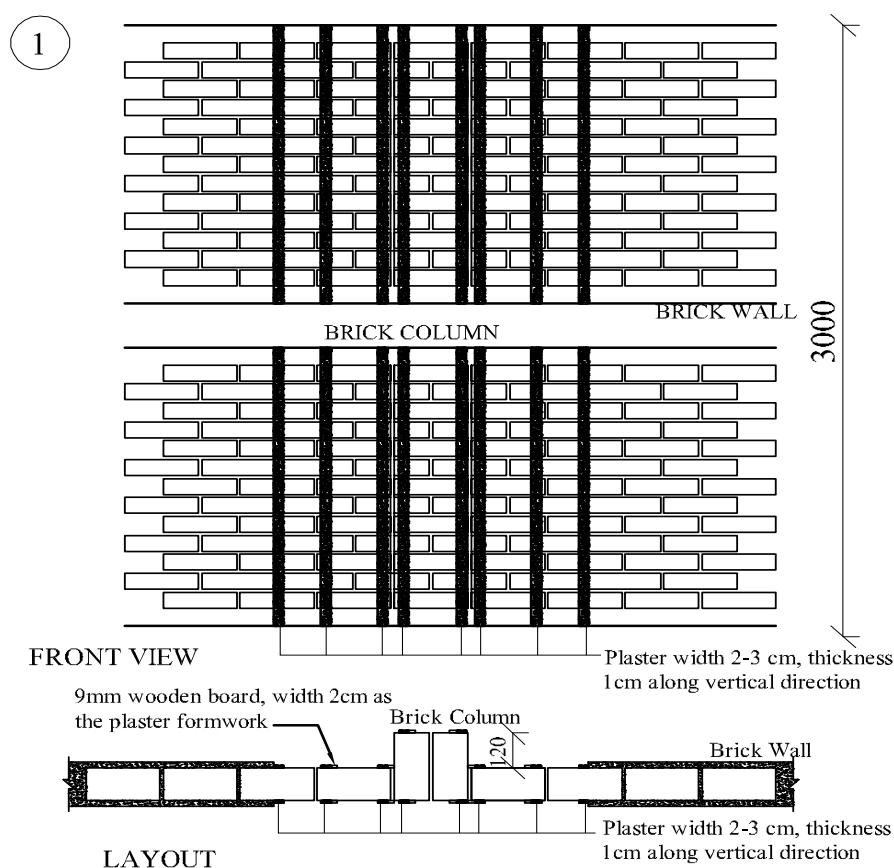
B.2.4 RETROFITTING DESIGN

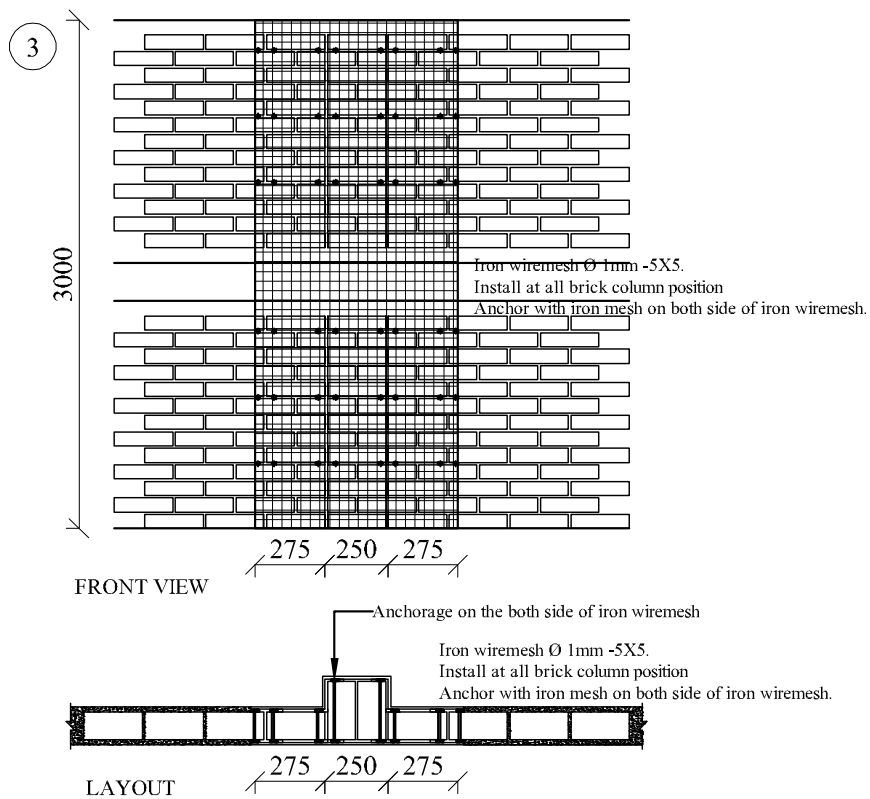
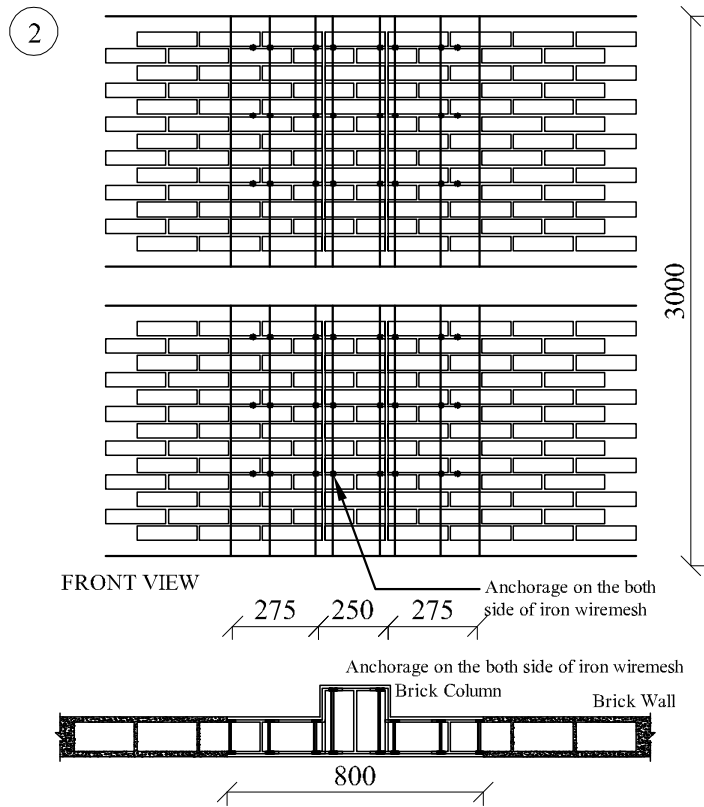
B.2.4.1 RETROFITTING APPROACH

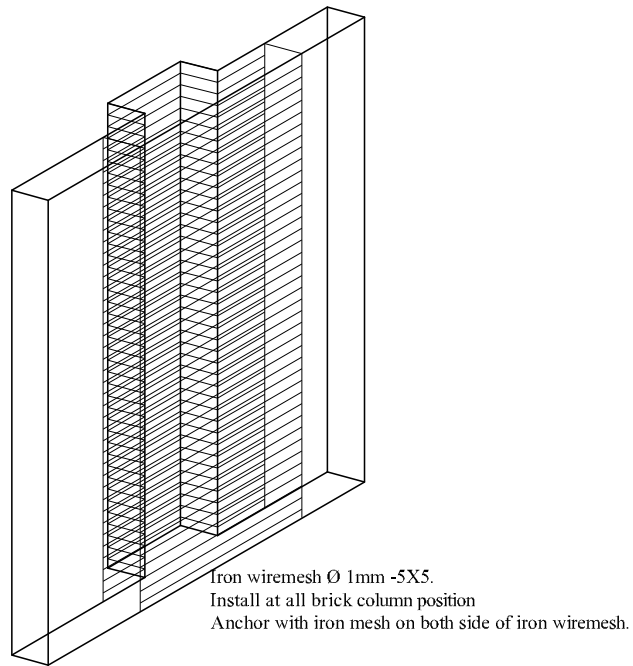
The buildings were retrofitted using iron wire mesh for strengthening wall elements, inserting columns on building corners. The wire mesh was installed in the locations of ordinary beams and columns, as well as diagonally on the perimeter walls. The wire mesh was installed on the both side of the wall and anchored using iron wire. Tie beams were added underneath the walls for strengthening the foundation system. The roof system was retrofitted using proper material and detailing. Repair was conducted for nonstructural elements such as doors/windows and ceilings. Finishing/cosmetic repair and improvement of sanitary facilities were also conducted for the schools buildings.

(a) *Retrofitting strategy for columns of SDN Padasuka II*

Detail of Column with Iron Wiremesh Reinforcement



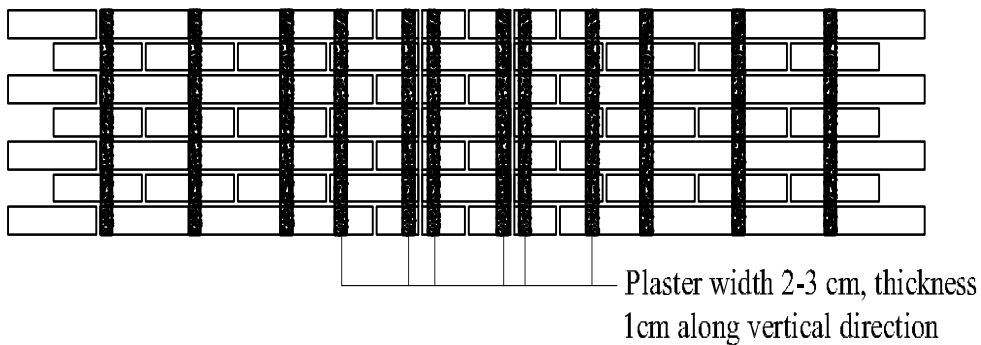
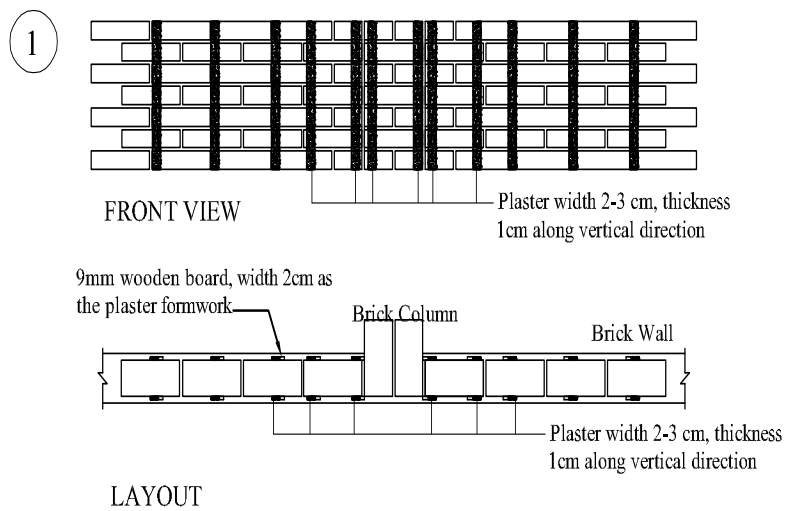


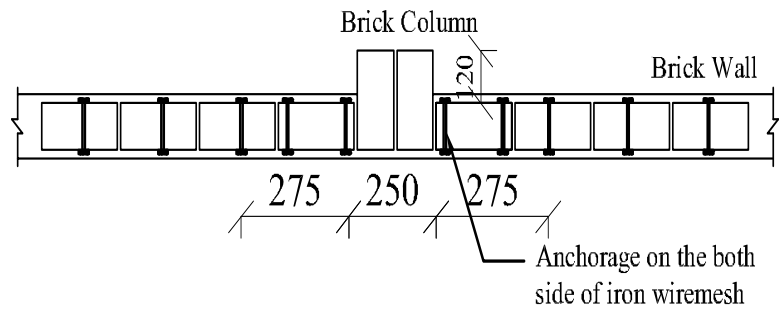


PERSPECTIVE VIEW

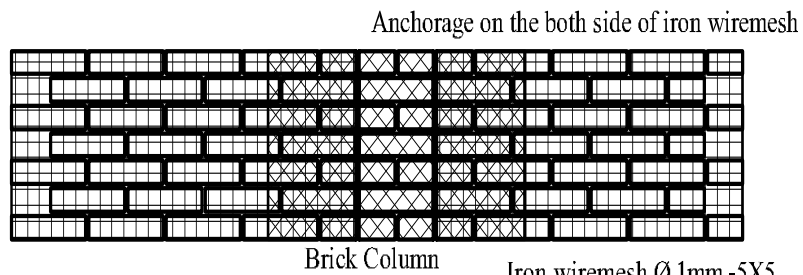
Figure B-4: Retrofitting of SDN Padasuka II (Courtesy of PT Teddy BoenKonsultan)

Detail of Beam with Iron Wire mesh Reinforcement



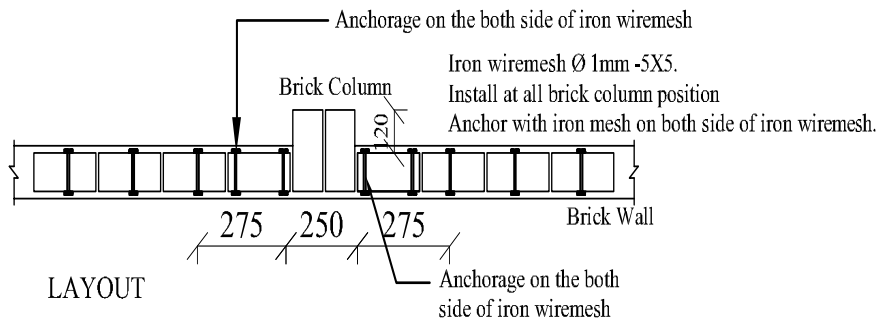


3



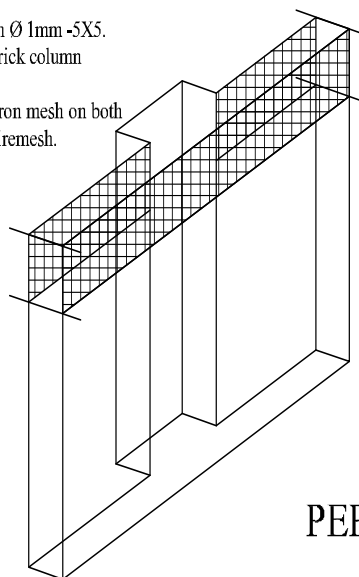
FRONT VIEW

Iron wiremesh $\text{\O} 1\text{mm} - 5\text{X}5$.
Install at all brick column position
Anchor with iron mesh on both side of iron wiremesh.



LAYOUT

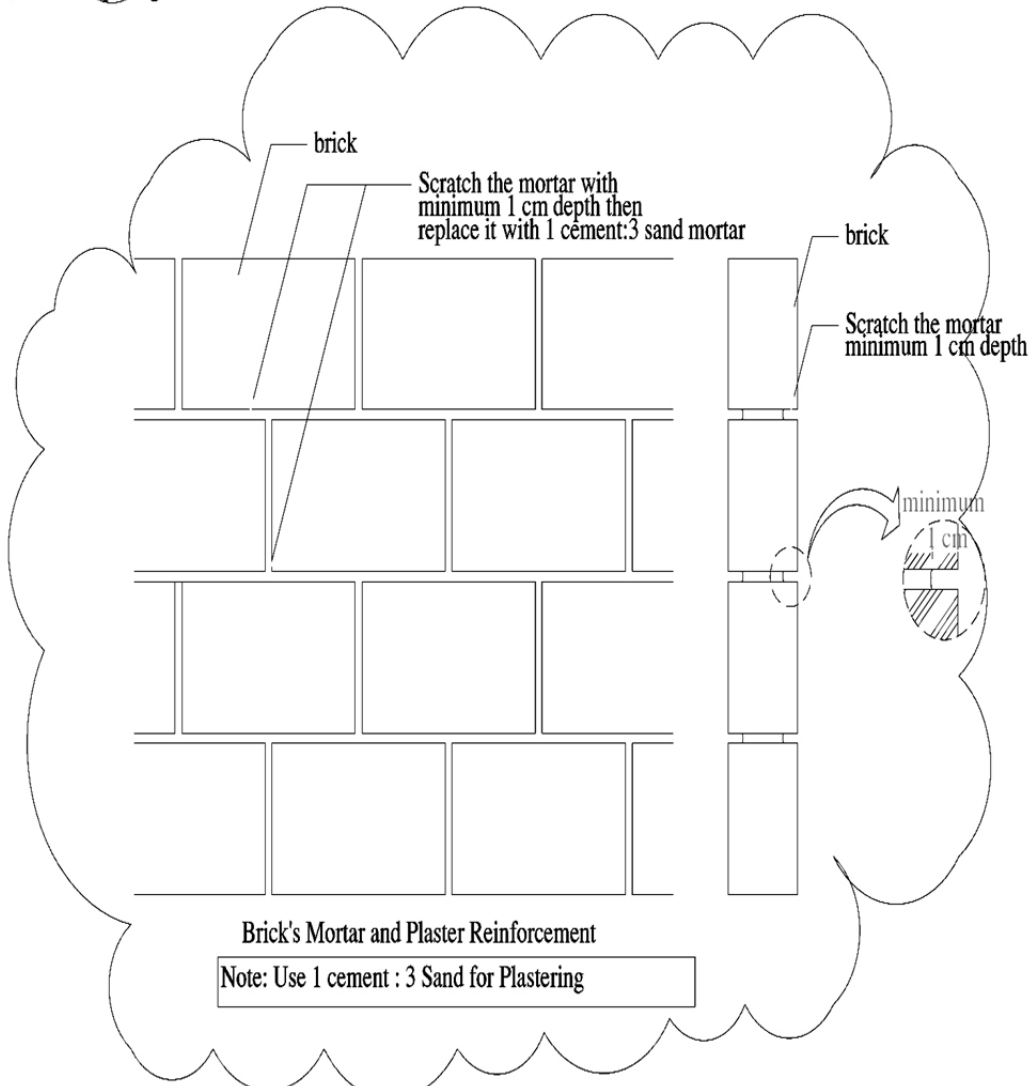
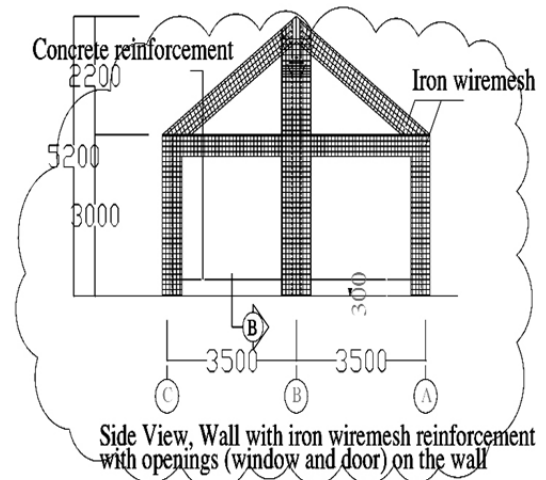
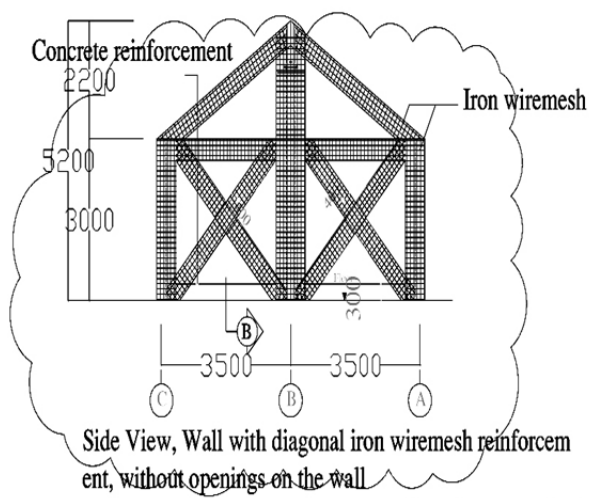
Iron wiremesh $\text{\O} 1\text{mm} - 5\text{X}5$.
Install at all brick column position
Anchor with iron mesh on both side of iron wiremesh.



PERSPECTIVE VIEW

Figure B-5: Retrofitting strategy for beams at SDN Padasuka II

Retrofitting strategy for walls of SDN Padasuka II



*B-6: Retrofitting of SDN Padasuka II
(Courtesy of PT Teddy BoenKonsultan) (cont'd)*

Retrofitting strategy for tie beams of SDN Padasuka II

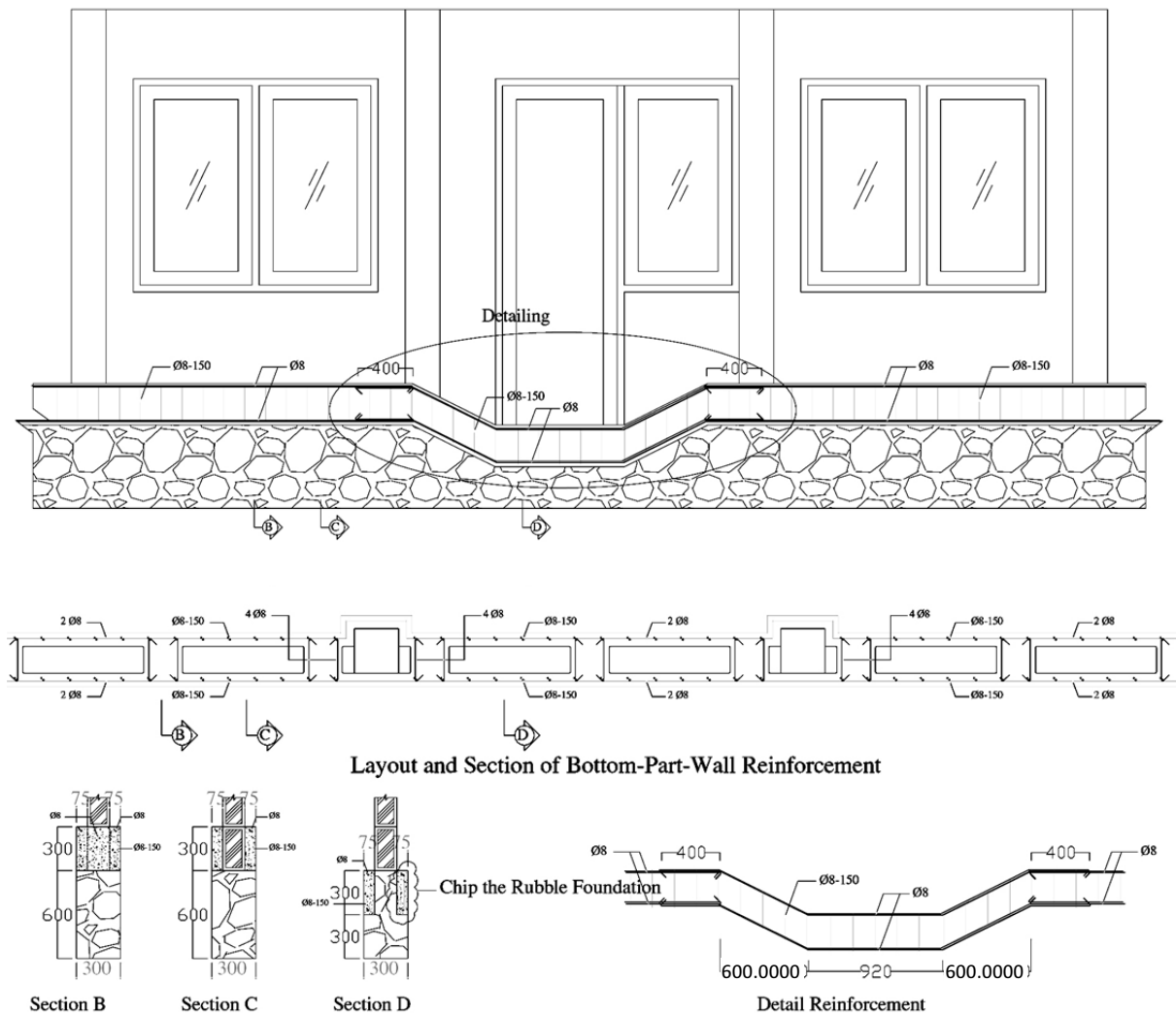


Figure B-7: Retrofitting of SDN Padasuka II
(Courtesy of PT Teddy Boen Konsultan) (cont'd)

B.2.4.2 RETROFITTING PROCESS

Following figures show the retrofitting stages conducted on the buildings



Figure B-8: Retrofitting works for the tie beam (inserting tie beam)



Figure B-9: Retrofitting works for the wall

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

a. Application of iron wire-mesh in column position



Figure B-10: Retrofitting works for the wall (cont'd)

a. Application of iron wire mesh in beam position



Figure B-11: Sanitary works (drainage system (left) and toilet (right))

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



Figure B-12: Retrofitting works for the trusses and roof (providing proper materials and detailing)



Figure B-13: Finishing works (painting, architectural gravel installation, etc)



Figure B-14: Finishing works (painting, architectural gravel installation, etc)

B.2.4.3 IMPLEMENTATION

B.2.4.3.1 POST-EARTHQUAKE CONDITION

On September 2nd 2009, approximately 7 months after the retrofitting works completed, a 7.3 Richter scale earthquake occurred with the epicenter located south of Tasikmalaya. The earthquake caused damages on many buildings, including the buildings at Bandung County. Survey was then conducted to evaluate the post-earthquake condition of SDN Padasuka II.

Based on the survey conducted, there was no significant damage on SDN Padasuka II, with only a few non-structural cracks occurred. From the post-earthquake condition, it can be concluded that the retrofitting approaches adopted on SDN Padasuka II has successfully prevented the buildings from major damage. As a comparison, there were some schools and houses located near SDN Padasuka II that were strongly affected by the earthquake shaking. Following figures show the post-earthquake condition of SDN Padasuka II.



Figure B-15: Post-earthquake condition

CASE STUDY – 3

B.3 VULNERABILITY ASSESSMENT OF FIELD OFFICE BUILDING

B.3.1 BACKGROUND

This report is based on the best engineering judgment arrived at from visual inspection and findings during the site investigations. Also, non-destructive test to obtain the strength of the structural members was carried out in the sites using Schmidt Hammer at possible locations.

This Report describes method and findings of the qualitative earthquake vulnerability assessment done as per the requirement of the Client.

B.3.2 METHODOLOGY

This assessment is done based on visual inspection, drawings developed based on site measurements, earthquake vulnerability factors identification, their qualitative analysis and short mathematical calculations to check some vulnerability factors. For the assessment of the chosen building structure, FEMA “Handbook for the Seismic Evaluation of Buildings” (FEMA 310) has been followed. FEMA 310 suggests the procedure for the identification of deficiencies in the building structure; however, it does not give the level of vulnerability if the building is non-compliant. The checklists suggested by FEMA 310 were followed and other pertinent observations necessary for the assessment were noted during the site survey. The details of the checklists are given in each section.

The preliminary assessment shall include the following:

- Detailed site survey of the structures.
- Development of drawings based on the site survey.
- Identification of the strengths and weaknesses of the building.
- Identification of structural vulnerability factors of the existing lateral force resisting system of the structures.
- General recommendations that would serve as an aid for further detailed analysis and design of retrofitting options.

B.3.2.1 INTRODUCTION

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

B.3.2.2 DESCRIPTION OF THE EXISTING STRUCTURAL SYSTEM

The structure is Two and half storied with a storey height of 3.25 meters. It was built 10 years ago. The structure is being used as Field Office . The building is situated in a flat land. The structure is composed of load bearing walls of brick in cement-sand mortar masonry in the superstructure 250mm thick and brick in cement-sand mortar masonry in the foundation. The roof consists of RCC slab, 125mm thick. The beams are provided at centre part of the building but at outer part, beams are not provided, only lintel bands (250mm x 250mm) are provided around the wall.

The size of tie beam are different, at corner, size of tie beam are 250mm x 125mm up to 400mm length, rest of other tie beams, size 150mm x 125mm around the wall.



Figure B-16: Excavation for Foundation depth check



Figure B-17: Foundation Exploration



Figure B-18: Tie Beam Exploration



Figure B-19: Crack at wall



Figure B-20: Crack at Parapet wall



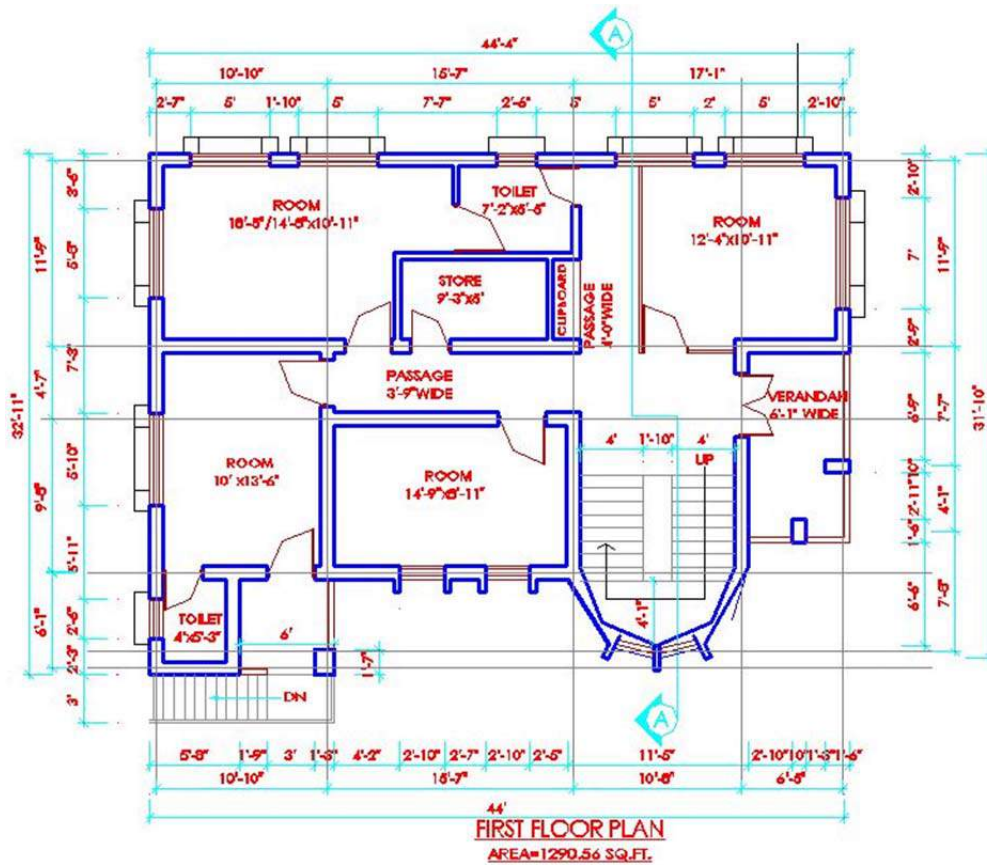
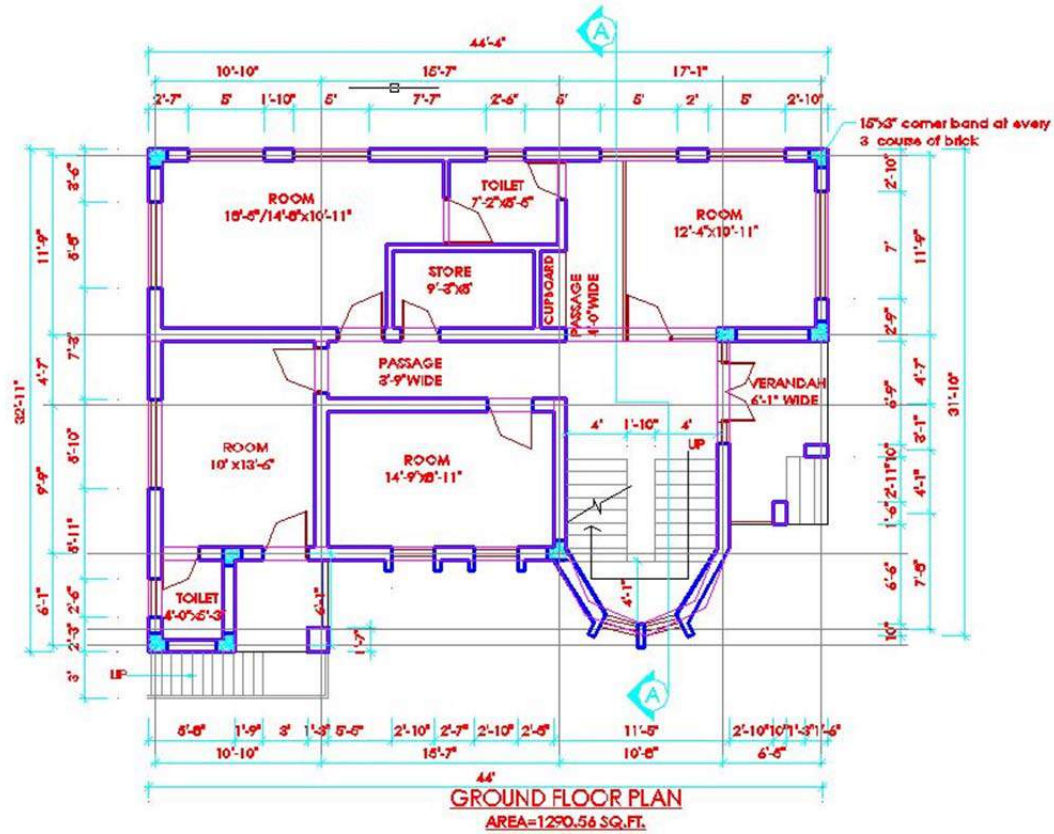
Figure B-21: Crack at Plinth Protection.

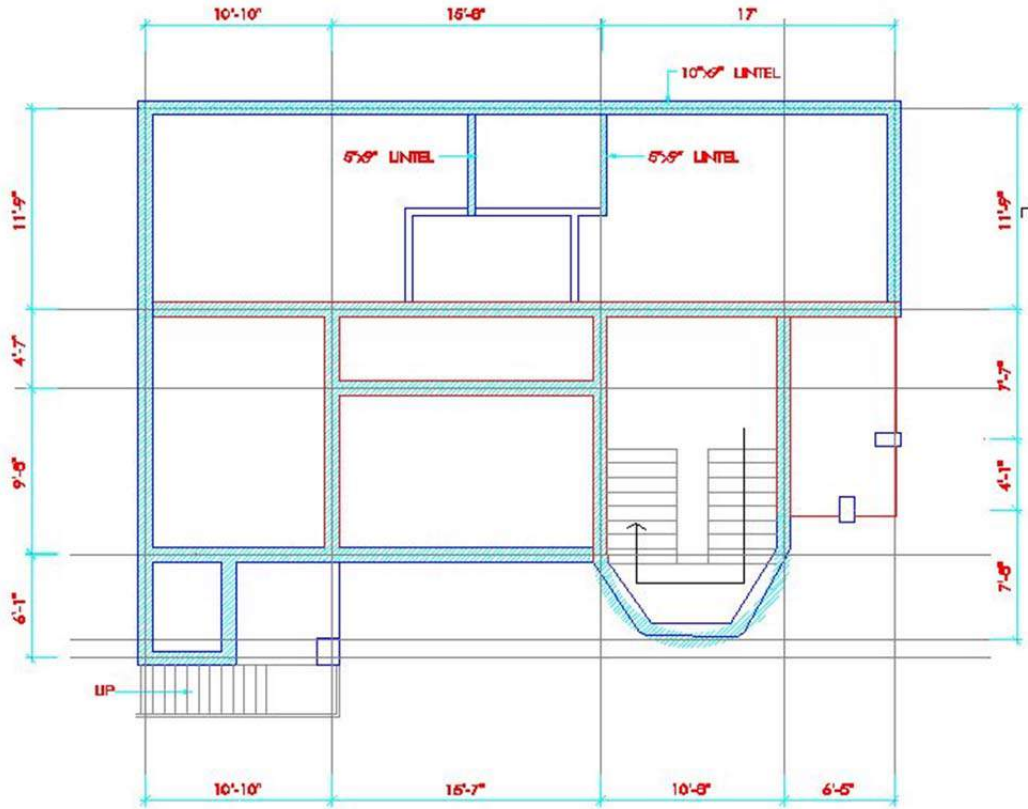


Figure B-22: Vertical crack on wall due to pipe duct

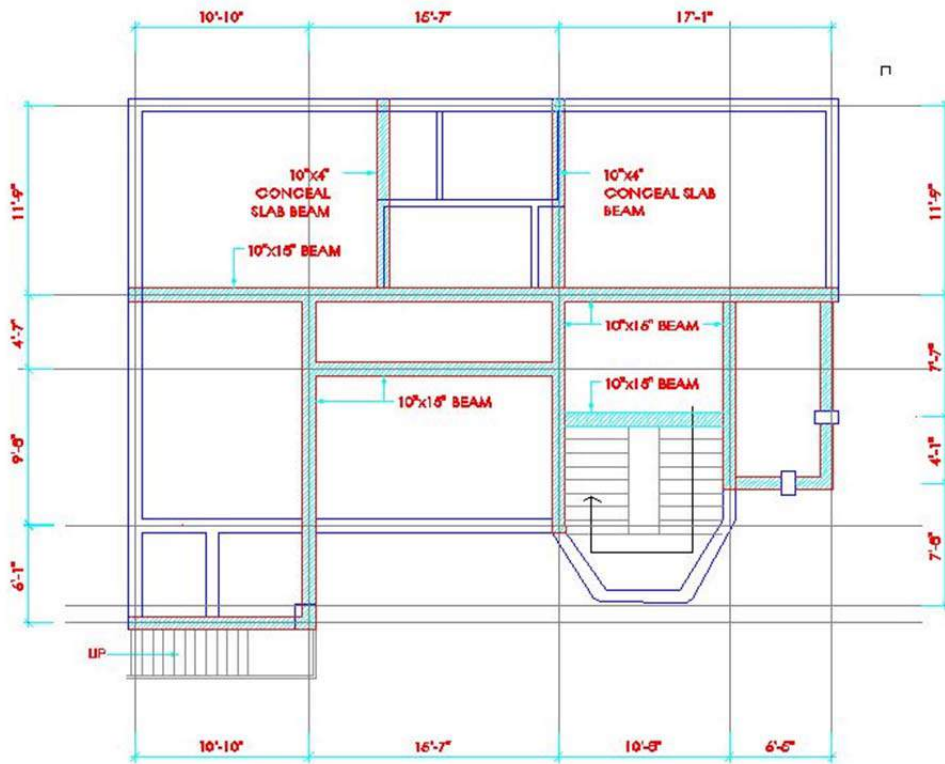


B.3.3 BUILDING DRAWINGS

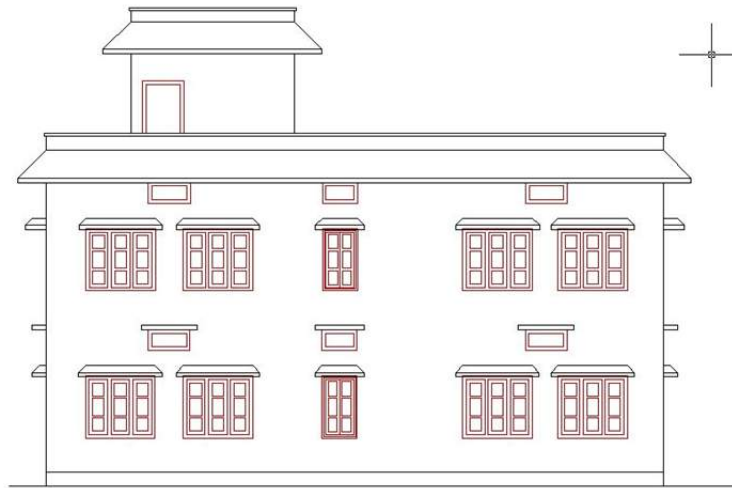




LINTEL LAYOUT PLAN



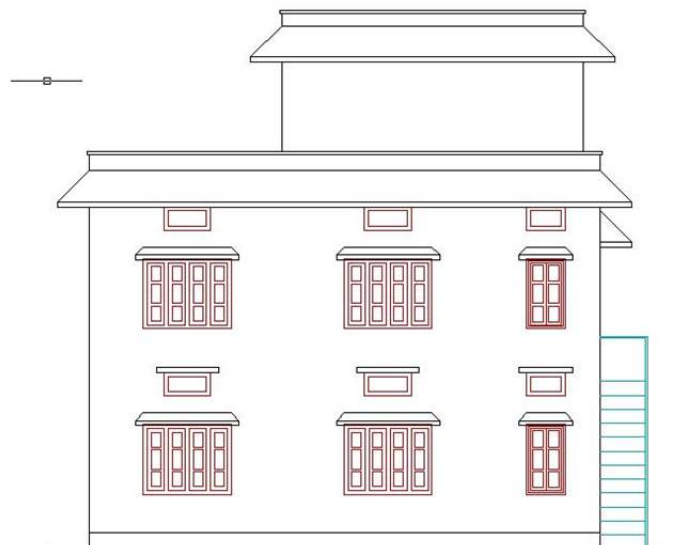
BEAM LAYOUT PLAN



SIDE VIEW-1



FRONT VIEW



BACK VIEW

ANNEX C- 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 2 or Brick in Cement masonry.

Refer: Annex D for details of the identification of different building typology.

Probable damage grades of type-2 building typology at different intensities

MMI	VI	VII	VIII	IX	X	
Damage Grades for Different Classes of Buildings	Weak	DG2	DG3	DG4	DG5	DG5
	Average	DG1	DG2	DG3	DG4	DG5
	Good	-	DG1	DG2	DG3	DG4

(Note: The description of different damage degrees is provided in Annex 2 and the details of the MMI scale is given in Annex E)

We can see from table 9 that weaker buildings in the type-2 category suffer a damage grade of 5 at an intensity of X whereas good buildings of this type-2 category suffer a damage grade of 4 at an intensity of X.

The building in Nepalgunj can be categorized as a weak building in the type-2 typology as we can already observe cracks in the structural system.

C.1 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 influence of different vulnerability factors to the building on the basis of visual inspection for building is tabulated below:

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			✓		
	Masonry Wall Cracks			✓		
Lateral Force Resisting System	Redundancy			✓		
	Shear Stress		✓			
Connection	Wall Anchorage		✓			
	Transfer of Shear Walls				✓	
Diaphragm	Plan Irregularities		✓			
	Diaphragm Reinforcement at Openings		✓			

C.2 CONCLUSION

After careful observation of the structures the following conclusions were reached. The existing structure is likely to undergo heavy structural damage during earthquakes. The crack patterns observed in the structure clearly indicates the lack of reinforcing of the walls. The cracks observed in the corners and below slabs show that the structural components have not been properly integrated for optimum structural performance during earthquakes. The beams are provided at centre part of the building but at outer part, beams are not provided, only lintel bands are provided around the wall.

C.3 RECOMMENDATIONS

The existing cracks in the building indicate the necessity of retrofitting and rehabilitation work on the structure. The following recommendations are made:

- Providing corner stitches and stitches at T-junctions.
- Providing splint and bandage wherever necessary.
- Providing continuous RCC band at the slab level.
- Provision of water proofing layer and IPC Layer in roof slab as top layer (IPC layer) is cracked all over the roof.

ANNEX D- 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 Nepal	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.
6	Others	Mixed buildings like Stone and Adobe, Stone and Brick in Mud, Brick in Mud and Brick in cement etc. are other building type in Nepal.

CLASSIFICATION OF DAMAGE GRADES:

Classification of damage to masonry buildings



Grade 1: Negligible to slight damage (no structural damage, slight non-structural damage)

Hair-line cracks in very few walls.
Fall of small pieces of plaster only.
Fall of loose stones from upper parts of buildings in very few cases.



Grade 2: Moderate damage (slight structural damage, moderate non-structural damage)

Cracks in many walls.
Fall of fairly large pieces of plaster.
Partial collapse of chimneys.



Grade 3: Substantial to heavy damage (moderate structural damage, heavy non-structural damage)

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).



Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage)

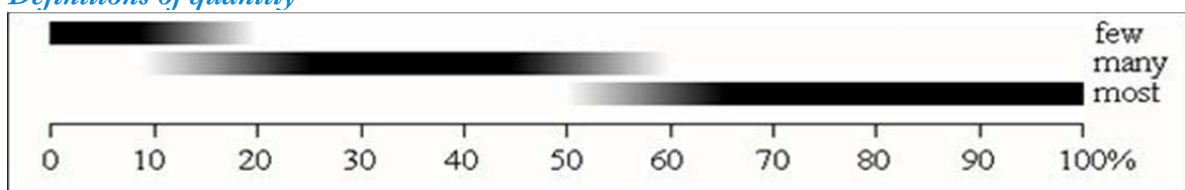
Serious failure of walls; partial structural failure of
roofs and floors.



Grade 5: Destruction (very heavy structural damage)

Total or near total collapse.

Definitions of quantity



ANNEX E- MODIFIED MERCALLY 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 F- STRESS CHECK CALCULATIONS

The following is a detail of quick check calculations based on FEMA 310 for the seismic evaluation of building under consideration.

Assumptions:

Unit weight of brickwork	= 19 KN/m ³
Dead load	= 4.125 KN/m ²
Live load	= 3 KN/m ²
Live load at roof	= 1.5 KN/m ²
Weight of plaster and floor finishes	= 1 KN/m ²

Characteristics strength of concrete of Beams and slabs = M20

Grade of steel = Fe415

Summary of Lumped Load Calculation

Level	Floor Area	Dead Load	Live Load	25% Live Load	Seismic Weight
Top Floor	66.31	273.53	99.46	24.865	298.395
1 st Floor	119.95	494.79	359.88	89.97	584.76
Ground Floor	119.95	494.79	359.88	89.97	584.76
					1467.915

F.1 CALCULATION OF BASE SHEAR

The total design lateral force or design seismic base shear is given by

Based on IS 1893 (Part 1): 2002, Criteria for earthquake resistant design of structures, Calculation of earthquake loads using Seismic coefficient method:

The design horizontal seismic coefficient, $A_h = \frac{ZIS_a}{2Rg}$

Where

- Z = Zone Factor
- I = Importance Factor
- R = Response Reduction Factor
- $\frac{S_a}{g}$ = Average Response Acceleration Coefficient

The total design lateral force or Design Seismic Base Shear (V_B) along any principal direction is determined by the following expression

$$V_B = m_1 A_h W$$

Where, A_h = The Design Horizontal Seismic Coefficient
 W = Seismic weight of the building
 m_1 = Factor for reduced useable life = 0.67

The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

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

Where, h = Height of Building in meter = 9.75

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

$$d_x = 13.67\text{m}$$

$$dz = 10.238\text{m}$$

$$T_{ax} = \frac{0.09h}{\sqrt{dx}}$$

$$= \frac{0.09 \times 9.75}{(13.67)^{0.5}}$$

$$= 0.237 < 0.55$$

$$T_{az} = \frac{0.09h}{\sqrt{dz}}$$

$$= \frac{0.09 \times 9.75}{(10.238)^{0.5}}$$

$$= 0.274 < 0.55$$

Therefore, $g = 2.5$ for medium soil (IS :1893(Part 1) : 2002)

$$Z = 0.36$$

$$I = 1.5 \text{ (6.4.2, IS 1893 (Part 1) 2002)}$$

$$\frac{S_a}{g}$$

$$= 2.5 \text{ (from graph 1893 (part 1)-2002)}$$

$$R = 1.5$$

$$A_h = \frac{ZIS_a}{2Rg}$$

$$= \frac{0.36 \times 1.5 \times 2.5}{2 \times 1.5}$$

$$= 0.45$$

$$\text{Base shear} = V_b = m_1 A_h W$$

$$= 0.67 \times 0.45 \times 1467.92$$

$$= 442.577 \text{ KN}$$

F.2 CALCULATION OF STOREY SHEAR

$$V_{bz} = 442.577 \text{ KN}$$

Floor	Total Weight Wi(kN)	Height hi (m)	Wi*hi ²	$\frac{Wi*hi^2}{\sum Wi*hi^2}$	Qi (kN)	Storey Shear Vi (kN)
Top Floor	298.40	9.75	28,366.17	0.479	211.89	211.89
1 st Floor	584.76	6.5	24,706.11	0.417	184.55	396.44
Ground Floor	584.76	3.25	6,176.53	0.104	46.14	442.58
	1.467.92		59,248.81	1.000	442.58	

F.3 SHEAR STRESS CHECK:

Floor	Storey Shear Vj (kN)	Storey Shear Vj (lb)	Storey Shear (in2)	Storey Shear (PSI)	Remarks
Top Floor	211.89	47634.78	6644.16	7.169420965	<15 Psi Hence safe
1 st Floor	396.44	89123.289	28169.28	3.163846893	<15 Psi Hence safe
Ground Floor	442.58	99495.97	28169.28	3.532073592	<15 Psi Hence safe

Hence the structure is safe in shear stress in the floor.

F.4 CHECKLIST FOR FIELD ASSESSMENT

PROJECT: Earthquake Vulnerability Assessment of FIELD OFFICE BUILDING .Kathmandu, Nepal			
Building	OFFICE	Date of survey	22 nd July, 2010
Name of building	Field Office Building	Assessment team	MRB
No. of storey	Two storey and half	Flooring material	IPC Flooring
Structural system	Load Bearing structure	Roofing material	RCC slab
Walling material	Fired brick in cement mortar cement mortar Surkhi mortar. Adobe	Roof ceiling	
Age of Building	10 yrs. (2000 AD)	Terrain	Flat

F.5 CHECKLIST FOR FIELD ASSESSMENT (TYPE 2: BRICK IN CEMENT MORTAR)

The checklist covers the basic vulnerability factors related to building systems, lateral force resisting systems, connections and diaphragms which will be evaluated mostly based on visual observation.

(**Note:** C = Compliance to the statement; NC = Not Compliance to the statement; N/A = Not Applicable, NK = Not Known)

The evaluation of different statements is made and is noted by Underlined Bold letter.

General

The structure system of building is load bearing system, which is around 10 years old. The structure is being used as Field Office. The building is situated in a flat land.

Building System: Type 2: Brick in Cement mortar

- CNC N/A NK** **LOAD PATH:** The structure shall contain one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation.
The building contains a load path except the top floor.
- CNC N/A NK** **WEAK STORY:** The strength of the lateral-force-resisting system in any story shall not be less than 80% of the strength in an adjacent story above.
The building is two and half storey. Top floor of the building does not meet this criterion . This may suffer stress concentration
- CNC N/A NK** **SOFT STORY:** The stiffness of the lateral-force-resisting system in any story shall not be less than 70% of the stiffness in an adjacent story above or below or less than 80% of the average stiffness of the three stories above or below.
Only top floor does not meet this criterion.
- CNC N/A NK** **GEOMETRY:** There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30% in a story relative to adjacent stories.
Only top floor does not meet this criterion.
- CNC N/A NK** **VERTICAL DISCONTINUITIES:** All vertical elements in the lateral-force-resisting system shall be continuous to the foundation.
This is not a problem.
- CNC N/A NK** **MASS:** There shall be no change in effective mass more than 50% from one story to the next.
Only top floor does not meet this criterion.
- CNC N/A NK** **TORSION:** The distance between the story center of mass and the story center of rigidity shall be less than 20% of the building width in either plan dimension.
Only top floor does not meet this criterion.
- CNC N/A NK** **DETERIORATION OF CONCRETE:** There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force-resisting elements.
Deterioration of concrete is not seen.
- CNC N/A NK** **MASONRY UNITS:** There shall be no visible deterioration of masonry units.
Deterioration of masonry is not seen.
- CNC N/A NK** **MASONRY JOINTS:** The mortar shall not be easily scraped away from the joints by hand with a metal tool, and there shall be no areas of eroded mortar.
This is not a problem.

- CNC N/A NK** UNREINFORCED MASONRY WALL CRACKS: There shall be no existing diagonal cracks in wall elements greater than 1/16" or out-of-plane offsets in the bed joint greater than 1/16".
Cracks observed in inner and outer walls.
- CNC N/A NK** PROPORTIONS: The height-to-thickness ratio of the shear walls at each story shall be less than the following for Life Safety and Immediate Occupancy:
Top story of multi-story building: 9
First story of multi-story building: 15
All other conditions: 13
*The height-to-thickness ratio = 3.25/0.25 = 13.
The building meets this criterion*
- CNC N/A NK** 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.
Absence of vertical reinforcement.
- CNC N/A NK** HORIZONTAL BANDS: There shall be steel or wooden bands located at the plinth, sill and lintel levels of the building in each floor.
Presence of RCC horizontal band at lintel level.
- CNC N/A NK** 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.
Corner stitches are available in the structure.
- CNC N/A NK** GABLE BAND: If the roof is sloped roof, gable band shall be provided to the building.
Flat roof available.
- CNC N/A NK** 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 within 1.2m horizontally and 0.6m vertically.
Load bearing brick wall.
- CNC N/A NK** REDUNDANCY: The number of lines of shear walls in each principal direction shall be greater than or equal to 2.
Only 2 shear walls in the some portion of the structure.
- CNC N/A NK** SHEAR STRESS CHECK: The shear stress in the un-reinforced masonry shear walls shall be less than 15 psi for clay units and 30 psi for concrete units.
Shear stress of Ground Floor = 3.53 psi, First Floor = 3.16 psi, Top Floor = 7.169 psi << 15 psi .Hence Safe .Refer Annex 4 for details.
- DIAPHRAGMS**
- CNC N/A NK** OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls shall be less than 15% of the wall length.

- CNC N/A NK** **OPENINGS AT EXTERIOR MASONRY SHEAR WALLS:** Diaphragm openings immediately adjacent to exterior masonry shear walls shall not be greater than 4 ft. long.
Some openings are greater than 5 ft. long.
- CNC N/A NK** **PLAN IRREGULARITIES:** There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities
- CNC N/A NK** **DIAPHRAGM REINFORCEMENT AT OPENINGS:** There shall be reinforcing around all diaphragms openings larger than 50% of the building width in either major plan dimension.
- CNCN/A NK** **DIAGONAL BRACING:** If there is flexible 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 NK** **LATERAL RESTRAINERS:** For flexible roof and floor, each joists and rafters shall be restrained by timber keys in both sides of wall.

CONNECTIONS

- CNC N/A NK** **WALL ANCHORAGE:** Exterior concrete or masonry walls shall be anchored for out-of-plane forces at each diaphragm level with steel anchors or straps that are developed into the diaphragm.
- CNC N/A NK** **TRANSFER TO SHEAR WALLS:** Diaphragms shall be reinforced and connected for transfer of loads to the shear walls and the connections shall be able to develop the shear strength of the walls.
- CNC N/A NK** **ANCHOR SPACING:** Exterior masonry walls shall be anchored to the floor and roof systems at a spacing 3 ft. or less.

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SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL

MASONRY STRUCTURES



SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL

2016

R C C



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.

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SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL, 2016
RCC STRUCTURES

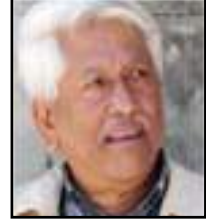
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MESSAGE

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.

Arjun Narasingha K.C.
Honorable Minister
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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
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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 academician 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 helpful in raising the safety awareness and making to community disaster resilient.

My sincere thanks goes 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 Illam 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 Julliard
UNDP Resident Representative &
United Nations Resident Coordinator

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Sa/g	Average response acceleration coefficient
Ac	Summation of cross-sectional area of all columns in the
Ac	Actual concrete to be provided in the jacket
Ah	Design horizontal seismic coefficient
As	Actual steel to be provided in the jacket
C	Basic seismic coefficient
CFRP	Carbon Fiber Reinforced Polymer
CoRD	Center of Resilient Development
CP	Collapse Prevention
d	Base dimension of the building at plinth level
DCR	Demand Capacity Ratio
dh	Diameter of stirrup
DL	Dead load
EDU	Energy Dissipation unit
F0	Axial stress of column due to overturning forces
fck	Characteristic strength of concrete
FRP	Fiber Reinforced Polymer
fy	Yield strength of steel
H	Height of the building
hi	Height of floor 'i' measured from base
I	Importance Factor
IO	Immediate Occupancy
IS	Indian Standard
K	Structural performance factor
L	Length of the building
LL	Live load
LS	Life Safety
M	Moment of resistance
NBC	National Building Code
nc	Total number of columns resisting lateral forces in the
nf	Total number of frames in the direction of loading
P	Axial load
p	Percentage of steel
R	Response reduction factor
RC	Reinforced Concrete
RCC	Reinforced Cement Concrete
S	Spacing of ties to be provided in the jacket
Ta	Natural time period of vibration
Tcol	Average shearing stress in column
tj	Thickness of Jacket
V or VB	Design seismic base shear
Vj	Maximum storey shear at storey level 'j'
W or Wt	Seismic weight of the building
W-i	Proportion of Wt contributed by level 'i'
Z	Zone Factor
UNDP	United Nations Development Programme

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1. INTRODUCTION

1.1. BACKGROUND

Nepal is located in the boundary between the Indian and Tibetan plates, along which a relative shear of about 2 cm per year has been estimated. The Indian plate is also sub-ducting at a rate of, thought to be, about 3 cm per year. The existence of the Himalayan range with the world's highest peak is an evidence of continued uplift. As a result, Nepal is seismically very active. Nepal lies in the seismic zone V which is the most vulnerable zone.

As Nepal lies in the seismic prone area and earthquake occurs frequently, people here in Nepal are now more earthquake concern. The damages caused by earthquake, small damage or large damage show the vulnerability of buildings in Nepal.

The structures of Nepal are mostly non-engineered and semi – engineered construction, which are basically lack of seismic resistance detailing. The main causes of above are lack of awareness of seismic resistance importance and strictly implementation of the codes by government level.

The non -engineered, semi –engineered structures or structures which were built before the building code was implemented can be rebuilt or reconstructed to reduce certain degree of seismic vulnerability.

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

1.3. OBJECTIVE AND SCOPE

The objective of this document is to reduce risk of loss of life and injury. This is accomplished by limiting the likelihood of damage and controlling the extent of damage.

Buildings are designed to perform at required performance level throughout its life. The material degradation due to aging and alterations carried out during use over time necessitates the operations like Repair, Restoration and Retrofit. The decay of building occurs due to original structural inadequacies, weather, load effects, earthquake, etc.

2. CONCEPT OF REPAIR, RESTORATION AND RETROFITTING

2.1. REPAIR

Repair is the process to rectify the observed defects and bring the building to reasonable architectural shape so that all services start to function. It consists of actions taken for patching up superficial defects, re-plastering walls, repairing doors and windows and services such as following:

- i. Patching up of defects as cracks and fall of plaster and re-plastering if needed.
- ii. Repairing doors, windows, broken glass panes, etc.
- iii. Rebuilding non-structural walls, chimneys, boundary walls
- iv. Relaying cracked flooring at ground level, tiles
- v. Redecoration work
- vi. Re-fixing roof tiles

It would be seen that the repairing work carried out as above does not add any strength to the structure. In fact, repair will hide the existing structural defects and hence do not guarantee for good performance when the structure is shaken by an earthquake.

2.2. RESTORATION

Restoration aims to restore the lost strength of structural elements of the building. Intervention is undertaken for a damaged building by making the columns, piers, beams and walls at least as strong as original.

Some of the common restoration techniques are:

- i. Removal of portions of cracked masonry wall and piers, and rebuilding them in richer mortar. Use of non-shrinking mortar will be preferable.
- ii. Adding wire mesh on either side of a cracked component, crack stitching etc. with a view to strengthen it.
- iii. Injecting neat slurry or epoxy like material, which is strong in tension, into the cracks in walls, columns, beams, etc.

2.3. SEISMIC STRENGTHENING (RETROFITTING)

When the existing building is incapable of withstanding the earthquake forces, it requires to be re-strengthened for safety. The complete replacement of such buildings in a given area may not be possible due to the historical importance or due to financial problems. Therefore, seismic strengthening of existing undamaged or damaged buildings is a definite requirement. The strengthening works must be fully justified from the cost point of view.

Retrofitting is undertaken to enhance the original strength to the current requirement so that the desired protection of lives can be guaranteed as per the current codes of practice against possible future earthquakes. Retrofitting of a building will involve either component strength enhancement or structural system modification or both. It is expected to improve the overall strength of the building.

2.3.1. MATERIAL AND CONSTRUCTION TECHNIQUES

Material and construction techniques are often done after damaging earthquake for repair and

strengthening of the structure. Even though cement and steel are most commonly used as repair and strengthening materials, some of the techniques and material might not be familiar to the designer.

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2.3.1.1. Conventional Cast-in-Situ Concrete

Conventional cast in situ concrete process is used in repair and strengthening works in the cases where due to the change in volume or shrinkage of the convection cement based concrete, causing unsatisfactory results. The change in volume results in loss of good contact between the new concrete and the old element preventing sound transfer of stress at the contact surface. In order to improve bond characteristics and minimize the shrinkage, it is recommended to use higher strength concrete with low slumps and minimum water. In cases where super plasticizer are used to reduce shrinkage, a slump of about 20 cm is expected, while without super plasticizers the slump should not exceed 10 cm, using standard Abrams cone.

Placement techniques are very important with cast in situ concrete to insure that the new concrete will perform adequately with the older materials. Existing surfaces which will be in contact with new cast in situ concrete must be thoroughly roughened and cleaned for good bonding characteristic (fig: 2.2). After anchorages are installed (fig:2.1), forms are constructed to meet the desired surfaces. Special chutes or access hole are frequently required in the forms to allow the placement of concrete. Immediately before placement, a final cleaning of the form is essential to remove all sawdust, etc. and the existing concrete should be moistened. The concrete should be thoroughly vibrated to insure that it completely fills the forms and voids or rock pockets are avoided. Proper curing of the newly cast concrete is also important to prevent rapid drying of the surface.



Figure 2.1 Anchors driven inside concrete after placing epoxy resin

Photo Source: MRB & Associates



Photo Source: MRB & Associates

Figure 2.2 Roughing of concrete surface for proper bondage with new concrete

2.3.1.2. Shotcrete

Shotcrete is the method of repair and strengthening reinforced concrete member where mortar is forcefully sprayed through nozzle on the surface of the concrete member at high velocity with the help of compressed air. With shotcrete method a very good bond between new shotcrete and old concrete can be obtained while repair and strengthening process. This method can be applied vertically, inclined, and over head surfaces with minimum or without formwork. Generally the materials used in this method are same as conventional mortar, and reinforcement are welded fabric and deformed bars tacked onto surface.

Shotcrete process is carried out either by these two processes:

- a) Wet process
- b) Dry process

a) Wet process:

In the wet process mixture of cement and aggregate premixed with water and the pump pushes the mixture through the hose and nozzle. Compressed air is introduced at nozzle to increase the velocity of application.

b) Dry process:

In dry mix process, compressed air propels premixed mortar and damp aggregate and at the nozzle end water is added through a separate hose. The dry mix and water through the second hose are projected on to a prepared surface.

Surface preparation before shotcreting involves a thorough cleaning and removing all loose aggregate and roughening the existing concreting surface for improved bond. Shotcrete frequently has high shrinkage characteristics and measures to prevent cracks using adequate reinforcement and proper curing is always necessary. The shotcrete surface can be left as sprayed which is somewhat rough. If a smoother surface is required, a thin layer can be sprayed on the hardened shotcrete and then reworked and finished to the required texture or plaster can be applied.

The equipment required for a minimum shotcrete operation consists of the gun, an air compressor, material hose, air and water hose, nozzle, and some time a water pump. Miscellaneous small hand tools and wheelbarrows are also required. With this minimum equipment, shotcrete works can be accomplished satisfactorily.

2.3.1.3. Grouts

Grouts are frequently used in repair and strengthening work to fill voids or to close the space between adjacent portions of concrete. Many types of grouts are available and the proper grouts must be chosen for intended usage.

Conventional grout consist of cement , sand and water and is proportioned to provide a very fluid mix which can be poured into the space to filled. Forms and closure necessary to contain the liquid grout until it has set. Conventional grout of this type has excessive shrinkage characteristics due to the high volume of water in the mix. Placing grout in a space of 2 cm to 5 cm wide will result in enough shrinkage to form a very visible crack at one side of the grouted space. Thus, conventional grouts should be used only when such cracking due to shrinkage will be acceptable.

Cement milk is formed by mixing cement with water into a fluid to place in the very small cracks. Super plasticizers are required with such mixes to maintain the water at an appropriate quantity required to hydrate the cement.

Non- shrink grouts are available for use when it is desirable to fill a void without the normal shrinkage cracks. The dry ingredients for non-shrink grout comes premixed in sacks from the manufacturer and are mixed with water in accordance with manufacturer's instruction. There are many types of non-shrink grouts available, but designers should be aware that the cost of these materials is considerably more than that of conventional grout. The properties of mixed with these materials should be known before specifying their use on a repair or strengthening project.

Epoxy or resin grouts are also available for conditions when high shear force or positive bonding is necessary across a void. Epoxy grouts come prepackaged from the manufacturer and must be mixed and used in strict accord to the instruction. Placement must be completed within the pot life of the resin before the ingredients have set. Epoxy grout generally does not shrink and provides a bonding similar to that of epoxy products. (fig:2.3)

Many other types of grouts can be created using polymer products and other newer concrete products. Shrinkage of conventional grout can be reduced using super plasticizers. The designer should become thoroughly familiar with the properties of the materials which are to be used on his project, and trial batch should be mixed and tested where appropriate.

Injection of grouts required special equipment and specially trained personnel .this method is used to repair of the members that are compressed by filling the joints, cracks, or gaps. It is also used in the restoration of the bearing surface or footing.(fig:2.4)

In many instances, it is inappropriate to fill a void with a fluid grout and a dry material that is packed or tamped into the void is used. Such a material is called a dry pack and consists of cement and sand with only a slight bit of water to moisten the dry ingredient. Dry pack is placed in the void and hand tamped with the rod until the void is filled. Dry pack should be used only in sizable voids which are wide enough to allow through compaction by tamping. Due to its low water content, dry pack generally has low shrinkage properties.



Figure 2.3 Epoxy grouting machine (Source: MRB & Associates)



Figure 2.4 Grouting on weak column (Source: MRB & Associates)

2.3.1.4. Resin concretes

In resin based concrete mixes, the cement is replaced by two component system, one component being based on liquid resin (epoxy, polyester, polyurethane, acrylic, etc.), which will react by cross linking with the second component, called hardener. Resin concrete can be useful in patching small spalled areas of concrete and are not in general use for large volumes of new concrete. Resin concretes require not only a special aggregate mix to produce the desired properties but also special working conditions, since all two component systems are sensitive to humidity and temperature.

The properties of resin concrete are as various as the number of resins offered by the industry for this purpose. However, there are some common tendencies of this relatively new construction material that should especially be taken into consideration, when using it for repair and/or strengthening works:

- Resin has a pot life which must be strictly adhered to in use so that the work is complete before the resin hardens.
- For the resin types used for construction purposes, normal reaction cannot be reached at low temperature (below +10° c); in warm weather the heat developing during the reaction can be excessive and give rise to an excessive shrinkage of the mix.
- Although the direct bond of a resin compound on a clean and dry concrete surface is excellent, a resin concrete has generally poor direct bond on concrete, due to the fact that there can only be a point to point connection between the resin covered aggregates and the old concrete. Thus, to assure a good bond it is necessary to apply a first coating of pure liquid resin onto the existing concrete surface.
- Resin concrete will commonly have a much higher strength but also a different elasticity than normal concrete; problems resulting from the different elasticity must be appropriately considered

The designer should use resin concretes only after a thorough investigation of the properties and material limitation with the existing building materials.

2.3.1.5. Polymer Modified Concrete

Polymer modified concrete is produced by replacing part of conventional cement with certain polymers which are used as cementitious modifiers. The polymer which are normally supplied as dispersions in water, act in several ways. By functioning as water reducing plasticizer they can produce a concrete with better workability, lower water-cement ratio and lower shrinkage elements. They act as integral curing aids, reducing but not eliminating the need for effective curing. By introducing plastic links into the binding system of the concrete, they improve the strength of the harden concrete. They can also increase the resistance of the concrete to some chemical attacks. However, it must be cautioned that such polymer modified concretes are bound to lose all additional properties in case they come under fire. Their alkalinity and, thus, the resistance against carbonating will be much inferior to normal concrete. The design should use polymer modified concrete only after a thorough investigation of the properties for compatibility with the existing building materials.

2.3.1.6. Fiber or reinforced polymers (FRP and CFRP)

Fiber reinforced composite materials are blends of a high strength, high modulus fiber with a hardenable liquid matrix. In this form, both fiber and matrix retain their physical and chemical identities and gives combination properties that cannot be achieved with either of the constituents acting alone. The fibers are highly directional, resulting behavior much like steel reinforced con-

crete. This behavior of fiber gives designer freedom to tailor the strengthening system to reinforce specific stresses. FRP material properties includes low specific gravity, high strength to weight ratio, high modulus to weight ratio, low density, high fatigue strength, high wear resistance, vibration absorption, dimensional stability, high thermal and chemical stability. Also, FRP materials are very resistance to corrosion. Characteristic of FRP material is the almost linear to elastic stress- strain curve to failure.

FRP materials are very much suitable for repair and strengthening process, especially for seismic loading. Wrapping FRP sheet with epoxy resin around the column upgrades its ductility due to increase in shear strength.

Pre- treatment shall be made on the surface of the column to be wrapped with carbon fiber sheet. The corner cross section of column shall be rounded with the corner radius of 20 mm or larger. This rounded portion must be straight and uncurved along the column height. While wrapping, the fiber direction shall be perpendicular to the column axis and column shall be securely and tightly wrapped with FRP sheet. Overlap of FRP sheet shall be long enough to ensure the rupture in material, lap length shall not be less than 200 mm.

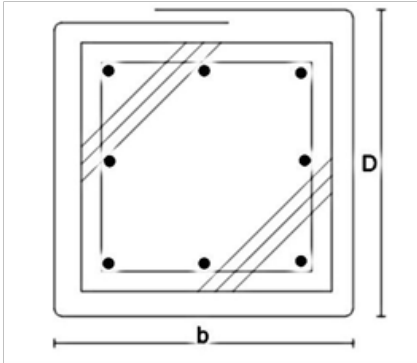
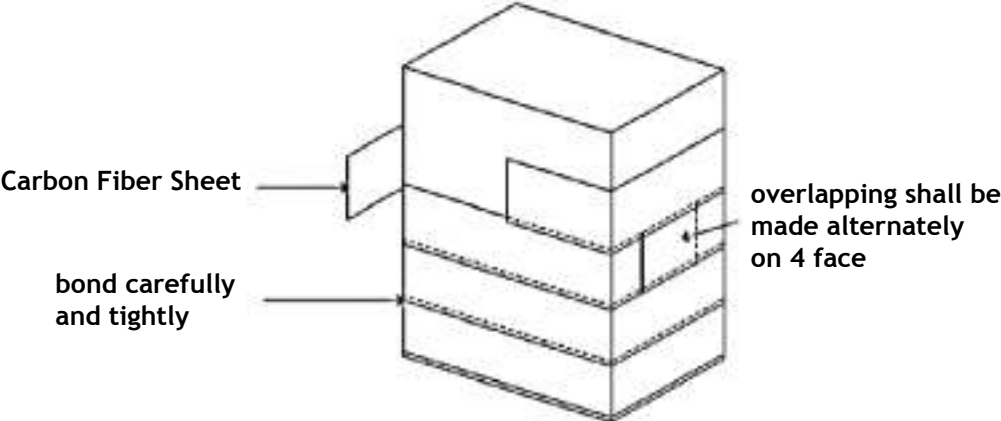


Figure 2.5 Cross- section of Column

FRP sheet shall be wrapped around the column. Position of lap splice shall be provided alternately. Impregnate adhesive resin shall be the one which has appropriate properties in construction and strength to bring the strength characteristic of FRP. After impregnation of adhesive resin has completed the initial hardening process, mortar, boards, or painting must be provided, for fire resistance, surface protection or design point of view.



Wrap and bond laterally at each tier

Figure 2.6 Column retrofit

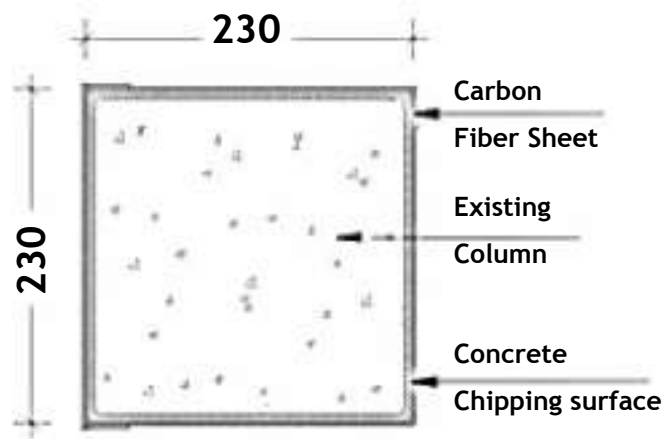


Figure 2.7 Plan view of column retrofit using carbon fiber sheet

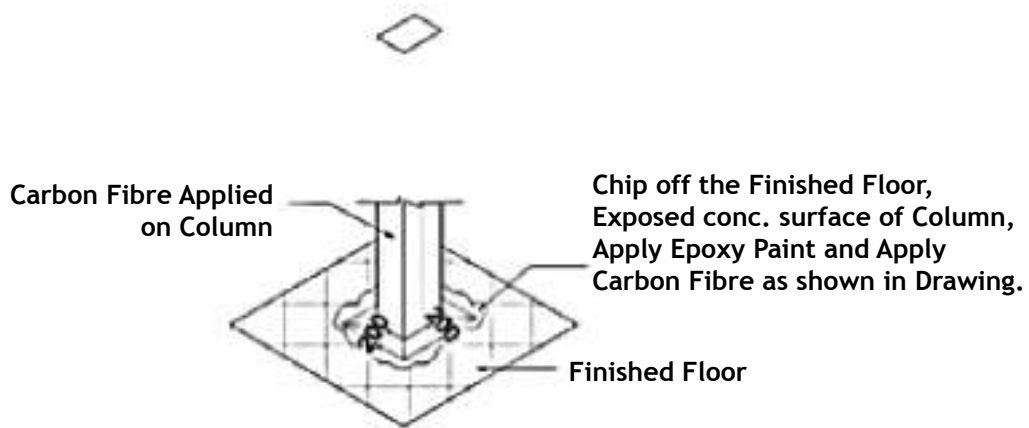


Figure 2.8 3D view of carbon fiber applied in column at floor level

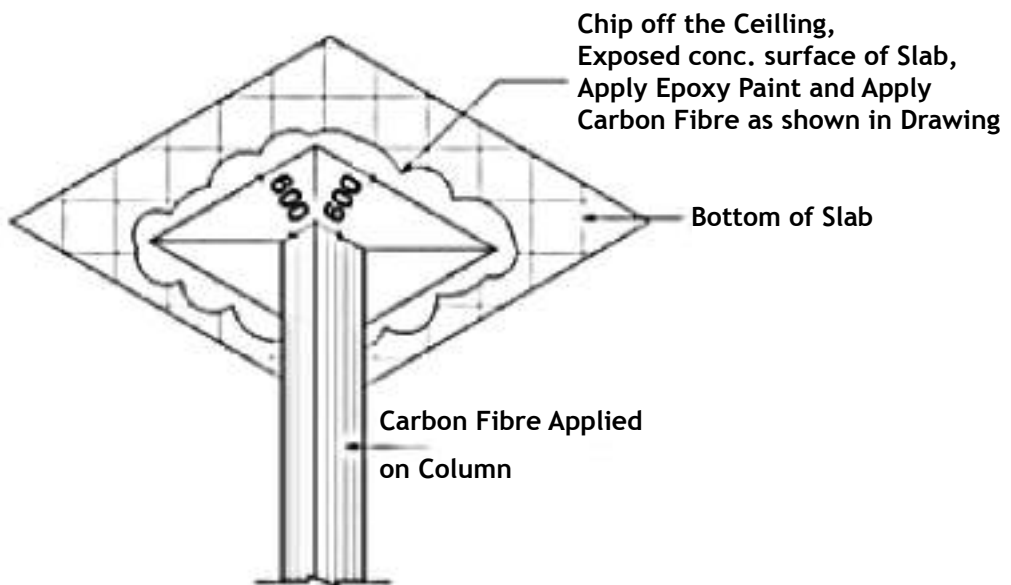


Figure 2.9 3D view of carbon fiber applied in column at ceiling level

The flowchart showing the construction process with continuous fiber sheet is illustrated in Figure 2-10.

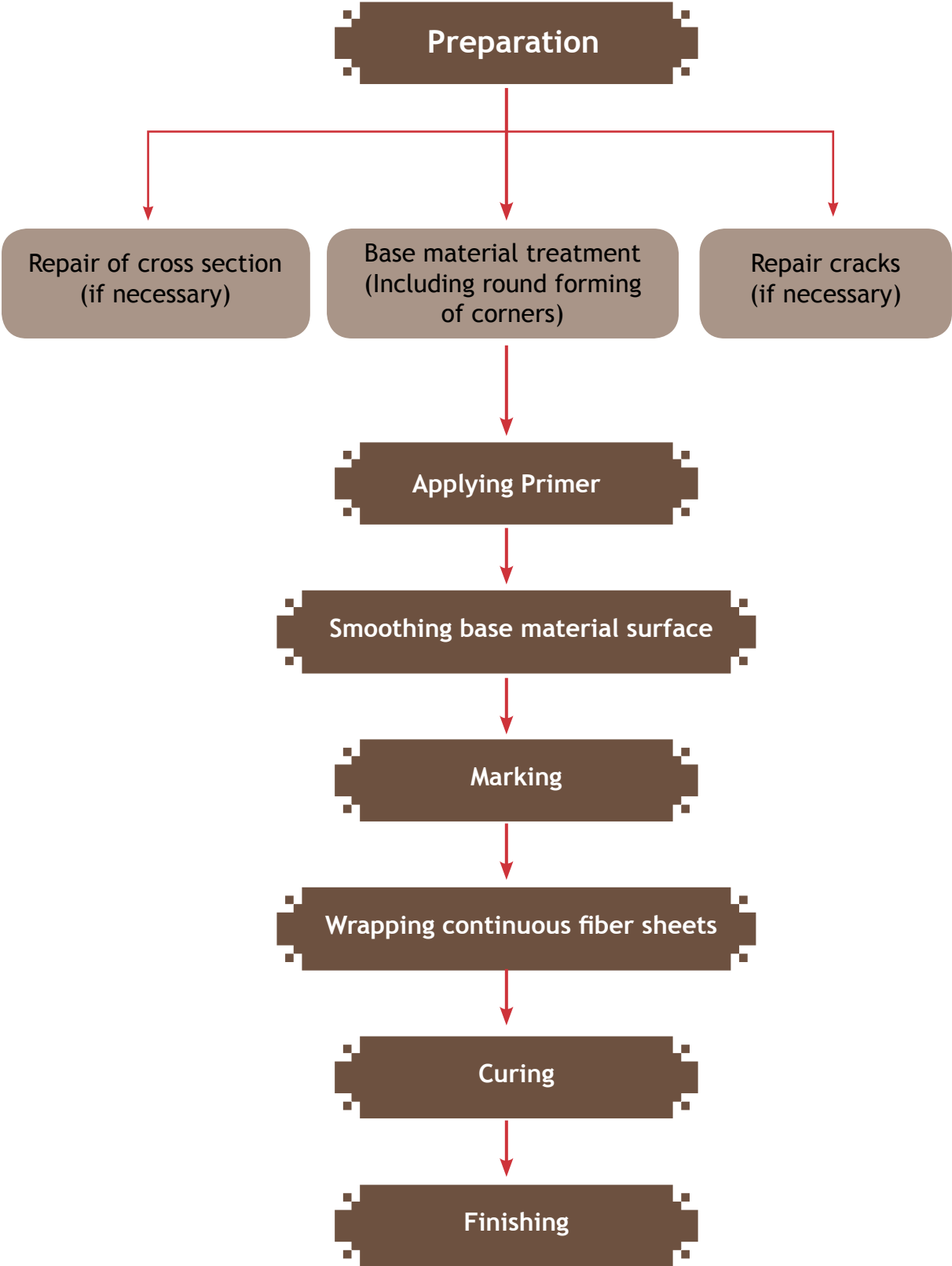


Figure 2.10. Construction process with continuous fiber sheet

3. REQUIRED PERFORMANCE LEVEL

3.1. FOR STRUCTURAL ELEMENTS

Limiting damage condition which may be considered satisfactory for a given building and given ground motion can be described as performance level.

The limiting condition is described by the physical damage within the building, the threat to life safety of the building's occupants created by the damage, and the post-earthquake serviceability of the building. The performance level ranges are assigned as:

3.1.1. IMMEDIATE OCCUPANCY (IO)

The post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical and lateral force resisting systems of the building retain nearly all of their pre-earthquake characteristic and capacities.

The risk of the life-threatening injury from structural failure is negligible, and the building should be safe for unlimited egress, ingress, and occupancy.

3.1.2. LIFE SAFETY (LS)

The post-earthquake damage state in which significant damage to the structure may have occurred but in which some margin against either total or partial structural collapse remains. Major structural components have not become dislodged and fallen, threatening life safety either within or outside the building. While injuries during the earthquake may occur, the risk of life threatening injury from structural damage is very low. It should be expected that extensive structural repairs will likely be necessary prior to reoccupation of the building.

3.1.3. COLLAPSE PREVENTION (CP)

This level is the limiting post-earthquake structural damage state in which the building's structural system is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system. Although the building retain its overall stability, significant risk of injury due to falling hazards may exist both within and outside the building and significant aftershock may lead to collapse. It should be expected that significant major structural repair will be necessary prior to re-occupancy.

3.2. FOR NON STRUCTURAL ELEMENTS

3.2.1. IMMEDIATE OCCUPANCY (IO)

The post-earthquake damage state in which nonstructural elements and systems are generally in place and functional. Although minor disruption and cleanup should be expected, all equipment and machinery should be working. Contingency plans to deal with possible difficulties with external communication, transportation and availability of supplies should be in place.

3.2.2. LIFE SAFETY (LS)

The post-earthquake damage state could include minor disruption and considerable damage to nonstructural components and system particularly due to damage or shifting of contents. Although equipment and machinery are generally anchored or braced, their ability to function after strong

shaking is not considered and some limitations on use or functionality may exist. Standard hazard from breaks in high pressure, toxic or fire suppression piping should not be present. While injuries during the earthquake may occur, the risk of life threatening injuries from nonstructural damage is very low.

3.2.2. COLLAPSE PREVENTION (CP)

This post-earthquake damage state could include extensive damage to nonstructural components or systems but should not include collapse or falling of large and heavy items that could cause significant injuries to group of people, such as parapets, masonry exterior walls, cladding or large heavy ceilings. Nonstructural systems, equipments and machinery may not be functional without replacement or repair. While isolated serious injuries could occur, risk of failures that could put large numbers of people at risk within or outside the building is very low.

4. SEISMIC ASSESSMENT

4.1. RAPID ASSESSMENT (VISUAL SURVEY)

Rapid Seismic Assessment is the preliminary assessment, which concludes the recent status of the building as is it is suitable to live in or not, can be retrofitted or not. In this process, the first level is site inspection, which is also called as visual survey.

4.1.1. METHODOLOGY FOR RAPID SEISMIC ASSESSMENT

1. Review available Structural and Architectural Drawings
2. Review of the Design Data. if available.
3. Interview with the Designer, if possible.
4. Inspection of the Buildings.
5. Identification of Vulnerability Factors as per FEMA 310.
6. Determination of Strength of the Structural Components using Schmidt Hammer
7. Analysis of the Structural Systems, as per guidelines of FEMA 310.
8. Latest Photographs of the Building

4.1.2. BUILDING – FACTS

1. Age of building
2. Structural System – Load bearing Or Frame Structure
3. Foundation Exploration
4. Load path
5. Geometry
6. Walls Detail – Size and mortar
7. Beam and Column Size
8. Water proofing method
9. Renovation of Building
10. Other Structures added

4.2. PRELIMINARY EVALUATION

A preliminary evaluation of building is carried out which involves broad assessment of its physical condition, robustness, structural integrity and strength of structure, including simple calculations. Approximate and quick checks shall be used to compute the strength and stiffness of building components. The seismic base shear and storey shears for the building shall be computed in accordance with IS1893 (Part1).

The preliminary evaluation is a quick procedure to establish actual structural layout and assess its characteristics that can affect its seismic vulnerability. It is a very approximate procedure based on conservative parameters to identify the potential earthquake risk of a building and can be used to screen buildings for detailed evaluation.

4.2.1. SITE VISIT

A site visit will be conducted by the design professional to verify available existing building data or collect additional data, and to determine the condition of the building and its components.

4.2.2. ACCEPTABILITY CRITERIA

A building is said to be acceptable if it meets all the configuration-related checks as well as global level checks on axial and shear stress as outlined in the following sections.

4.2.3. CONFIGURATION RELATED CHECKS

4.2.3.1. *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 all inertial forces in the building to the foundation.

4.2.3.2. *Redundancy:*

The number of lines of vertical lateral load resisting elements in each principal direction shall be greater than or equal to 2. In the case of moment/braced frames, the number of bays in each line shall be greater than or equal to 2.

4.2.3.3. *Geometry:*

No change in the horizontal dimension of lateral force resisting system of more than 50% in a storey relative to adjacent stories, excluding penthouses and mezzanine floors, should be made.

4.2.3.4. *Weak Storey:*

The strength of the vertical lateral force resisting system in any storey shall not be less than 70% of the strength in an adjacent storey.

4.2.3.5. *Soft Storey:*

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

4.2.3.6. *Vertical Discontinuities:*

All vertical elements in the lateral force resisting system shall be continuous from the root to the foundation.

4.2.3.7. *Mass:*

There shall be no change in effective mass more than 100% from one storey to the next. Light roofs, penthouses, and mezzanine floors need not be considered.

4.2.3.8. *Torsion:*

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

4.2.3.9. *Adjacent Buildings:*

The clear horizontal distance between the building under consideration and any adjacent building shall be greater than 4% of the height of the shorter building, except for buildings that are of the same height with floors located at the same levels.

4.2.3.10. *Short Columns:*

The reduced height of a column due to surrounding parapet, infill wall, etc. shall not be less than five times the dimension of the column in the direction of parapet, infill wall, etc. or 50% of the nominal height of the typical columns in that storey.

4.2.3.11. Strength-Related Checks

Approximate and quick checks shall be used to compute the strength and stiffness of building components. The seismic base shear and storey shears for the building shall be computed in accordance with IS 1893 (Part1) and NBC 105 (Table 41 and Table 42).

	IS 1893:2002	NBC105:1994
The design horizontal seismic coefficient	$A_h = \frac{ZIS_a}{2R_g}$ Where, Z = Zone factor I = Importance factor R = Response reduction factor $\frac{S_a}{g}$ = Average response acceleration coefficient	$C_d = CZIK$ Where, C = Basic seismic coefficient Z = Seismic zoning factor K = Structural performance factor
The total design lateral force or design seismic base shear (V _B) along any principal direction is determined by using the expression	$V_B = A_h W$ Where, W = Seismic weight of the building IS 1893	$V = C_d W_t$ NBC 105
The approximate fundamental natural period of vibration (T _a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:	$T_a = \frac{0.09b}{\sqrt{d}}$ Where, h = Height of Building in meter d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force	$T_a = \frac{0.09b}{\sqrt{d}}$

Table 4.1 Calculation of earthquake load using seismic coefficient method

	IS 1893:2002	NBC 105:1994
The design base shear (V _B) shall be distributed along the height of the building as per the expression	$Q_i = V_B \frac{W_i h_i^2}{\sum W_i h_i^2}$ Where, Q _i = Design lateral force at floor i, W _i = Seismic weight of floor i, h _i = height of floor i, from the base	$F_i = V \frac{W_i h_i}{\sum W_i h_i}$ Where, F _i = Horizontal seismic force at floor i

Table 4.2 Distribution of base shear and calculation of shear stress in RC columns

(a) Shear stress in RC frame columns

Average shearing stress in column is given as,

$$T_{col} = \left(\frac{n_c}{n_c - n_f} \right) \times \left(\frac{V_c}{A} \right) < \text{min. of } 0.4 \text{ Mpa and } 0.1 \sqrt{f_{ck}} \text{ (Ref IS 15988:2013 Clause 6.5.1 b)}$$

For Ground Storey columns,

n_c = Total no. of columns resisting lateral forces in the direction of loading

n_f = Total no. of frames in the direction of loading

A_c = Summation of the cross – section area of all columns in the storey under consideration

V_j = Maximum Storey shear at storey level ‘j’

(b) Axial Stress Check:

Axial stresses due to overturning forces as per FEMA 310 (clause 3.5.3.6)

Axial stress in moment frames

Axial force in columns of moment frames at base due to overturning forces,

The axial stress of columns subjected to overturning forces F_o is given by

$$F_o = \frac{2}{3} \left(\frac{V_b}{n_f} \right) \times \left(\frac{H}{L} \right) \text{ (Ref IS:15988:2009 clause 6.5.4)}$$

V_b = Base shear x Load Factor

H = total height

L = Length of the building

4.3. DETAILED EVALUATION

A detailed evaluation is required unless results of preliminary evaluation are acceptable. The detailed evaluation procedure is based on determining the probable strength of lateral load resisting elements and comparing them with the expected seismic demands.

4.3.1. CONDITION OF THE BUILDING COMPONENTS

The building should be checked for the existence of some of the following common indicators of deficiency.

4.3.1.1. Deterioration of Concrete

There should be no visible deterioration of the concrete or reinforcing steel in any of the vertical or lateral force resisting elements.

4.3.1.2. Cracks in Boundary Columns

There shall be no existing diagonal cracks wider than 3 mm in concrete columns that encase masonry infills.

4.3.1.3. Masonry Units

There shall be no visible deterioration of masonry units.

4.3.1.4. Masonry Joints

The mortar shall not be easily scraped away from the joints by hand with a metal tool, and there shall be no areas of eroded mortar.

4.3.1.5. Cracks in Infill Walls

There shall be no existing diagonal cracks in infill walls that extend throughout a panel, are greater than 3 mm, or have out-of-plane offsets in the bed joint greater than 3 mm.

4.3.1.6. Condition of the Building Materials

An evaluation of the present day strength of materials can be performed using on-site non-destructive testing and laboratory analysis of samples taken from the building. Field tests are usually indicative tests and therefore should be supplemented with proper laboratory facilities for accurate quantitative results.



Figure 4.1 Schmidt Hammer (Source: MRB & Associates)



Figure 4.2 Ferro- scanner (Source: MRB & Associates)



Figure 4.3 Ultra sonic range finder (source: MRB & Associates)

4.4. EVALUATION PROCEDURE

- Calculation of Base Shear as defined in Preliminary Evaluation
- Shear capacity of frame members shall be adequate to develop the moment capacity at the ends, and shall be in accordance with provisions of IS: 13920 for shear design of beams and columns.

The design shear force for columns shall be the maximum of:

a) Calculated factored shear force as per analysis,

b) A factored shear force given by,

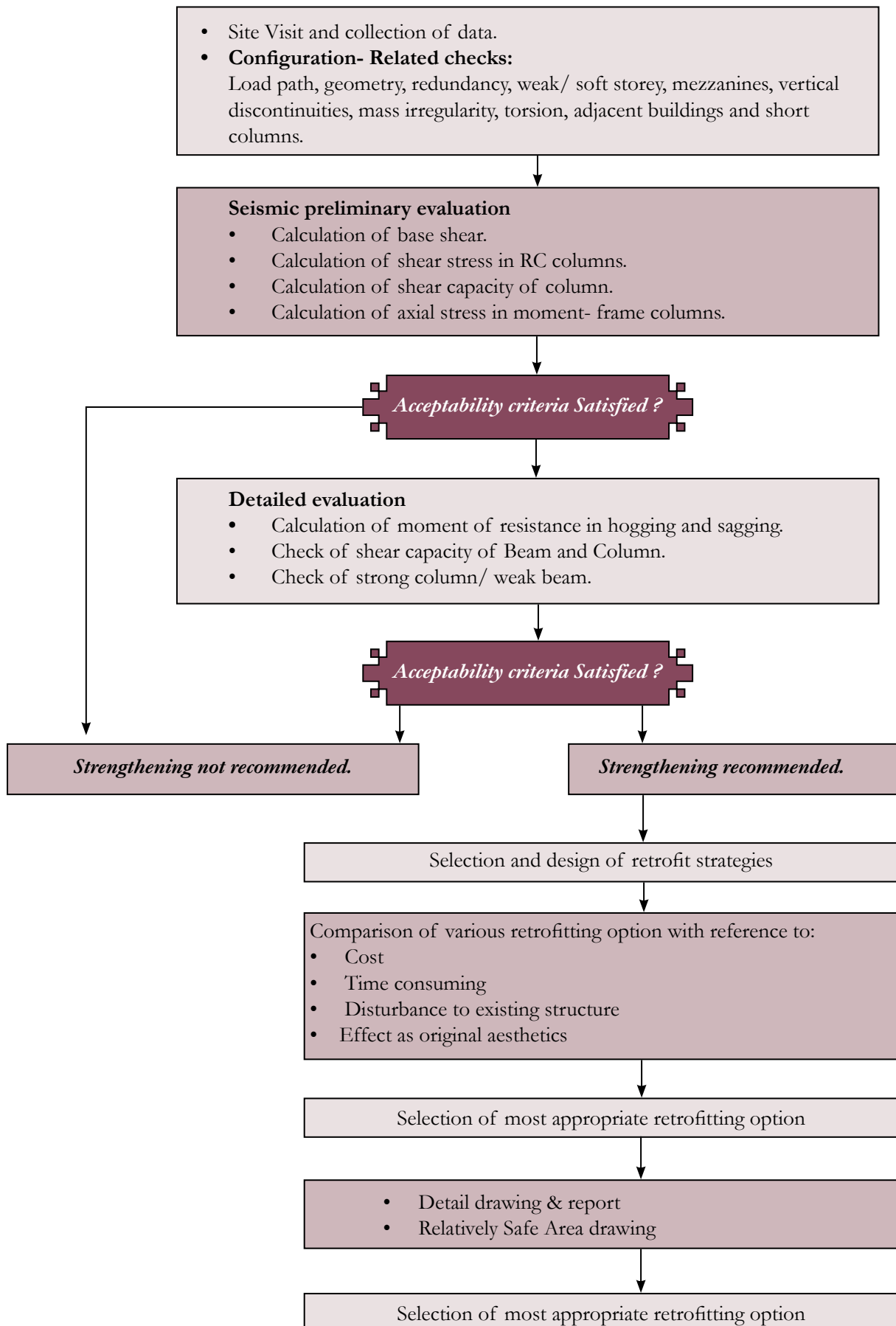
$$V_u = 1.4 \times \frac{M_1 + m_1'}{h_{st}} \quad (\text{Ref IS 13920:1993 Clause 7.3.4})$$

M_1 and m_1' are moment of resistance, of opposite signs, of beams framing into the column from opposite faces

- All concrete columns shall be anchored into the foundation.
- The sum of the moment of resistance of the columns ($\sum Mc$) shall be at least 1.1 times the sum of the moment of resistance of the beams ($\sum Mb$) at each frame joint.

$$\sum Mc \geq 1.1 \sum Mb \quad (\text{Ref IS15988:2013 Clause 7.4.1(c)})$$

Seismic Evaluation



5. CATEGORIZATION OF DAMAGE GRADE

5.1. Damage Categorization Table

S.N.	Damage Grades	Level of Damage	Recommendations after Earthquake	Remarks
G1	Negligible – slight damage (Non or slight structural)	Only thin cracks in some wall plaster, can fall of plaster parts, fall of loose brick or stone from upper parts.	Only architectural repair needed. Appropriate seismic strengthening advised.	



Figure 5.1 Damage grade 1 (source: MRB & Associates)

S.N.	Damage Grades	Level of Damage	Recommendations after Earthquake	Remarks
G2	Moderate damage. (Slight or moderate non-structural damage)	Many thin cracks in walls and in plasters fall of brick or stone work, fall of plaster but no structural part damage.	Only architectural repair needed. Appropriate seismic strengthening advised.	



Figure 5.2 Damage grade 2 (source: MRB & Associates)

S.N.	Damage Grades	Level of Damage	Recommendations after Earthquake	Remarks
G3	Moderate to heavy damage. (Moderate Structure, heavy non structure damage)	Thick and large cracks in many walls, upper structure like tiles or chimney damage failure or non-structural partition wall	Architectural and structural repair required. Grouting in crack advised and strongly advised structure strengthening with technical support.	



Figure 5.3 Damage grade 3 (source: MRB & Associates)

S.N.	Damage Grades	Level of Damage	Recommendations after Earthquake	Remarks
G4	Very heavy damage (Heavy structure, very heavy non structure damage)	Large gap occurs in main walls, wall collapses, some structural floor or roof damage.	Immediately vacate the building, demolish and construct with seismic designs. In some case extensive restoration and strengthening can be apply.	Technical Assistance Recommended



Figure 5.4 Damage grade 4 (source: MRB & Associates)

S.N.	Damage Grades	Level of Damage	Recommendations after Earthquake	Remarks
G5	Destruction (Very heavy structure Damage)	Floor collapse due to soft storey, partial or total collapse of building.	Immediately clear the site and reconstruction the building following seismic design.	Technical Assistance Recommended



Figure 5.5 Damage grade 5 (source: MRB & Associates)

6. SEISMIC STRENGTHENING STRATEGY AND SEISMIC RETROFITTING OPTIONS

Seismic strengthening for improved performance in the future earthquakes can be achieved by using one of the several options that will be discussed in this section once an evaluation has been conducted and the presence of unacceptable seismic deficiencies has been detected.

Basic issues that might rise while retrofitting the buildings are:

- **Socio-cultural issues**
 - Heritage sites

- **Economic issues**
 - Cost of demolition & rubble removal
 - Cost of reconstruction
 - Real state
 - Built-up area vs. carpet area

- **Technical issues**
 - Type of structural system
 - Construction materials
 - Site
 - Damage intensity level

- **Legal issues**

For most buildings and performance objectives, a number of alternative strategies and systems may result in acceptable design solutions. Prior to adopting a particular strategy, the engineer should evaluate a number of alternatives for feasibility and applicability and together with the owner, should select the strategy or combination of strategies that appears to provide the most favorable overall solution.

The strategies that are discussed in the following stages describe a methodology for the design of the strengthening measures at a general level as modifications to reduce/correct seismic deficiency.

6.1. RETROFIT STRATEGIES

A retrofit strategy is a basic approach adopted to improve the probable seismic performance of a building or otherwise reduce the existing risk to an acceptable level. Strategies relate to modification or control of the basic parameters that affect a buildings earthquake performance. These include the building's stiffness, strength, deformation capacity, and ability to dissipate energy, as well as the strength and character of ground motion transmitted to the building. Strategies can also include combinations of these approaches. For example, the addition of shear walls or braced frames to increase stiffness and strength, the use of confinement jackets to enhance deformability.

There is wide range of retrofit strategies available for reducing the seismic risk inherent in an existing building. These strategies include:

6.1.1. SYSTEM STRENGTHENING AND STIFFENING

System strengthening and stiffening are the most common seismic performance improvement strategies adopted for buildings with inadequate lateral force resisting systems.

Introduction of new structural elements to the building system can improve the performance of the building. This can be achieved by introducing,

6.1.2. SHEAR WALL INTO AN EXISTING CONCRETE STRUCTURE

The introduction of shear walls into an existing concrete structure is one of the most commonly employed approaches to seismic upgrading. It is an extremely effective method of increasing both building strength and stiffness. A shear wall system is often economical and tends to be readily compatible with most existing concrete structures.

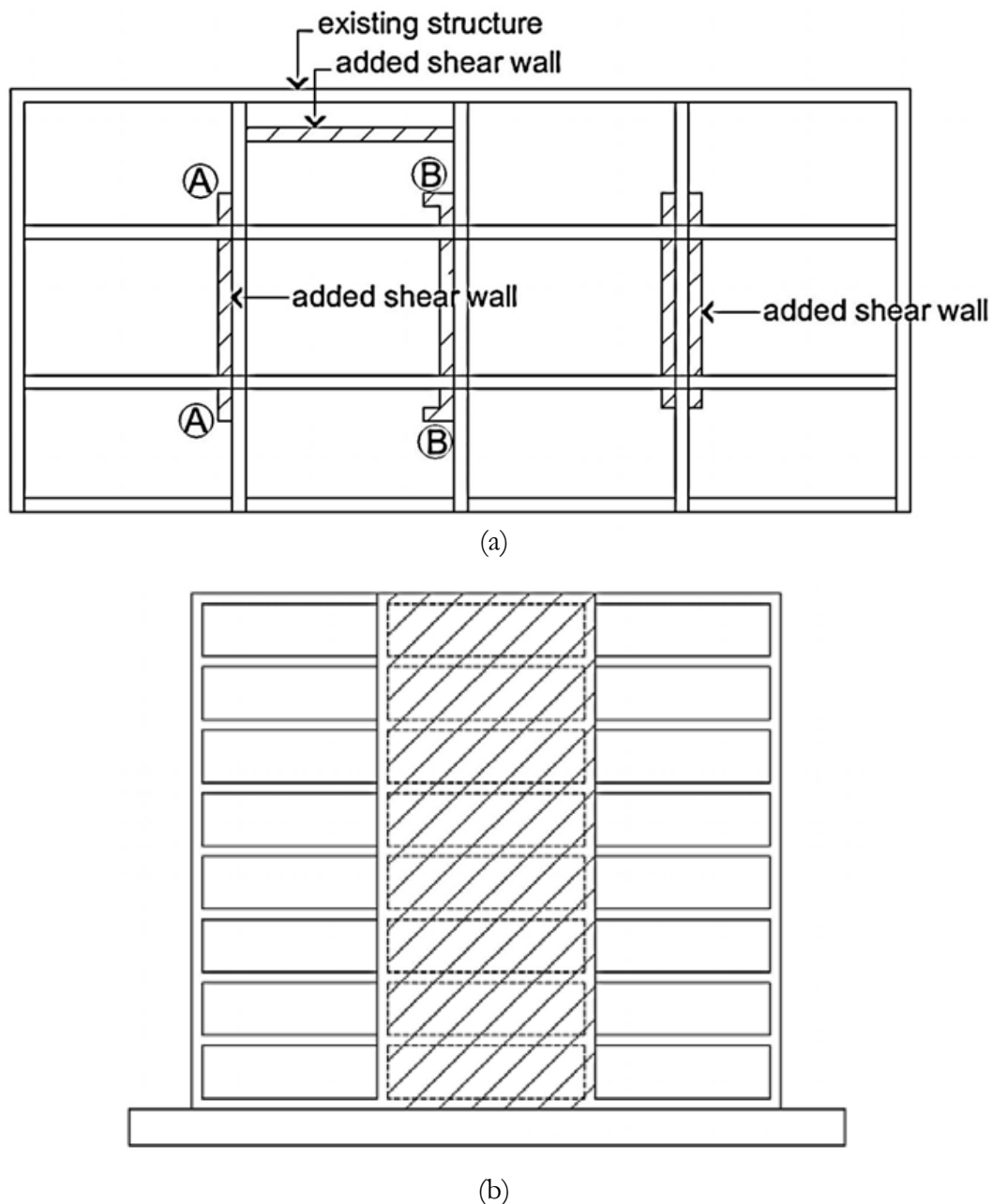


Figure 6.1 Shear wall in existing structure (a: Plan, b: Elevation)

6.1.3. BUTTRESSES PERPENDICULAR TO AN EXTERNAL WALL OF THE STRUCTURE

Buttresses are braced frames or shear walls installed perpendicular to an exterior wall of the structure to provide supplemental stiffness and strength. This system is often a convenient one to use when a building must remain occupied during construction, as most of the construction work can be performed on the building exterior, minimizing the convenience to building occupants.



Figure 6.2 Buttress provide to exterior building (source: Hari Darshan Shrestha)

6.1.4. MOMENT RESISTING FRAMES

Moment-resisting frames can be an effective system to add strength to a building without substantially increasing the buildings stiffness. Moment frames have the advantage of being relatively open and therefore can be installed with relatively minimal impact on floor space.

6.1.5. INFILL WALLS

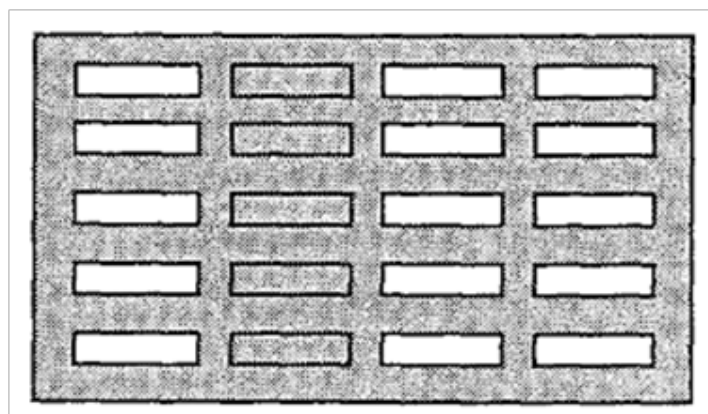


Figure 6.3 Building retrofit with infill windows

6.1.6. TRUSSES AND DIAGONAL BRACES

Braced steel frames are another common method of enhancing an existing buildings stiffness and strength. Typically, braced frames provide lower levels of stiffness and strength than do shear walls, but they add far less mass to the structure than do shear walls, can be constructed with less disruption of the building, result in less loss of light, and have a smaller effect on traffic patterns within the building.

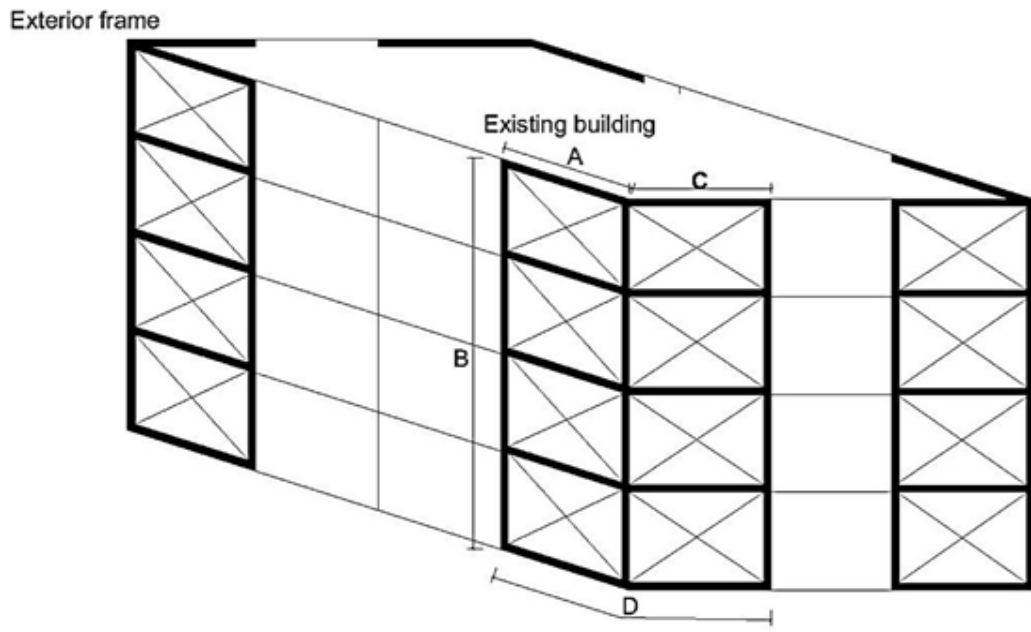


Figure 6.4 Exterior frame (steel framed brace)

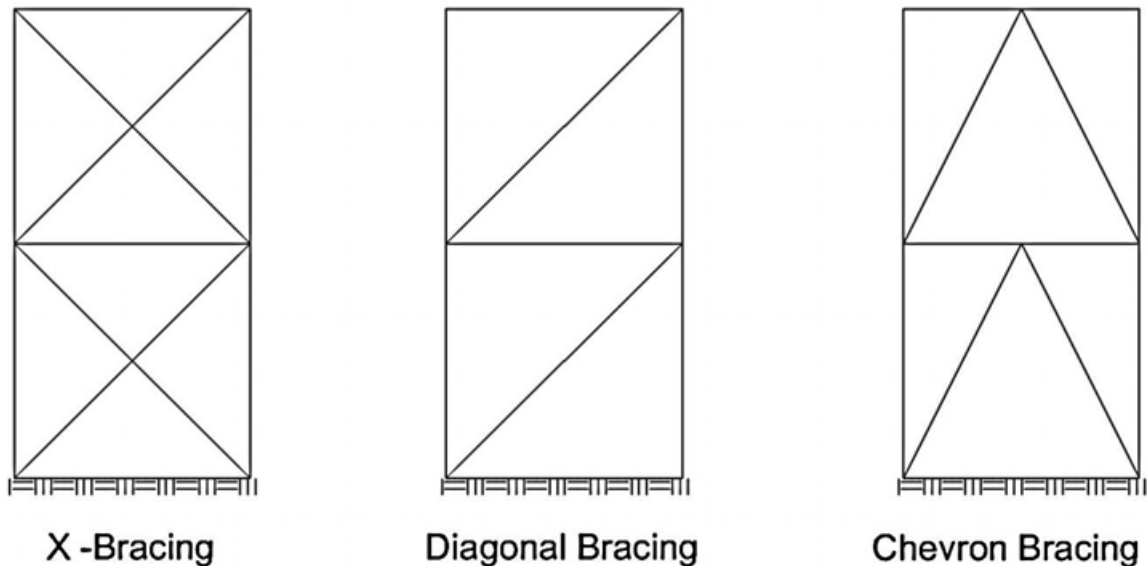


Figure 6.5 Types of bracings

Angle or channel steel profile can be used for the purpose of adding steel braces. Braces should be arranged so that their center line passes through the centers of the beam-column joints.

Likewise, eliminating or reducing structural irregularities can also improve the performance of the building in earthquake such as:

- Vertical irregularities
- Filling of openings in walls
- Pounding effect of the buildings
- Improving diaphragm in the presence of large openings by provision of horizontal bracing.

6.1.7. DIAPHRAGM STRENGTHENING

Most of the concrete buildings have adequate diaphragms except when there occur large openings. Methods of enhancing diaphragms include the provision of topping slabs, metal plates laminated onto the top surface of the slab, or horizontal braced diaphragms beneath the concrete slabs.

6.1.8. STRENGTHENING OF ORIGINAL STRUCTURAL ELEMENTS

Strengthening of reinforced concrete structural elements is one method to increase the earthquake resistance of damaged or undamaged buildings. Repair of reinforced concrete elements is often required after a damaging earthquake to replace lost strength.

Establishing sound bond between the old and the 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 surface with glues (as epoxy prior concreting), by additional welding of bent reinforcement bars or by formation of reinforced concrete or steel dowels.



Figure 6.6 Strengthening of original structure (Source: MRB & Associates)

Strengthening of original structural elements includes strengthening of:

- *Columns*

The damage of reinforced concrete columns without a structural collapse will vary, such as a slight crack (horizontal or diagonal) without crushing in concrete or damage in reinforcement, superficial damage in the concrete without damage in reinforcement, crushing of the concrete, buckling of reinforcement, or rupture of ties. Based on the degree of damage, techniques such as injections, removal and replaced or jacketing can be provided. Column jacketing can be reinforced concrete jacketing, steel profile jacketing, steel encasement.

The main purpose of column retrofitting is to increase column flexure and shear strength, improving ductility and rearrangement of the column stiffness.

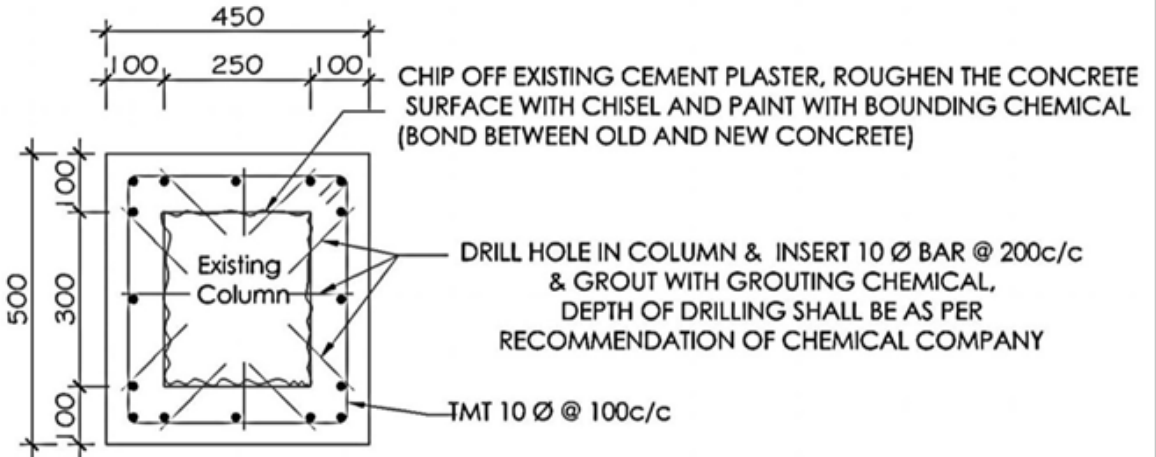


Figure 6.7 Column RC jacketing plan



Figure 6.8 Jacketing of column (source MRB & Associates)

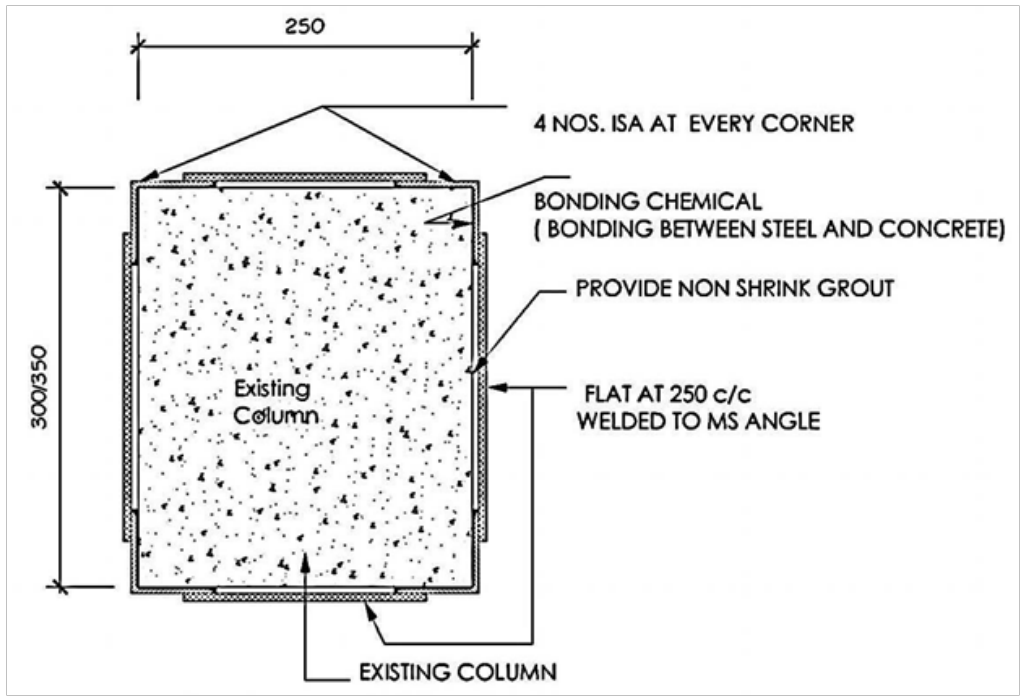
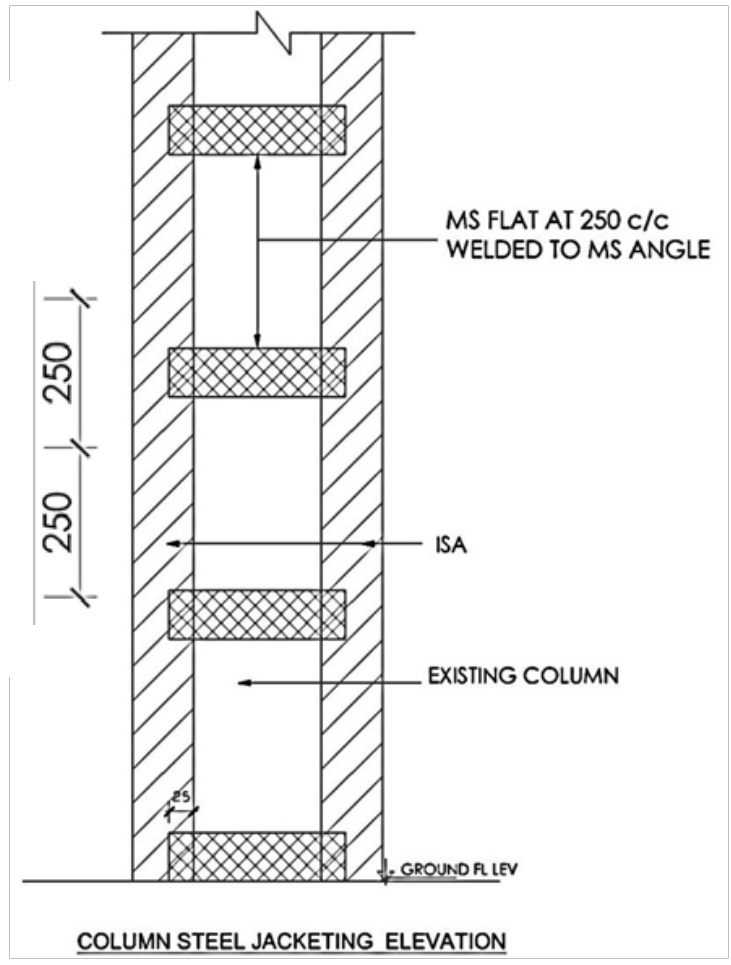


Figure 6.9 Column steel jacketing (a: Elevation, b: plan)



Figure 6.10 Steel jacking of columns (source: MRB & Associates)

• Beams

The aim of strengthening of beams is to provide adequate strength and stiffness of damaged or undamaged beam which are deficit to resist gravity and seismic loads. It is very important that the rehabilitation procedure chosen provides proper strength and stiffness of the beams in relation to adjacent columns in order to avoid creating structures of the “strong beam weak column” type which tend to force seismic hinging and distress into the column, which must also support major gravity loads.

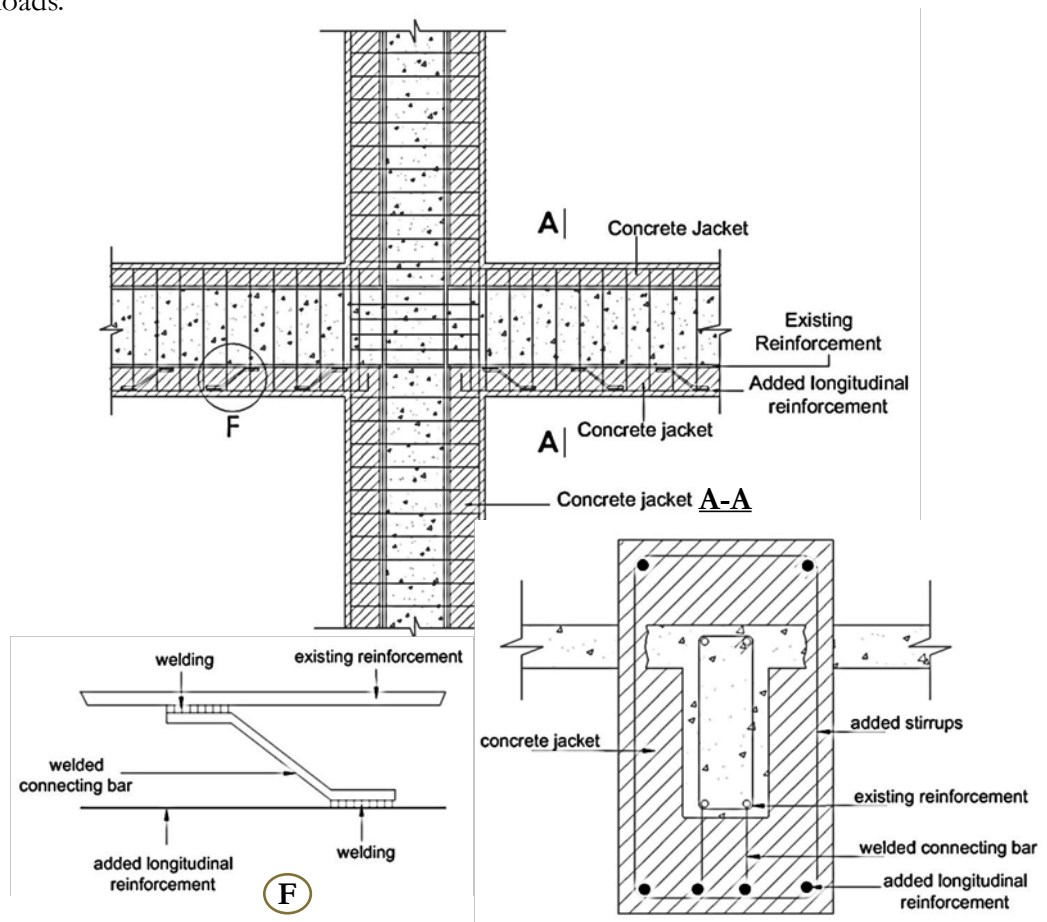


Figure 6.11 RC Jacketing of beam



Figure 6.12 Reinforcement placing for beam jacketing
(source MRB & Associates)



Figure 6.13 Top reinforcement detailing of beam jacketing
(Source MRB & Associates)



Figure 6.14 Beam Jacket
(Source: MRB & Associates)



Figure 6.15 Encasement of existing beam
(Source: MRB & Associates)

• Beam-Column Joints

The most critical region of a moment resisting frame for seismic loading, the beam to the column joint, is undoubtedly the most difficult to strengthen because of the great number of elements assembled at this place and the high stresses this region is subjected to in an earthquake. Under earthquake loading joints suffer shear and/or bond failures.

The retrofitting at the beam column joint can be done using methods like, reinforced concrete jacketing and steel plate reinforcement.

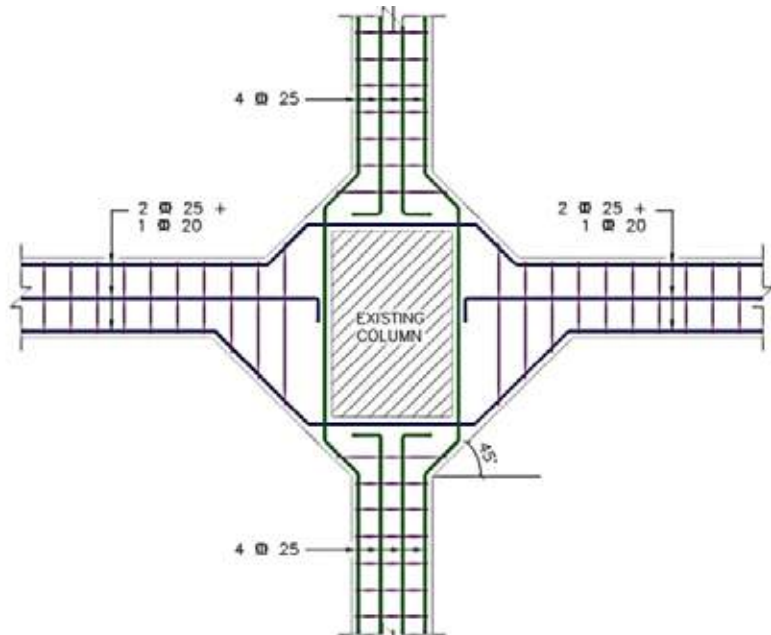


Figure 6.16 Example of beam column joint

• **Concrete Shear wall**

Shear wall possess great stiffness and lateral strength which provides most significant part of the earthquake resistance of the building. Therefore, a severely damaged or a poorly designed shear wall must be repaired or strengthened in order that the structure's strength for seismic force can be significantly improved.

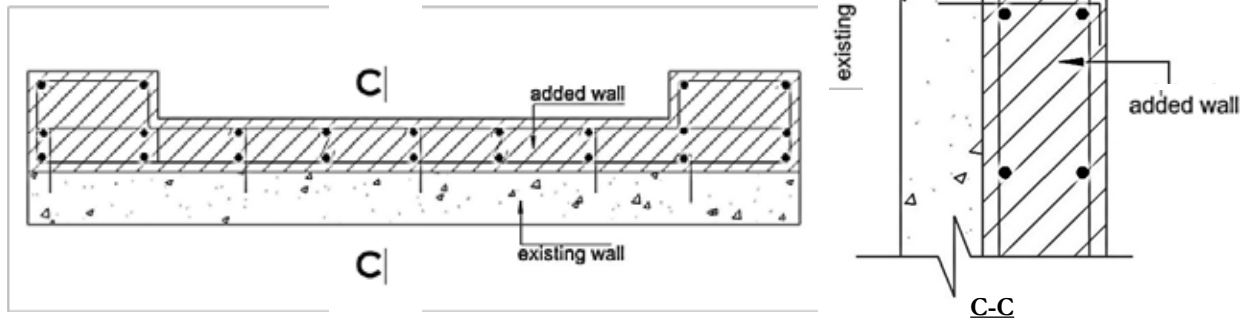


Figure 6.17 Example of shear wall retrofit

• **Slabs**

Primarily, slabs of floor structures have to carry vertical gravity loads. However, they must also provide diaphragm action and be compatible with all lateral resistant element of the structure. Therefore, slab must possess the necessary strength and stiffness. Damages in slabs generally occur due to large openings, insufficient strength and stiffness, poor detailing, etc.

Strengthening of slab can be done by thickening of slabs in cases of insufficient strength or stiffness. For local repairs, injections should be applied for repair of cracks. Epoxy or cement grout can be used.

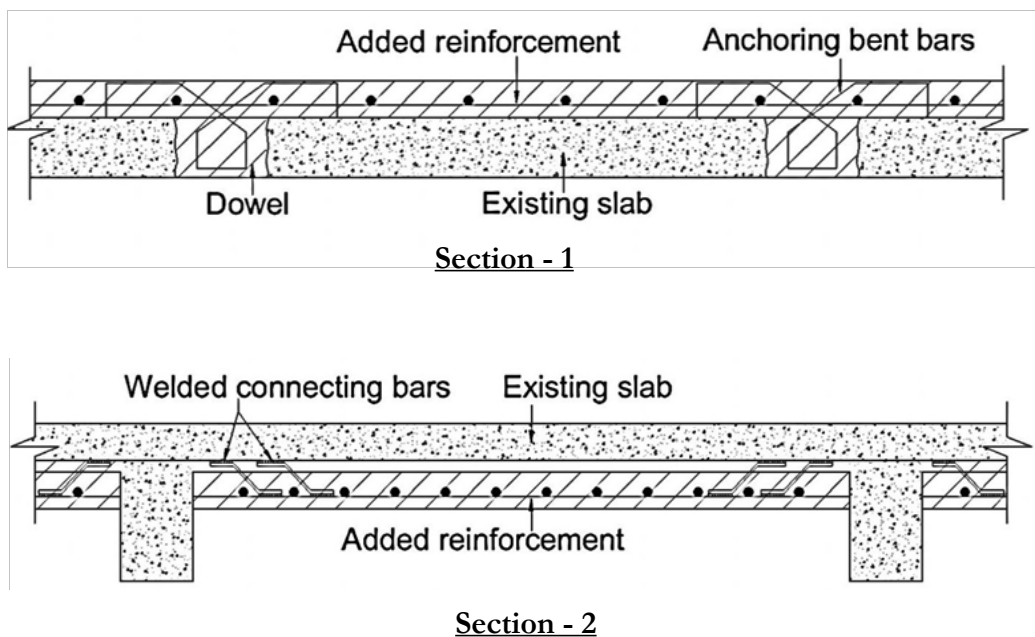


Figure 6.18 Increasing slab thickness

• *Infill Partition wall*

Generally, infilled partition walls in concrete framed buildings are unreinforced although it is highly desirable to be reinforced in seismic region like Nepal. Infilled partition walls in concrete framed buildings often sustain considerable damage in an earthquake as they are relatively stiff and resist lateral forces, often they were not designed to resist, until they crack or fail. Damage may consist of small to large cracks, loose bricks or blocks or an infill leaning sideways. Damage may also result in the concrete frame members and joints which surrounds the infilled wall.

The effect of strengthening an infilled wall must be considered by analysis on the surrounding elements of the structure. Infilled walls are extremely stiff and effective in resisting lateral forces, but all forces must be transferred through the concrete elements surrounding the infilled walls.

• *Foundation*

Retrofitting of foundation is often required when the strength of foundation is insufficient to resist the vertical load of the structure. Strengthening of foundations is difficult and expensive construction procedure. It should be performed in the following cases:

- Excessive settlement of the foundations due to poor soil conditions.
- Damage in the foundation structure caused by seismic overloading.
- Increasing the dead load as a result of the strengthening operations.
- Increasing the seismic loading due to changes in code provisions or the strengthening operations.
- Necessity of additional foundation structure for added floors.

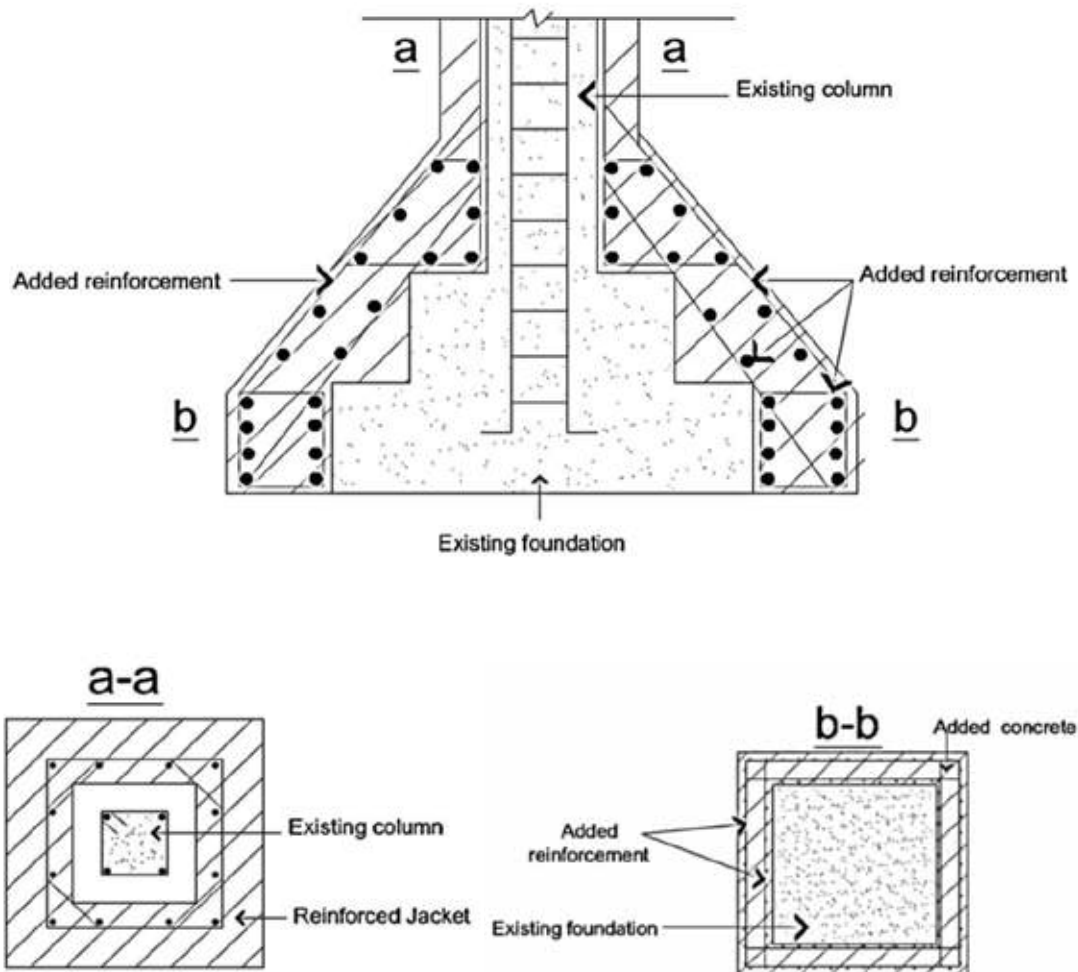


Figure 6.19 Foundation retrofit



Reinforcement layout at foundation for retrofit



Reinforcement layout at foundation for retrofit

Figure 6.20 Reinforcement layout at foundation for retrofit (source: MRB & Associates)



Foundation retrofit with concrete



Reinforcement layout at foundation for retrofit

Figure 6.21 Reinforcement layout at foundation for retrofit (source: MRB & Associates)

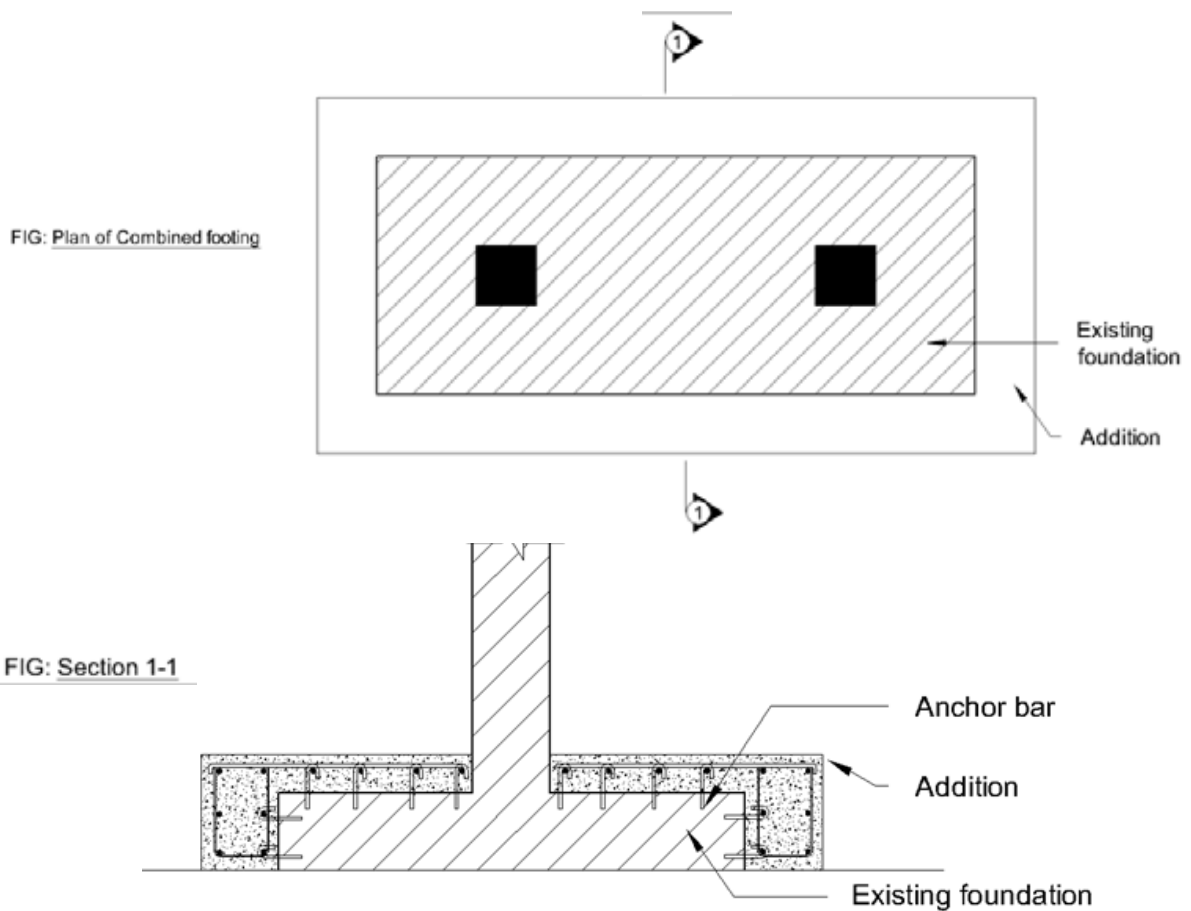


Figure 6.22 Foundation retrofit (Combined)

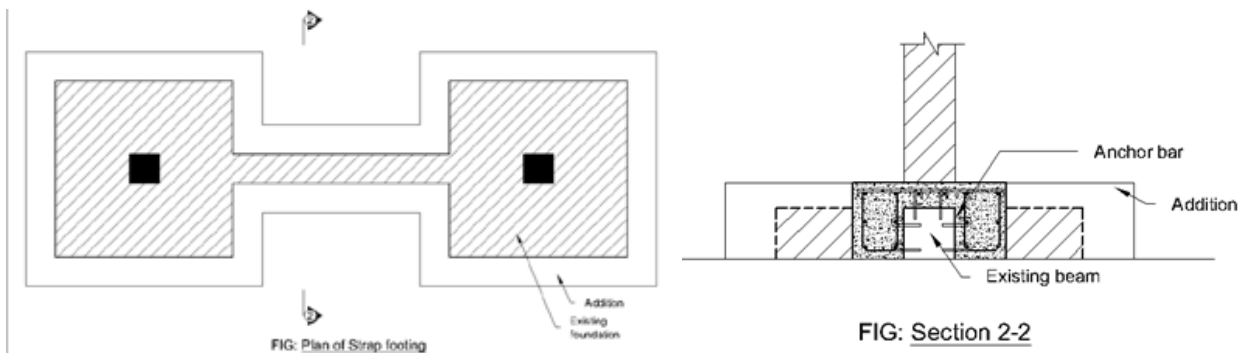


Figure 6.23 Foundation retrofit (Strap)

6.1.9. REDUCING EARTHQUAKE DEMANDS

Rather than modifying the capacity of the building to withstand earthquake-induced forces and deformations, this strategy involves modification of the response of the structure such that the demand forces and deformations are reduced. Irregularities related to distribution of strength, stiffness and mass result poor seismic performance.

The methods for achieving this strategy include reduction in the building's mass and the installation of systems for base isolation and/or energy dissipation. The installation of these special protective systems within a building typically entails a significantly larger investment than do more-conventional approaches. However, these special systems do have the added benefit of providing for reduced demands on building contents.

6.1.9.1. Base Isolation

This approach requires the insertion of compliant bearing within a single level of the building's vertical load carrying system, typically near its base. The bearings are designed to have relatively low stiffness, extensive lateral deformation capacity and may also have superior energy dissipation characteristics. Installation of an isolation system results in a substantial increase in the building's fundamental response period and, potentially, its effective damping. Since the isolation bearings have much greater lateral compliance than does the structure itself, lateral deformation demands produced by the earthquake tend to concentrate in the bearings themselves. Together these effects result in greatly reduced lateral demands on the portion of the building located above the isolation bearings.

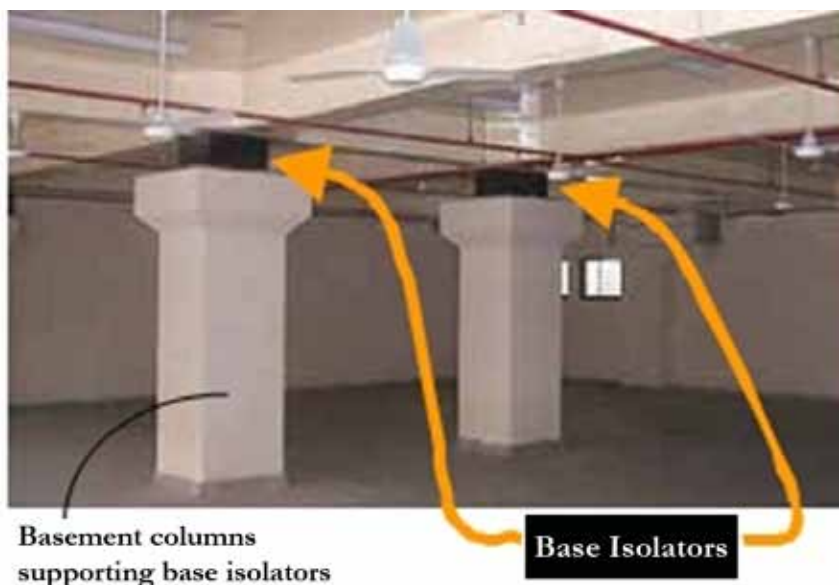


Figure 6.24 Base isolation

6.1.9.2. Energy Dissipation Systems

Energy dissipation systems directly increase the ability of the structure to dampen earthquake response in a benign manner, through either viscous or hysteretic damping. This approach requires the installation of energy dissipation units (EDUs) within the lateral force resisting system. The EDUs dissipate energy and in the process reduce the displacement demands on the structure. The installation of EDUs often requires the installation of vertical braced frames to serve as a mounting platform for the units and therefore, typically results in a simultaneous increase in system stiffness. Energy dissipation systems typically have greater cost than conventional systems for stiffening and strengthening a building but have the potential to provide enhanced performance.

6.1.9.3. Mass Reduction

The performance of some buildings can be greatly improved by reducing the building mass. Building mass reductions reduce the building's natural period, the amount of inertial forces that develops during its response, and the total displacement demand on the structure.

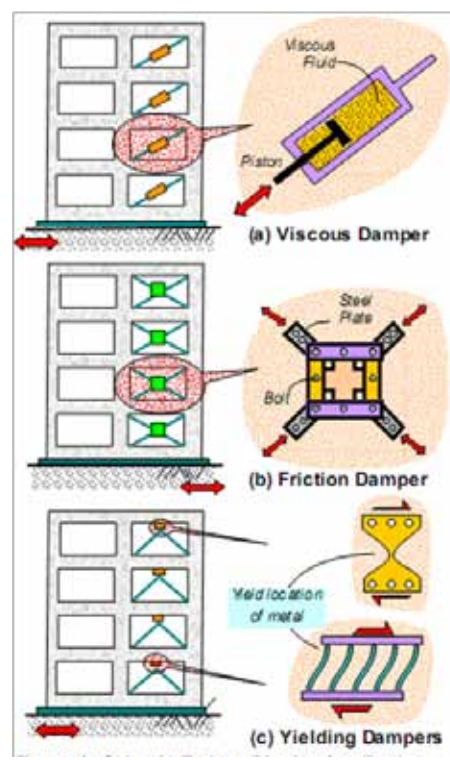


Figure 6.25 Energy dissipation system

Mass can be reduced by removing heavy nonstructural elements such as cladding, water tanks, storage, heavy antenna, etc. In the extreme, mass reduction can be attained by removing one or more building stories.

6.2. STRENGTHENING OPTIONS FOR RC FRAMED STRUCTURES

Members requiring strengthening or enhanced ductility can be jacketed by reinforced concrete jacketing, steel profile jacketing, steel encasement or wrapping with FRP's. Depending on the desired earthquake resistance, the level of the damage, the type of the elements and their connections, members can be strengthened by injections, removal and replacement of damaged parts or jacketing.

- RC jacketing involves placement of new longitudinal reinforcement and transverse reinforcement bars in the new concrete overlay around existing member. Perfect confinement by close, adequate shaped stirrups and ties contributes to the improvement of the ductility of the strengthened members.
- Steel profile jacketing can be done through steel angle profiles placed at each corner of the existing reinforced concrete member and connected together as a skeleton with transverse steel straps.
- Another way is by providing steel encasement with thin plates in existing members. Jacketing with steel encasement is implemented by gluing of steel plates on the external surfaces of the original members. The steel plates acting as reinforcement are glued to the concrete by epoxy resin. This technique doesn't require any demolition. It is considerably easy for implementation and there is a negligible increase in the cross section size of the strengthened members.
- Retrofitting using FRPs involves placement of composite material made of continuous fibers with resin impregnation on the outer surface of the RC member.

6.2.1. RC JACKETING OF COLUMNS

Reinforced concrete jacketing improves column flexure strength and ductility. Closely spaced transverse reinforcement provided in the jacket improves the shear strength and ductility of the column. The procedure for reinforced concrete jacketing are:

- i. The seismic demand on the columns in terms of axial load (P) and moment (M) is obtained.
- ii. The column size and section details are estimated for P and M as determined above.
- iii. The existing column size and amount of reinforcement is deducted to obtain the amount of concrete and steel to be provided in the jacket.
- iv. Increase the amount of concrete and steel actually to be provided as follows to account for losses.
 $A_c = 1.5 A_c'$ and $A_s = 4/3 A_s'$ (IS:15988:2013 Clause 8.5.1.1(e))
Where, A_c and A_s = Actual concrete and steel to be provide in the jacket
 A_c' and A_s' = Concrete and steel values obtained for the jacket after deducting the existing concrete and steel from their respective required amount.
- v. The spacing of ties to be provided in the jacket in order to avoid flexure shear failure of column and provide adequate confinement to the longitudinal steel along the jacket is given as:

$$s = \frac{f_y d h^2}{\sqrt{f_{ck} \cdot t_j}} \quad (\text{IS:15988:2013 Clause 8.5.1.1(f)})$$

Where,

f_y = yield strength of steel

f_{ck} = cube strength of concrete

dh = diameter of stirrup

t_j = thickness of jacket

- vi. If the transfer of axial load to new longitudinal steel is not critical then friction present at the interface can be relied on for the shear transfer, which can be enhanced by roughening the old surface.
- vii. Dowels which are epoxy grouted and bent into 90° hook can also be employed to improve the anchorage of new concrete jacket.
The minimum specifications for jacketing of columns are:
 - a. Strength of the new materials must be equal or greater than those of the existing column. Concrete strength should be at least 5MPa greater than the strength of the existing concrete.
 - b. For columns where extra longitudinal reinforcement is not required, a minimum of 12 ϕ bars in the four corners and ties of 8 ϕ @ 100 c/c should be provided with 135° bends and 10 ϕ leg lengths.
 - c. Minimum jacket thickness should be 100mm.
 - d. Lateral support to all the longitudinal bars should be provided by ties with an included angle of not more than 135°.
 - e. Minimum diameter of ties should be 8mm and not less than 1/3 of the longitudinal bar diameter.
 - f. Vertical spacing of ties shall not exceed 200 mm, whereas the spacing close to the joints within a length of 1/4 of the clear height should not exceed 100 mm. Preferably, the spacing of ties should not exceed the thickness of the jacket or 200 mm whichever is less.

Option 1:

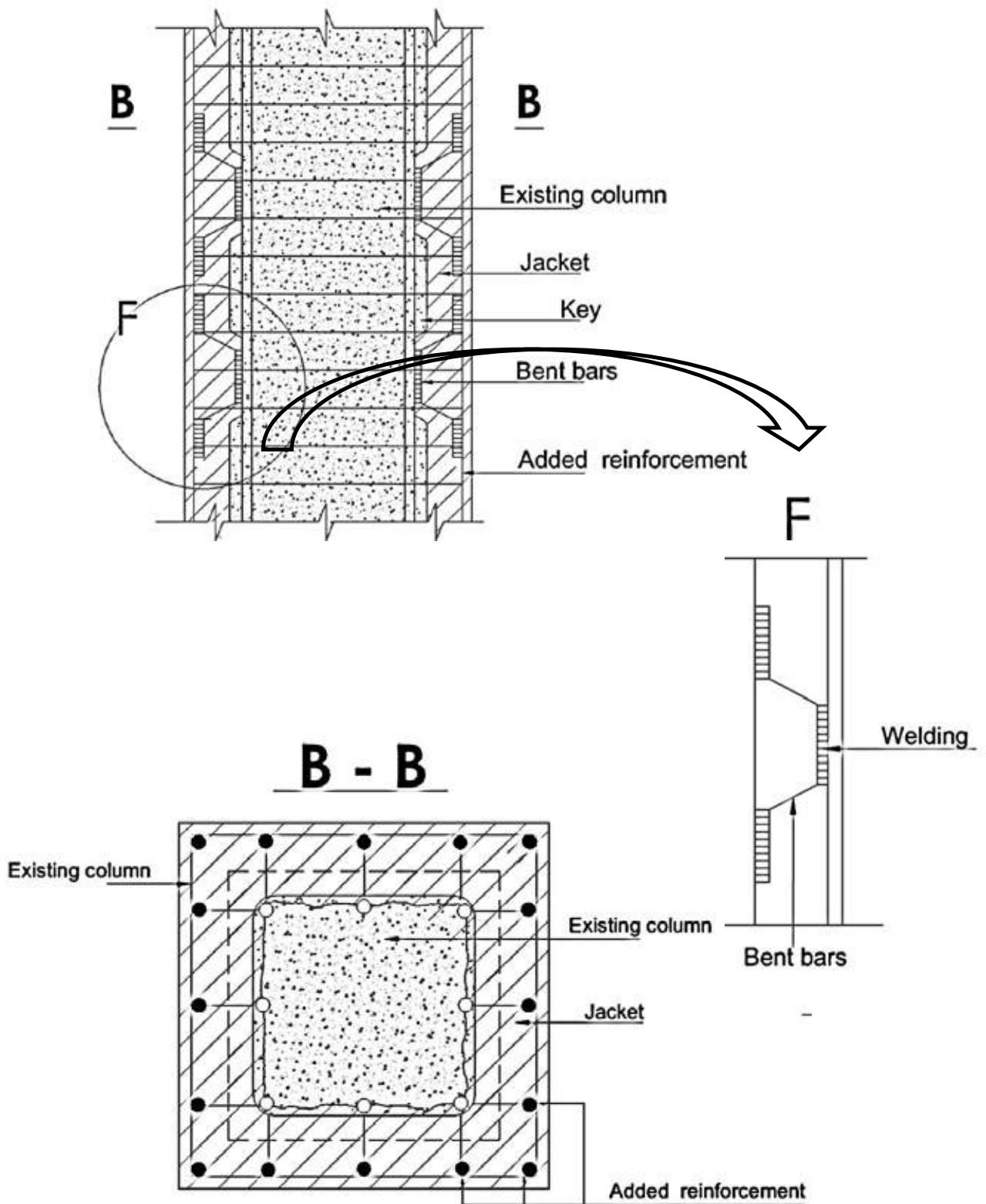


Figure 6.26 Column jacking with reinforced concrete- option 1

Option 2:

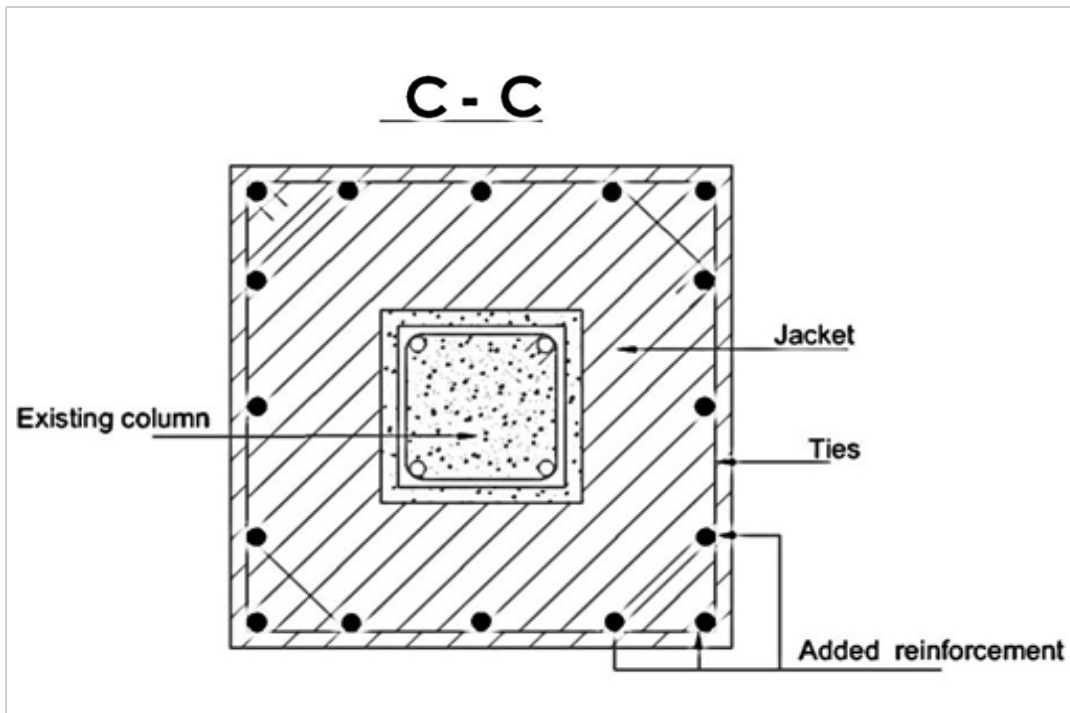
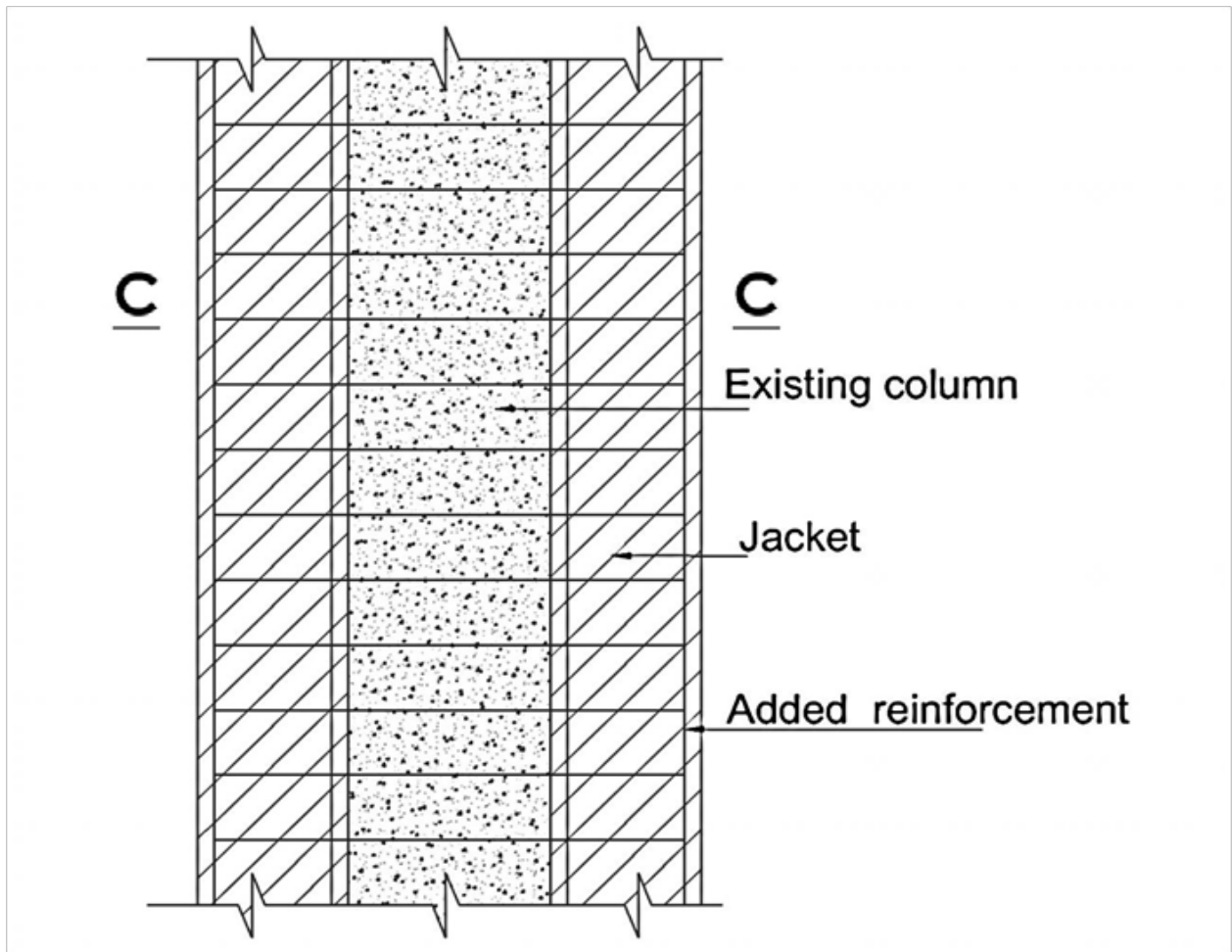


Figure 6.27 Column jacketing with reinforced concrete- option 2

6.2.2. STEEL JACKETING OF COLUMNS

Steel profile skeleton jacketing consists of four longitudinal angle profiles placed one at each corner of the existing reinforced concrete column and connected together in a skeleton with transverse steel straps. They are welded to the angle profiles and can be either round bars or steel straps. The angle profile size should be no less than $L 50 \times 50 \times 5$. Gaps and voids between the angle profiles and the surface of the existing column must be filled with non-shrinking cement grout or resin grout. In general, an improvement of the ductile behavior and an increase of the axial load capacity of the strengthened column is achieved.

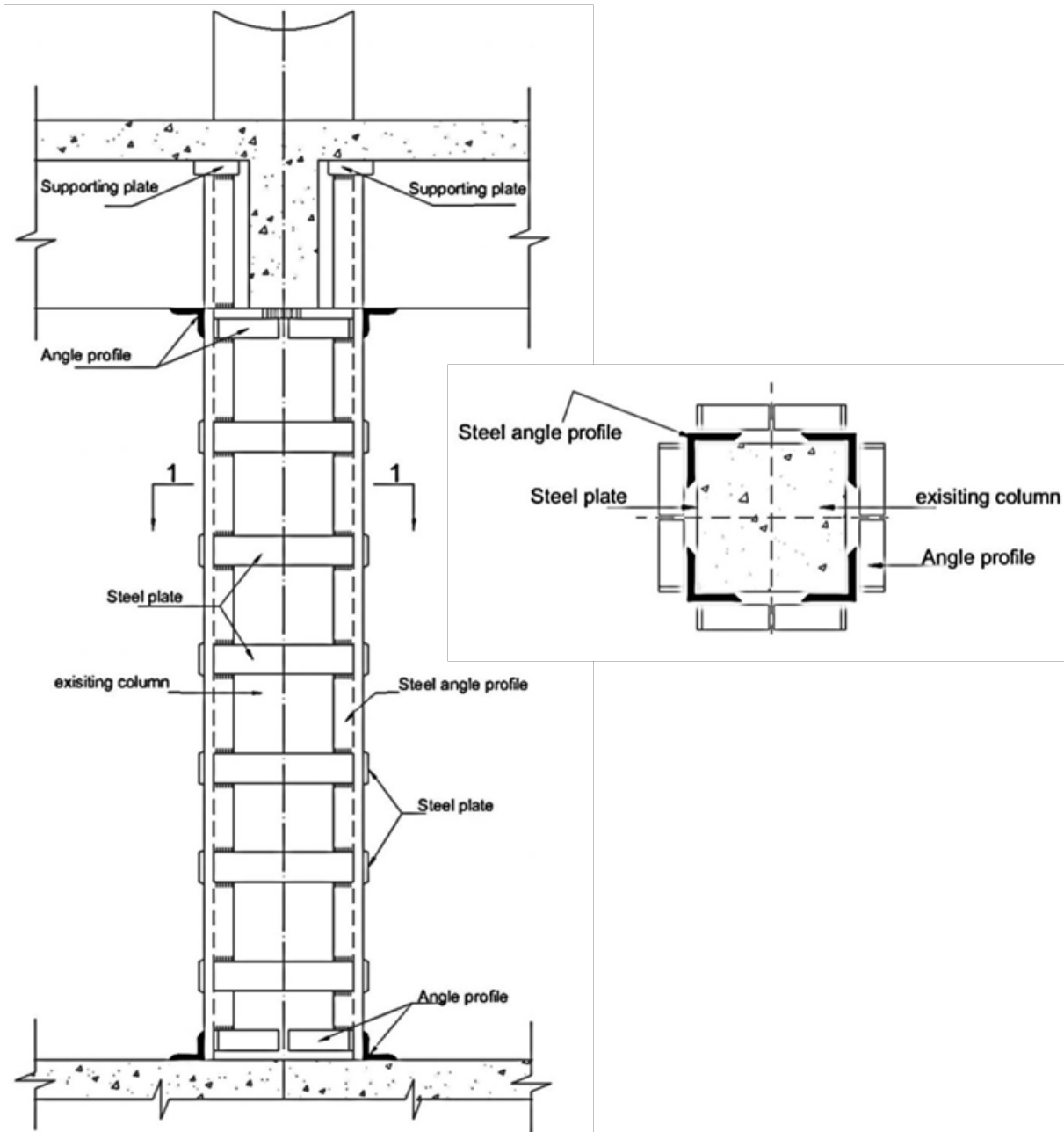


Figure 6.28 Steel jacketing of columns

6.2.3. ADDITION OF RC SHEAR WALL

The addition of new reinforced concrete shear walls provides the best option of strengthening an existing structure for improved seismic performance. It adds significant strength and stiffness to framed structures.

The minimum anchorage length of the grouted-in longitudinal and transverse reinforcement of the shear wall in to the existing components of the building shall not be less than 6 times the diameter of the bars.

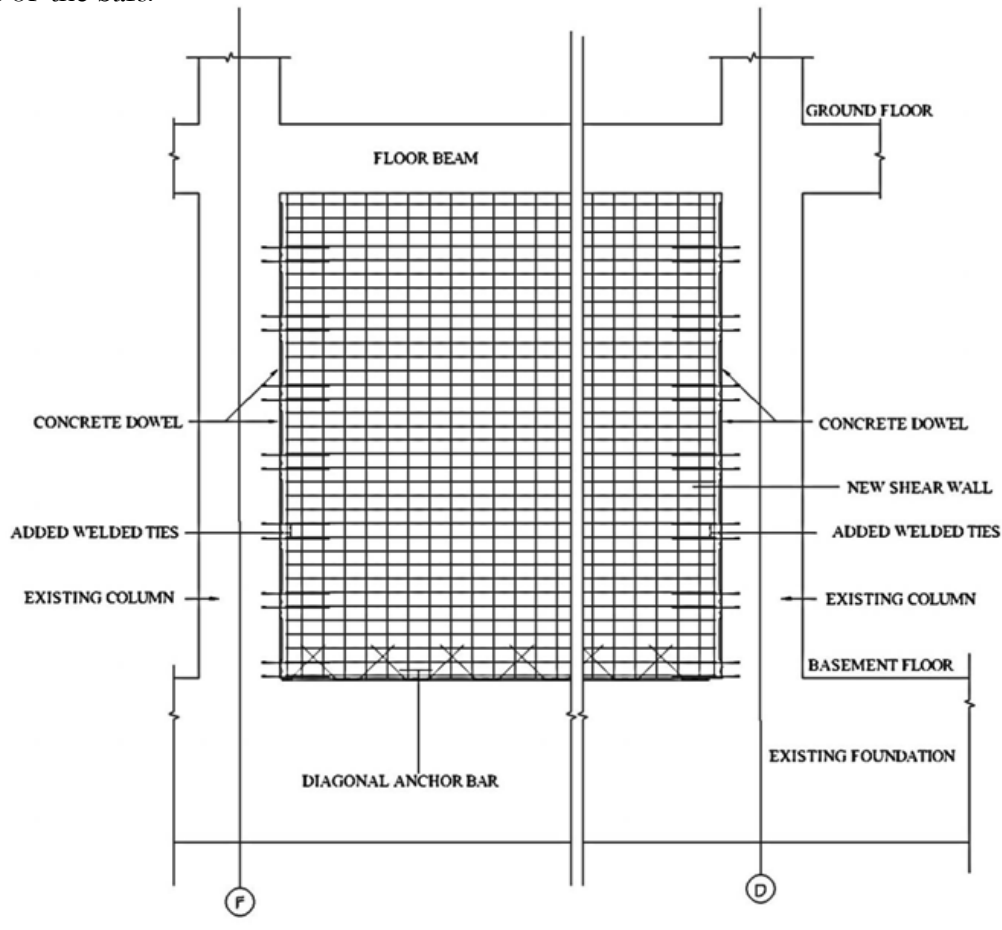


Figure 6.29 Addition of shear wall

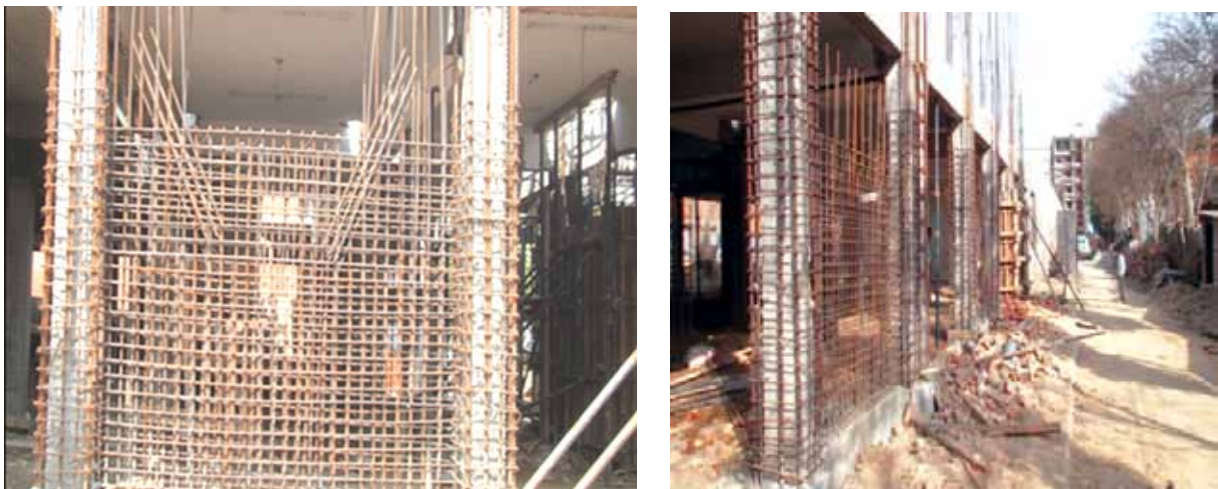


Figure 6.30 Shear wall addition with column jacketing (source: MRB & Associates)

6.2.4. ADDITION OF STEEL BRACING

Steel diagonal braces can be added to the existing concrete frames. Braces should be arranged so that their center line passes through the centers of the beam – column joints. The brace connection should be adequate against out-of-plane failure and brittle fracture.



Figure 6.31 Addition of steel bracing

6.3. RECOMMENDED DETAILING FOR EARTHQUAKE RESISTANCE BUILDING



Figure 6.32 Beam column joint detailing (source: MRB & Associates)



Figure 6.33 Confining hoop made with single reinforcing bar (source: MRB & Associates)

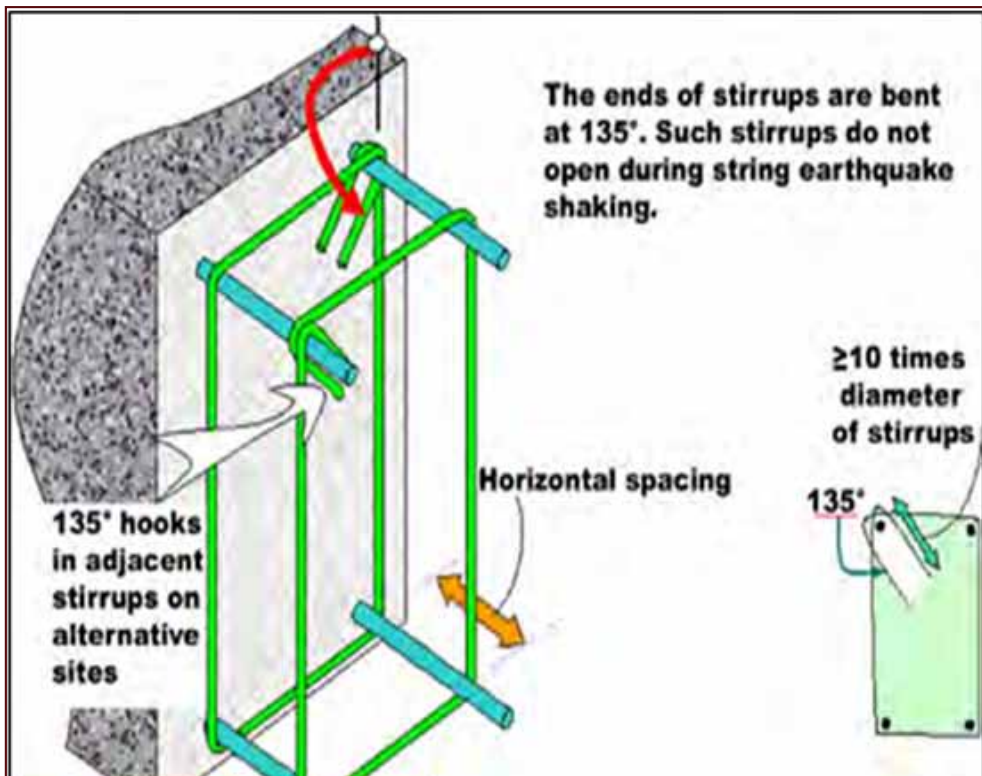


Figure 6.34 Stirrup detailing

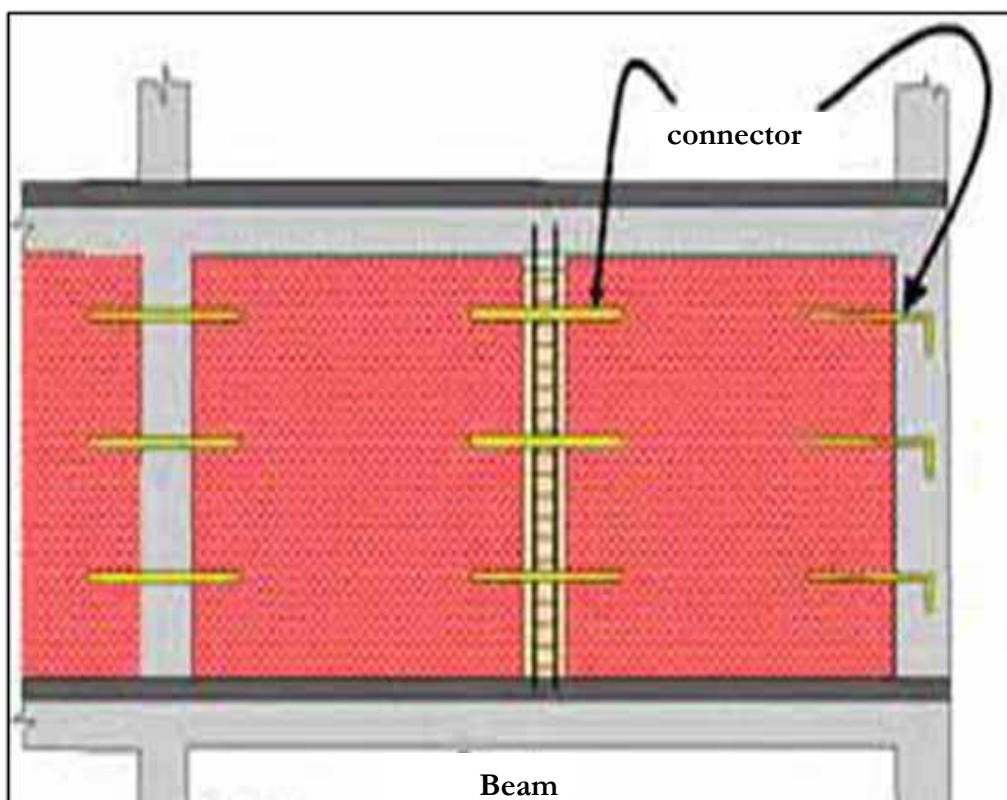


Figure 6.35 Detail of anchor between infill and the frame

7. STRUCTURAL VULNERABILITY ANALYSIS

Structural vulnerability analysis is very important to protect building or structures from damage. The structural vulnerability assessment is necessary due to many reasons. Structural vulnerability analysis may be necessary in Engineered or Non Engineered building; for many reasons, some of the reasons are listed below:

- Occupancy Change in the building
- Construction quality not appropriate
- Client interest
- Revision in the code
- Structural material degradation, etc.

1. **Engineered Building:**

In this category, buildings are designed with reference to codes and in the guidance of Engineer or Technical persons. But, vulnerability analysis or retrofitting may be required due to several reasons such as listed above.

2. **Non Engineered Building:**

In this category, buildings are built informally. These types of buildings are common in context of Nepal. These building are not structurally designed and supervised by engineers during construction. An example of structural vulnerability assessment and retrofit of engineered building has been demonstrated in this guideline in Annex A.

ANNEX A: STRUCTURAL VULNERABILITY ANALYSIS

EXAMPLE 1: ENGINEERED RC FRAME BUILDING

A.1. ENGINEERED RC FRAME BUILDING

A.1.1. BUILDING DESCRIPTION

This building is RCC Frame structure in burnt clay bricks in cement mortar. The structure is 5-storey + 1-Basement with storey height of 4m and 3.8m. The floor consists of reinforced concrete slab system. The total height of the building is 25.28m. There are 230mm thick outer walls and light weight partition wall as inner walls.

This building is engineered building with sufficient column size and beam size.

Vulnerability analysis was done and retrofitting is recommended to correct L- shape by adding shear wall or retrofitting columns for torsion. Finally, comparisons of different retrofitting options are done to select the most appropriate retrofitting option.

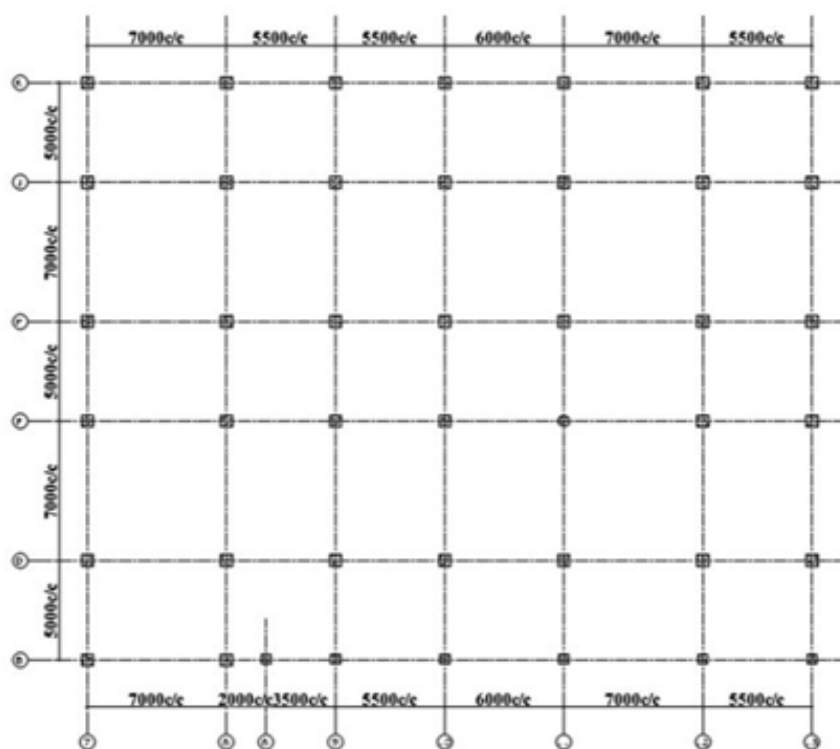
A.1.1.1. General Building Description

Building Plan Size	:	40.51m × 33m
No. of Story above ground level	:	5
No. of basement below ground level	:	1
Building Height	:	25.28m
Storey height	:	3.8m

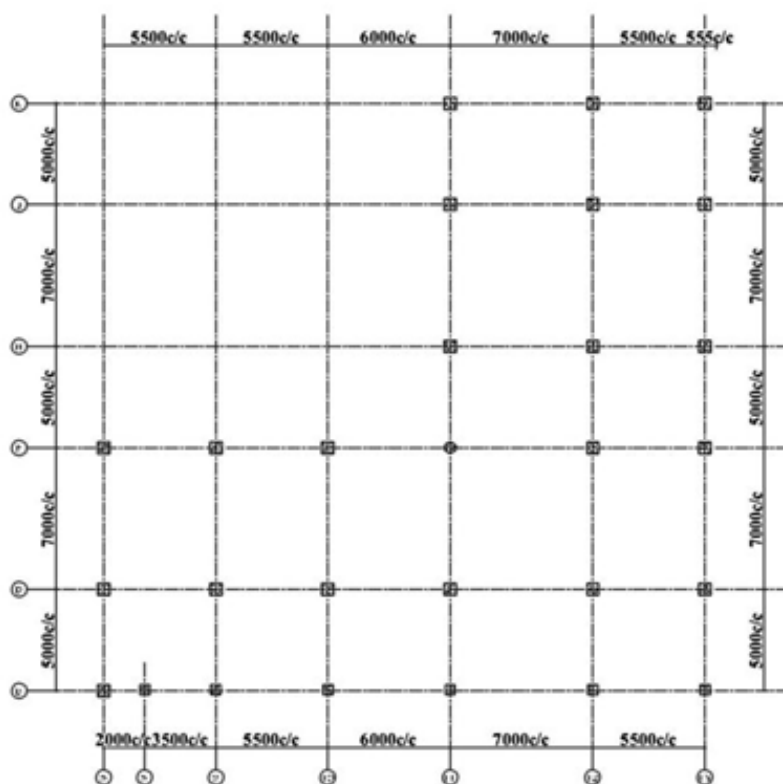
A.1.1.2. Structural System Description

Type of Structure	:	R.C Frame
Type of Foundation	:	Beam slab Footing
Roof Type	:	Sloped roof with clay tile
Column Sizes	:	400mm × 400mm , 500mm × 500mm, 600mm × 600mm, 700mm dia, 500mm dia, 600mm dia.
Beam Sizes	:	300mm × 550mm
Building Type	:	Building Type IV
Seismic Zone	:	1 (NBC 105:1994)

A.1.1.3. Building Drawings



BASEMENT PLAN



TYPICAL FLOOR PLAN

Figure A-1 Building drawings

A.1.2. ASSUMPTIONS

Unit weight of RCC = 25 KN/m³

Unit weight of brick = 19.6 KN/m³

Live load = 3.0 KN/m²

Weight of plaster and floor finish = 0.73 KN/m² i.e. 22mm screed + 12mm plaster

Partition load = 1.2 KN/m²

Grade of concrete = M20 for all the other structural elements

Grade of steel = Fe 415

Lateral load is solely carried by frame elements. Stiffness of the walls is not considered.

A.1.3. STRUCTURAL ASSESSMENT CHECKLIST

S.N.	CHECKS	REMARKS
1.	Load Path	The frame system provides a complete load path which transfers all inertial forces in the building to the foundation.
2.	Redundancy	There are more than two bays of frame in each direction.
3.	Geometry	The plan of the building is same in all stories except at basement. The building has basement for parking.
4.	Weak Storey / Soft Storey	There is no weak / soft storey.
5.	Vertical Discontinuities	Vertical elements in the lateral force resisting system are continuous to the foundation. Except for the basement columns.
6.	Mass	There is no change in effective mass in adjacent floors except at basement to ground floor.
7.	Torsion	The eccentricity of the building is not within the limit.
8.	Adjacent Buildings	There are no adjacent buildings.
9.	Short Column	No short column effect
10.	Deterioration of Concrete	No visible deterioration observed. No cracks were observed.

A.1.4. STRENGTH RELATED CHECKS

A.1.4.1. Calculation for Shear Stress Check

Lumped load

LEVEL	Combination DL+0.25LL	Seismic Weight (KN)
6.00	6342.59	6342.59
5.00	6073.94	6073.94
4.00	6124.08	6124.08
3.00	6132.29	6132.29
2.00	6068.88	6068.88
1.00	15717.80	15717.80
	∑	<u>46459.57</u> KN

A.1.4.2. Calculation of Base Shear (Using NBC 105:1994)

Design Horizontal Seismic Coefficient for the Seismic Coefficient Method

The design horizontal seismic force coefficient, C_d shall be taken as :

$$C_d = C Z I K$$

Where, C is the basic seismic coefficient for the fundamental translational period in the direction under consideration.

Z = Seismic Zoning Factor

I = Importance Factor

K = Structural Performance Factor

The total design lateral force or Design Seismic Base Shear (V_B) along any principal direction is determined by the following expression :

$$V_B = C_d W_t$$

Where, C_d = The Design Horizontal Seismic Coefficient

W_t = Total of the gravity loads of the whole building

The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

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

Where,

h = Height of Building in meter = 25.58m

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

dx = 40.51m

dz = 33m

$$T_{ax} = \frac{0.09h}{\sqrt{dx}} = 0.3617$$

$$T_{az} = \frac{0.09h}{\sqrt{dz}} = 0.4$$

Therefore, C = 0.08 for medium soil

Seismic zoning factor for Kathmandu is, Z = 1.0

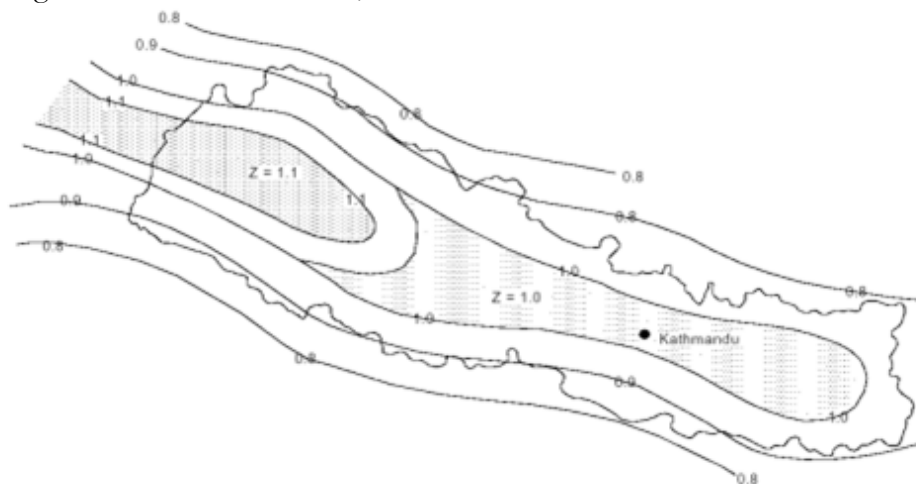


Figure A-2 Seismic zone for Kathmandu

$$\begin{aligned}
C_d &= C Z I K \\
&= 0.08 \times 1 \times 1 \times 1 \\
&= 0.08
\end{aligned}$$

$$\begin{aligned}
\text{Base shear} &= V_b = C_d W_t \\
&= 3716.766 \text{ KN}
\end{aligned}$$

A.1.4.3. Distribution of Base Shear And Calculation Of Shear Stress In RC Columns

The horizontal seismic force at each level i shall be taken as :

The design base shear (V) computed in 1.5 shall be distributed along the height of the building as per the following expression:

$$F_i = V \times \frac{W_i h_i}{\sum W_i h_i}$$

Where,

W_i = proportion of W_t contributed by level i,
 h_i = Height of floor i measured from base

Floor	Total weight W_i (KN)	Height h_i (m)	$W_i \times h_i$	$\frac{W_i h_i}{\sum W_i h_i}$	Q_i (KN)	Storey Shear V_i (KN)
6	6342.59	23.000	145879.64	0.271	1007.521	1007.521
5	6073.94	19.200	116619.69	0.217	805.436	1812.957
4	6124.08	15.400	94310.77	0.175	651.359	2464.317
3	6132.29	11.600	71134.52	0.132	491.292	2955.609
2	6068.88	7.800	47337.23	0.088	326.936	3282.544
1	15717.80	4.000	62871.19	0.117	434.221	3716.766
Σ			538153.04			

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings, 6.5.1)

Average Shearing stress in columns is given as

$$T_{col} = \left(\frac{n_c}{n_c - n_f} \right) \times \left(\frac{v_j}{A_c} \right) < \text{min. of } 0.4 \text{ Mpa and } 0.1 \sqrt{f_{ck}} \text{ (Ref IS:15988:2013 Clause 6.5.1)}$$

$$0.1 \sqrt{f_{ck}} = 0.1 \sqrt{20} = 0.45$$

For Ground Storey columns,

n_c = Total no. of Columns resisting lateral forces in the direction of loading

n_f = Total no. of frames in the direction of loading

A_c = Summation of the cross- section area of all columns in the storey under consideration

V_j = Maximum Storey shear at storey level 'j'

DCR = Demand Capacity Ratio

Storey	n _c	n _n	n _{t2}	A _c	Storey Shears	Shear Stress (σ)		DCR		Remarks
						Colx (MPa)	Colz (MPa)	in x-dir	in z-dir	
6	28	6	6	8.737	1007.52	0.15	0.15	0.37	0.37	Since Demand Capacity Ratio is less than 1, hence Safe in shear
5	28	6	6	8.737	1812.96	0.26	0.26	0.66	0.66	
4	28	6	6	8.823	2464.32	0.36	0.36	0.89	0.89	
3	28	6	6	9.593	2955.61	0.39	0.39	0.98	0.98	
2	28	6	6	10.80	3282.54	0.39	0.39	0.97	0.97	
1	43	6	7	17.12	3716.77	0.25	0.26	0.63	0.65	

$$\tau_{col} < \min \text{ of } 0.4 \text{ MPa}$$

Hence the check is satisfied

A.1.4.4. Axial Stress Check

Axial Stresses Due To Overturning Forces As Per FEMA310 (Clause 3.5.3.6)

i. Axial stress in moment frames for x-direction loading

Axial force in columns of moment frames at base due to overturning forces, The axial stress of columns subjected to overturning forces F_o is given by

$$F_o = \frac{2}{3} \left(\frac{V_b}{n_f} \right) \times \left(\frac{H}{L} \right)$$

$$\begin{aligned} V_b &= \text{Base shear x Load Factor} \\ &= 3716.8 \times 1.5 = 5575.15 \text{ KN} \\ A_c &= \text{column area} = 17.12 \text{ Sq.m.} \\ H &= \text{total height} = 24 \text{ m} \\ L &= \text{Length of the building} = 40.51 \text{ m} \end{aligned}$$

$$\begin{aligned} F_o &= \frac{2}{3} \left(\frac{V_b}{n_f} \right) \times \left(\frac{H}{L} \right) \\ &= 275.25 \text{ KN} \end{aligned}$$

Axial Stress for x-direction loading,

$$\sigma = \frac{275.25 \times 1000}{0.16} = 1.72 \text{ MPa}$$

$$\sigma_{all} = 0.25 f_{ck} \text{ (Reference IS15988:2013 Clause 6.5.4)} = 5.00 \text{ MPa}$$

therefore $\sigma < \sigma_{all}$ OK

DCR = 0.334

ii. Axial stress in moment frames for z-direction loading

Axial force in columns of moment frames at base due to overturning forces, The axial stress of columns subjected to overturning forces F_o is given by

$$F_o = \frac{2}{3} \left(\frac{V_b}{n_f} \right) \times \left(\frac{H}{L} \right)$$

V_b = Base shear x Load Factor Load Factor = 1.5 (Ref: IS456:2000)
 3716.8 x 1.5 = 5575.15 KN

A_c = Column area = 17.12 sq.m.
 H = Total height = 24 m
 L = Length of the building = 33.00 m

$$F_o = \frac{2}{3} \left(\frac{V_b}{n_f} \right) \times \left(\frac{H}{L} \right)$$

= 337.89 KN

Axial Stress for z-direction loading,
 $\sigma = \frac{337.89 \times 1000}{0.16} = 2.11 \text{ MPa}$

$\sigma_{all} = 0.25 f_{ck}$ (Reference IS15988:2013 Clause 6.5.4) = 5.00 MPa

therefore $\sigma < \sigma_{all}$ OK

DCR = 0.422 < 1

Hence the check is satisfied

A.1.4.5. Check for Out-Of-Plane Stability of Brick Masonry Walls

Wall Type	Wall Thickness	Recommended Height/ Thickness ratio (0.24 < Sx ≤ 0.35) (Ref: Fema:310 Table 4-2)	Actual Height/ Thickness ratio in building	Comments
Wall in ground storey	230mm	18	$\frac{(3800 - 450)}{230} = 14.56$	Pass
Wall in upper stories	230mm	16	$\frac{(3800 - 450)}{230} = 14.56$	Pass

The out of plain stability of ground floor wall and that for the upper stories are within the permissible limit, hence the check is satisfied.

A.1.5. DETAILED ANALYSIS

A.1.5.1. Column Flexure Capacity

Calculating the column bending capacity for ground storey column:

The column demand given by load case with maximum value is:

$$\begin{aligned}
P_u &= 2572.5 \text{ KN} \\
M_u &= 517 \text{ KNm} \\
f_{ck} &= 20 \text{ Mpa} \\
f_y &= 415 \text{ Mpa} \\
\text{Clear cover} &= 40 \text{ mm} \\
d' &= 40 + 10 + \frac{25}{2} = 62.5 \\
\frac{d'}{D} &= 0.104 \approx 0.1 \\
A_s &= 4555.278 \text{ mm}^2
\end{aligned}$$

Percentage of reinforcement,

$$\begin{aligned}
\rho &= 1.265\% \\
\frac{\rho}{f_{ck}} &= 0.063 \\
\frac{P_u}{(f_{ck}bd)} &= \frac{2572.51}{(20 \times 600 \times 600)} \\
&= 0.0378
\end{aligned}$$

Referring to chart 44 of SP:16,

$$\begin{aligned}
\frac{M'_u}{(f_{ck}bD^2)} &= 0.095 \\
M'_u &= 410.4 \text{ KNm} \\
\text{DCR} &= 1.259 > 1
\end{aligned}$$

Hence the check is not satisfied.

A.1.5.2. Shear Capacity of Column

Considering that the steel in one face will be in tension,

$$\begin{aligned}
A_s &= \frac{3 \times \pi \times \frac{25^2}{4}}{4} \\
&= 1472.62 \text{ mm}^2
\end{aligned}$$

$$\text{Therefore, } 100A_s/bd = 100 \times 1472.62 / 600 \times 537.5 = 0.456$$

For $P_t = 0.456$ and M20 grade of Concrete, From IS456:2000 Table 19 $\sigma_c = 0.47 \text{ Mpa}$
Stirrups are 4- legged, 10mm \varnothing @ 200mm c/c spacing

Then,

$$\begin{aligned}
V_{us} &= \frac{0.87 \times f_y \times A_{st} \times d}{s_v} \quad \text{Ref:IS:456:2000 Clause 40.4} \\
&= \frac{0.87 \times 415 \times 314.16 \times 537.5}{200} \\
&= 304 \text{ KN}
\end{aligned}$$

$$\begin{aligned}
\text{Therefore, } V_u &= V_{us} + T'bd \quad (\text{Ref:IS:456:2000 Clause 40.4}) \\
&= 456 \text{ KN}
\end{aligned}$$

$$\text{Shear force per analysis} = 332 \text{ KN}$$

Moment Capacity of Beam

$$M_{u,lim}^{bR} = 0.138 * f_{ck} * b * d^2 \text{ (Ref IS 456 Annex G)} = 194.27 \text{ kNm}$$

$$M_{u,lim}^{bL} = 0.138 f_{ck} b d^2 \text{ (Ref IS 456 Annex G)} = 497.67 \text{ kNm}$$

$$h_{st} = 3.8 \text{ m}$$

V from capacity design (IS13920)

$$= V_u = 1.4 \times \frac{M_{u,lim}^{bL} + M_{u,lim}^{bR}}{h_{st}}$$

Hence, $V_u = 254.925 \text{ KN}$

So, Final shear demand = 332 KN

$V_u (=456 \text{ KN}) > \text{Shear demand}$

DCR = 0.728

Hence, the check is satisfied.

A.1.5.3. Shear Capacity of Beam

The shear reinforcement provided in the existing beam at support section is 2-legged 10Φ @ 100mm c/c.

$$A_s = 4 \cdot 20\Phi = 1257 \text{ mm}^2$$

$$P_t = \frac{100 A_s}{b d} = \frac{100 \times 1257}{300 \times 515} = 0.813\%$$

Using table 19 of IS456:2000, for M20 grade of concrete and $\frac{100 A_s}{b d} = 0.813$,
 $r_c = 0.575 \text{ MPa}$

Stirrups are 2-legged 10Φ @ 100mm c/c, hence from cl. 40.4 of IS456:2000

$$V_{us} = \frac{0.87 \times f_y \times A_{st} \times d}{S_v}$$

$$V_u = V_{us} + r_c b d$$

$$= \frac{0.87 \times 415 \times 2 \times 78.57 \times 515}{100} + 0.575 \times 300 \times 515 = 381.0 \text{ KN}$$

Shear Demand in beam:

V as per analysis = 293.9kN

Moment capacity of beam

$$M_R^H = 194.27 \text{ kNm}$$

$$M_R^S = 497.67 \text{ kNm}$$

$$L_c = 7 - 0.6 = 6.4 \text{ m}$$

$$V_a^{D+L} = V_b^{D+L} = 126 \text{ kN}$$

$$\begin{aligned}
 & V \text{ from capacity design (IS13920)} \\
 & = V_u = 126 + 1.4 \times \frac{(M_R^H + M_R^S)}{L_c} \\
 & = 277.36 \text{ kN}
 \end{aligned}$$

Hence final shear demand in beam = 293.9kN
 $V_u (=381\text{KN}) > 293.9 \text{ KN}$
 $\text{DCR} = 0.771 < 1$

Hence, the check is satisfied.

A.1.5.4. Check for Strong Column Weak Beam

The flexure strengths of the columns shall satisfy the condition:

From IS15988:2013 7.4.1 , $\sum M_c \geq 1.1 \sum M_b$

- **Checking Capacity of Center Column at Ground Floor:**

The longitudinal beam of size 300×550 is reinforced with 3-20dia. + 3-25dia. (i.e 2415.09mm²) at top and 4-20 dia. (ie 1256.636mm²) at bottom.

Where,

$$b = 300\text{mm}; d = 515\text{mm}$$

The hogging and sagging moment capacities are evaluated as 303.406 KNm and 194.27 KNm respectively.

Factored column axial load = 4770 KN(1.2DL + 1.2Eqz + 1.2LL)

$$\frac{P_u}{(f_{ck}bd)} = 0.6625 \text{ where column size is } 600 \text{ mm} \times 600 \text{ mm}$$

The column is reinforced with 8-25dia. + 2-20dia.

$$A_{sc} = 4555.278\text{mm}^2; p_t = 1.265\% \text{ Referring to chart SP16 Clause 3.0 Chart}$$

Therefore,

$$\begin{aligned}
 \frac{M'_u}{(f_{ck}bD^2)} &= 0.01 \\
 M'_u &= 43.2\text{kNm} \\
 \sum M_c &= 43.2 + 43.2 = 86.4\text{KNm} \\
 \sum M_b &= 303.406 + 194.27 = 497.676\text{KNm} \\
 1.1\sum M_b &= 547.437\text{KNm}
 \end{aligned}$$

$$\sum M_c \lll 1.1 \sum M_b$$

Hence, check is not satisfied.

- **Checking Capacity of Center Column of Peripheral Frame at Ground Floor:**

The longitudinal beam of size 300×550 is reinforced with 3-20dia. + 2-25dia. (ie 1923.778mm^2) at top and 3-20 dia. (ie 942.477mm^2) at bottom.

Where,

$$b = 300\text{mm}; d = 515\text{mm}$$

The hogging and sagging moment capacities are evaluated as 265.3 KNm and 153.1 KNm respectively.

Factored column axial load = 2906.68 KN

$$\frac{P_u}{(f_{ck}bd)} = 0.404 \text{ where column size is } 600 \text{ mm} \times 600 \text{ mm}$$

The column is reinforced with 8-25dia.

$$A_{sc} = 3928.56\text{mm}^2 ; p_t = 1.09\% \text{ from SP-16}$$

Therefore,

$$\frac{M'_u}{(f_{ck}bD^2)} = 0.065$$

$$M_u = 280.8 \text{ KNm}$$

$$\sum Mc = 280.8 + 280.8 = 561.6 \text{ KNm}$$

$$\sum Mb = 265.3 + 153.1 = 418.4 \text{ KNm}$$

$$1.1\sum Mb = 460.24 \text{ KNm}$$

$$\text{From IS15988:2013 } 7.4.1 \sum Mc > 1.1 \sum Mb$$

Hence, check is satisfied.

A.1.6. EVALUATION SUMMARY

- The building is safe in strength related checks such as shear stress capacity, axial stress, out of plane stability.
- The computer analysis of the structure shows:
 - Foundation: Safe
 - Beam : Safe
 - Column : Not Safe
(The DCR lies in the range of 1.5 indicating more detailed analysis)
 - Floor slab: Safe
- Thus, the above evaluations state that the frame has to be strengthened and retrofitted.

A.1.7. RETROFITTING OPTIONS

A.1.7.1. Option1: RC Jacketing On Columns

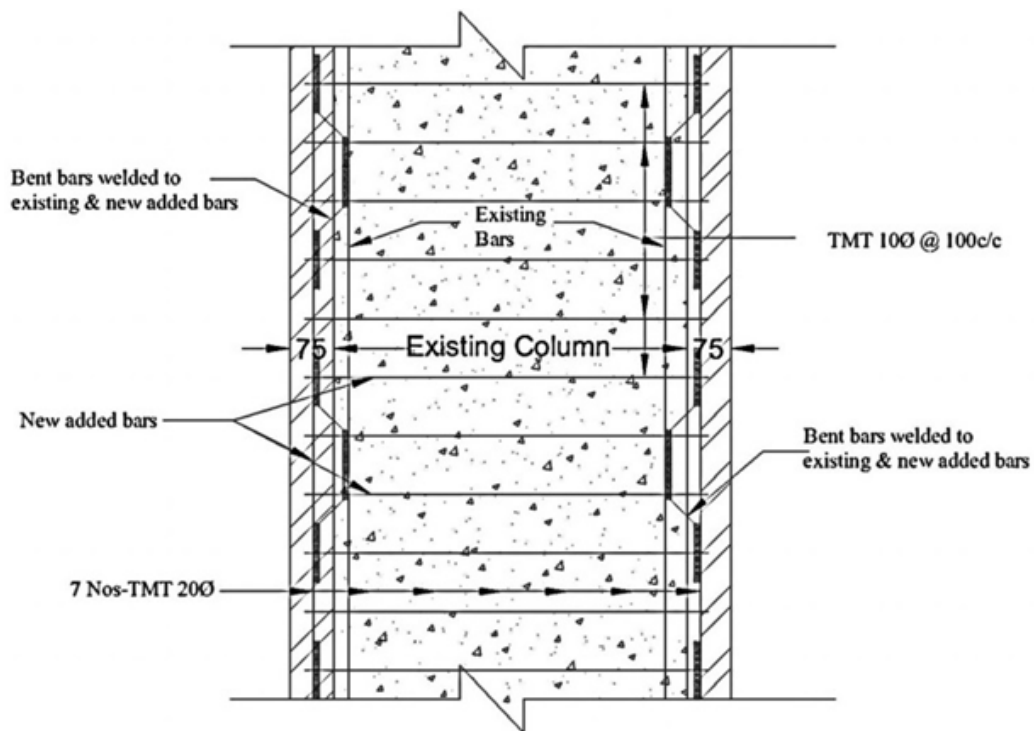
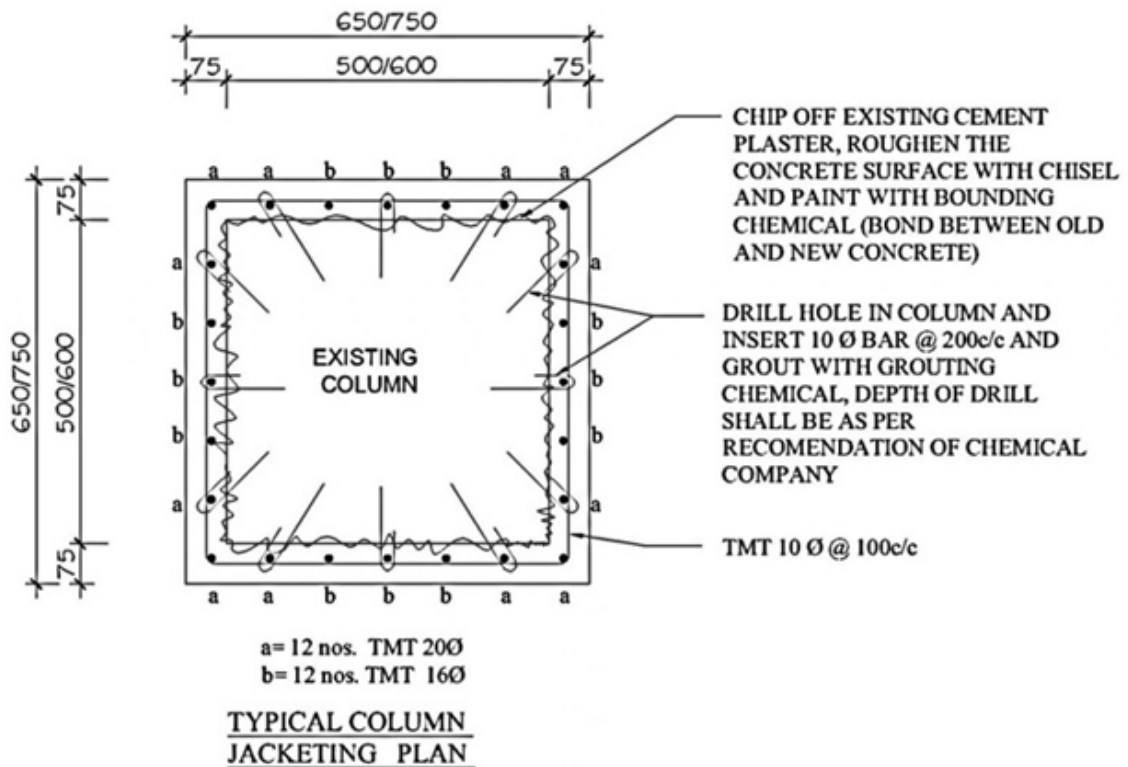


Figure A-3 Column Jacketing section

A.1.7.2. Option2: Steel Jacketing

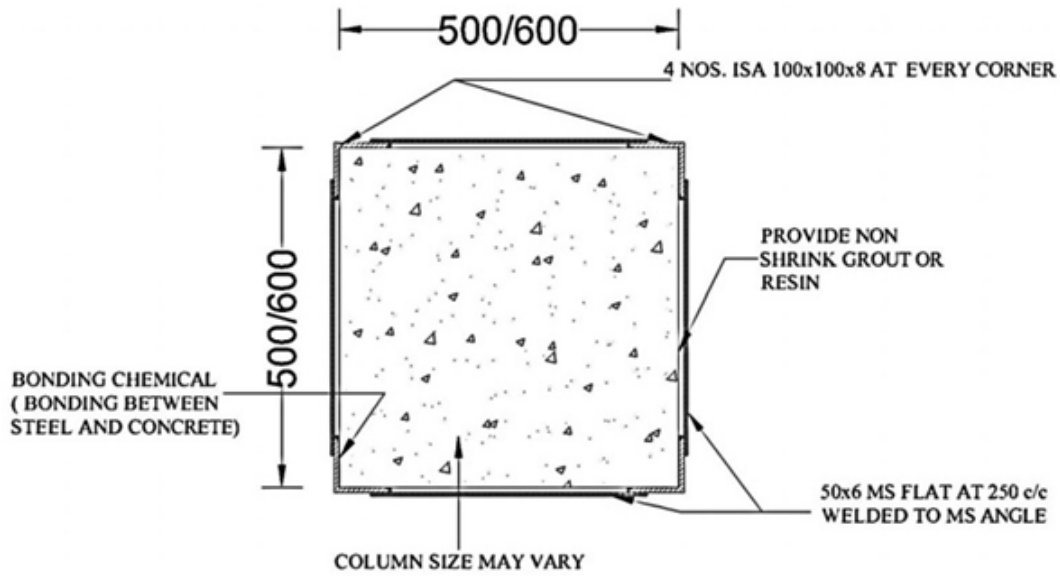


Figure A-4 Typical column steel jacketing detail plan

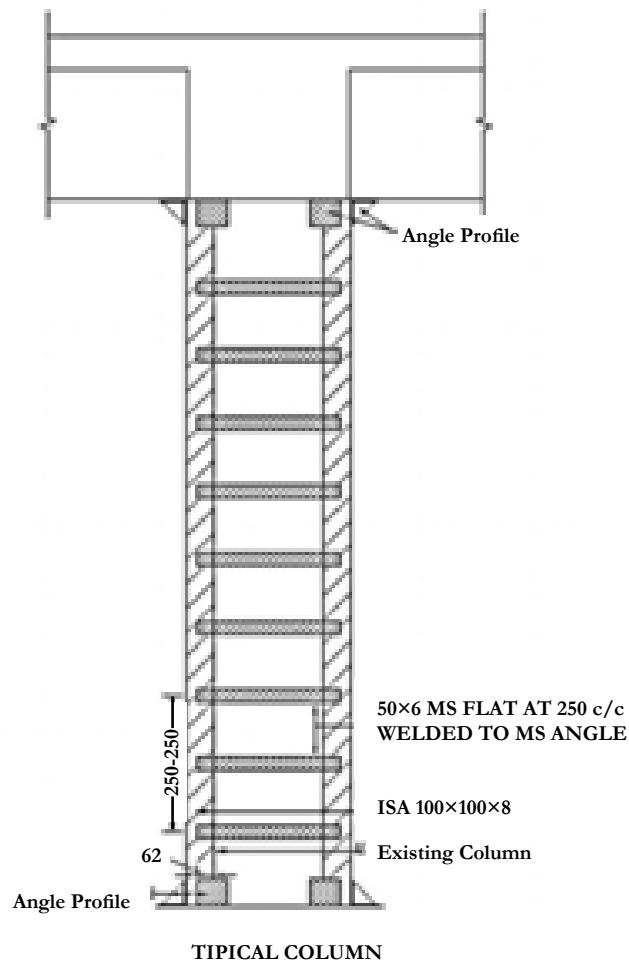


Figure A-5 Steel jacketing detail elevation

A.1.7.3. Option3: Shear Wall Addition with Column Jacketing

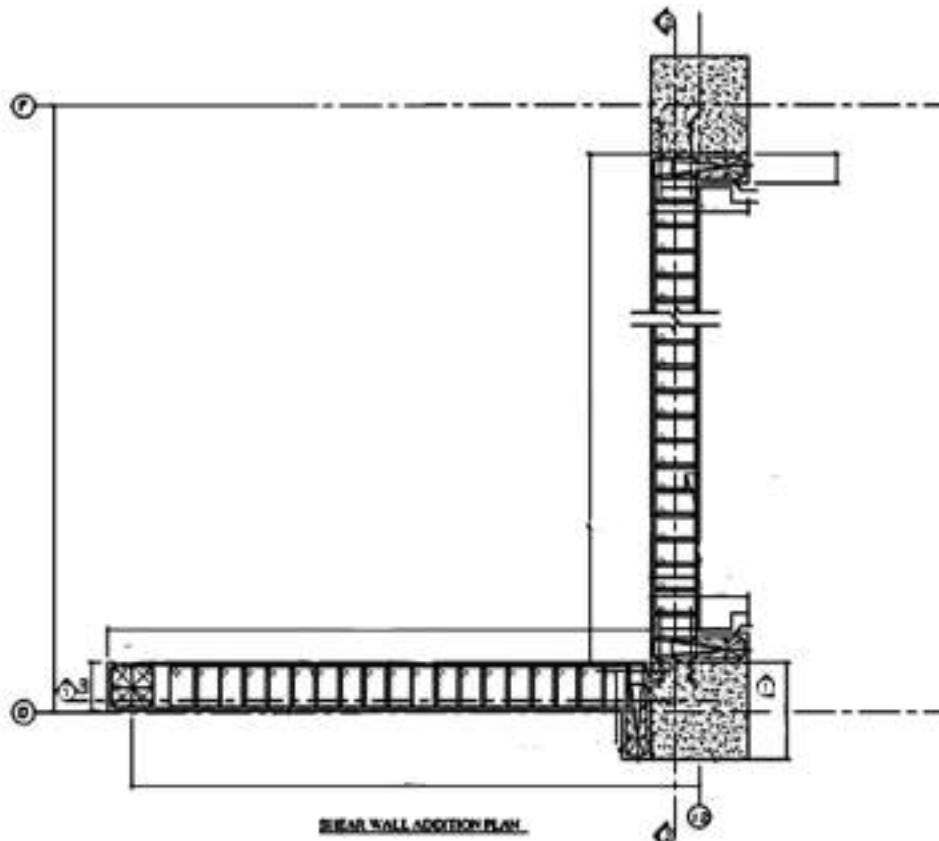
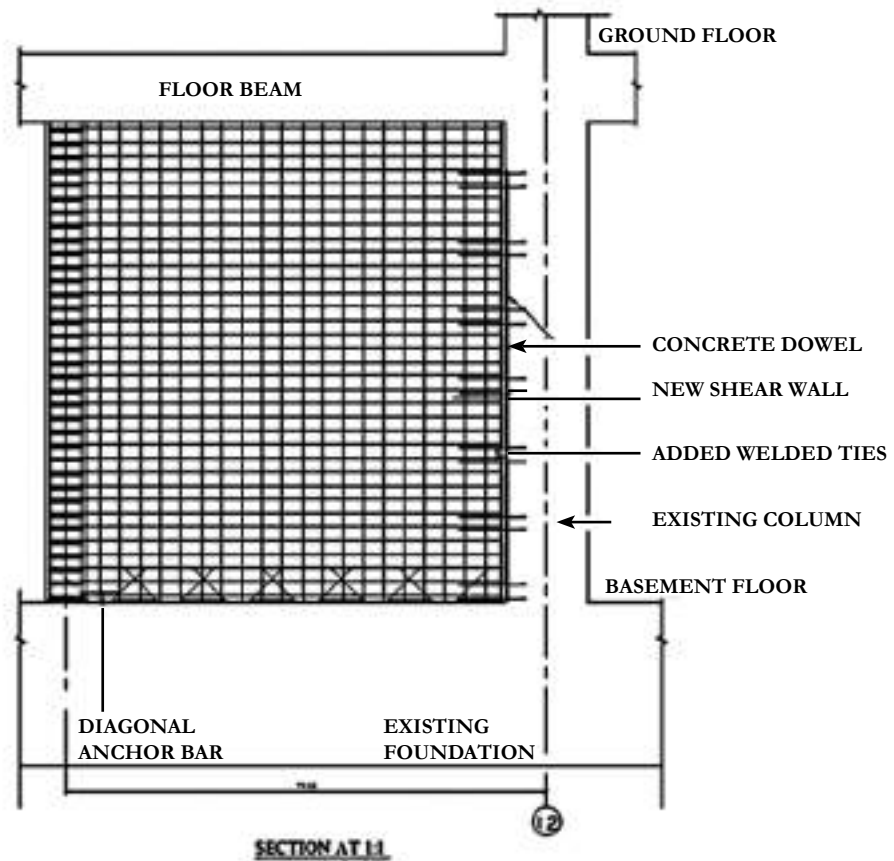


Figure A-6 Steel wall addition plan



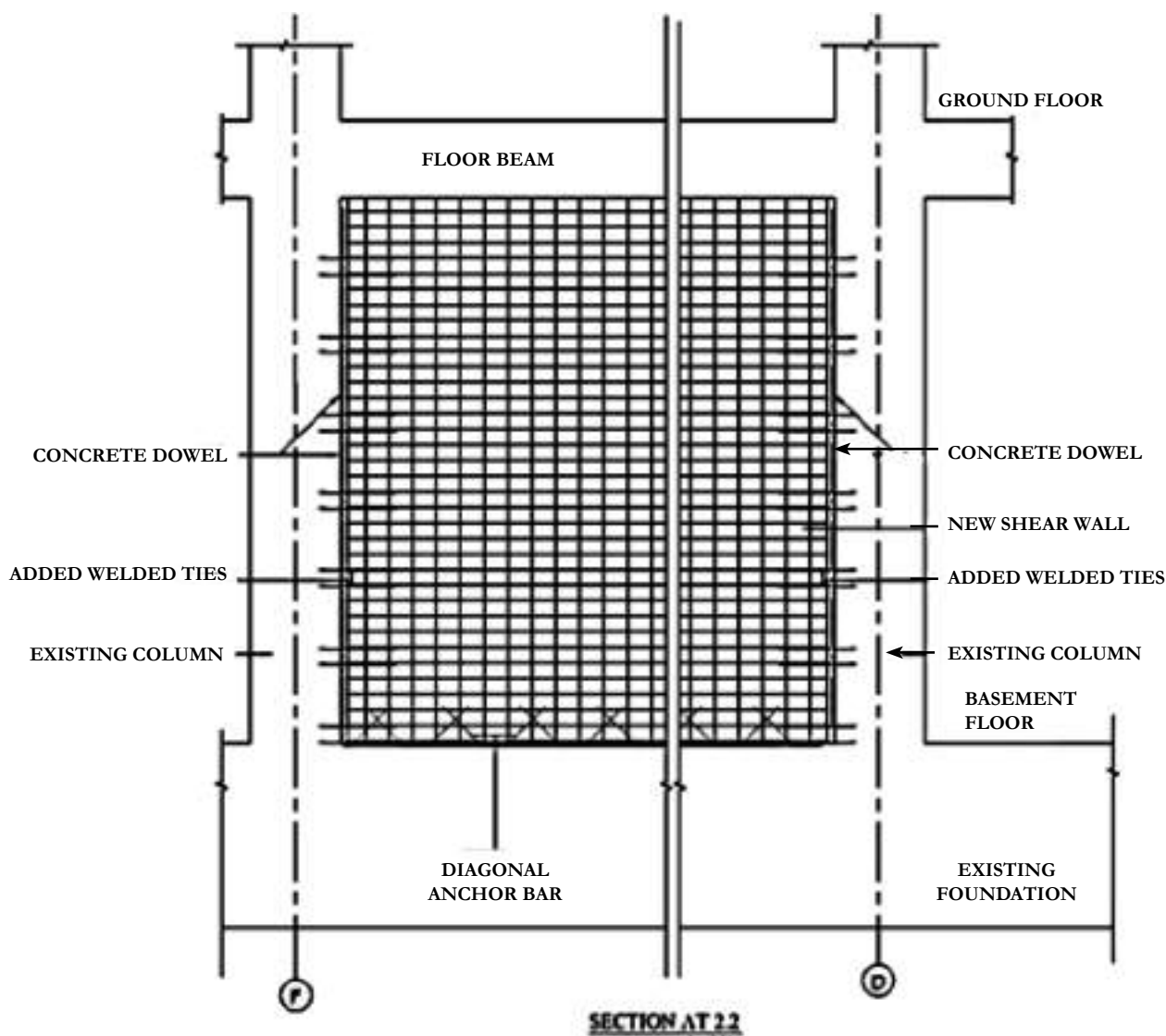


Figure A-7 Sections

A.1.8. COST ESTIMATION OF RETROFITTING OPTIONS

As per the District rate of 2012 AD, cost of the different options of building is mention below:

- Reinforced Concrete Jacketing on columns with approximate cost of NRs. 12,094,773
- Steel Jacketing on columns with approximate cost of NRs. 8,614,768
- Shear wall Addition and Column Jacketing with approximate cost of NRs. 8,176,350

S.N.	Alternatives	Disturbance to existing tenants	Estimated Time for work
1	RC Jacketing on column	High	6 months
2	Steel Jacketing on column	High	5 months
3	Shear wall addition and column jacketing	Medium	3.5 months

COMPARATIVE STUDY OF DIFFERENT OPTIONS

Alternatives	Options	Parameter						
		Disturbance to exiting structure	Time Consumption	Disurbance to existing function during constnction	Cost	Damagability after retrofitting	Effection present aesthetic	Requirement of foundation strengthening
1	RC Jacketing on Column	***	***	***	***	*	**	*
2	Steel Jacketing on Column	***	**	***	**	**	**	*
3	Shear wall addition and column jacketing	**	*	**	**	**	*	**

*** *High*
 ** *Medium*
 * *Low*

A.1.9. RECOMMENDATION

From the point of cost estimation and time of completion for the retrofitting, it is likely to adopt option 3, i.e. shear wall addition with concrete jacketing of columns.

EXAMPLE 2: OCCUPANCY CHANGE

A.2. SEISMIC EVALUATION OF RESIDENTIAL RCC BUILDING WHICH CONVERTED TO HEALTH CLINIC (OCCUPANCY CHANGE)

This building is RCC frame structure situated at Khusibu, Naya Bazar. This building is in good condition and well maintained but built before seismic code was introduced in Nepal.

The size of column is 230mm × 230mm, beam size of 230mm × 350mm, slab thickness of 125mm and storey height of 2.7m. It consists of 3- storey.

The building was built for the purpose of residential use. After the fast urbanization this locality of the building, Khusibu, is more commercial so now this building is to be converted into the health clinic.

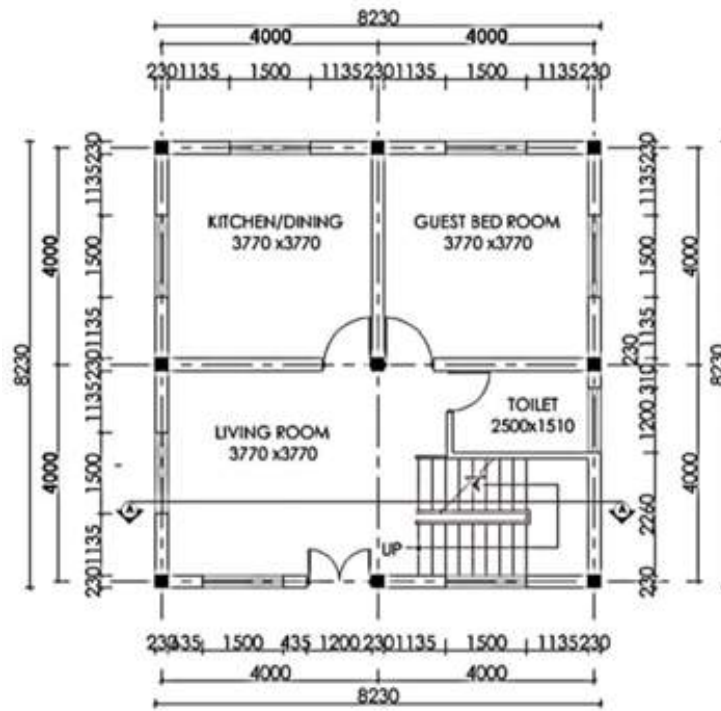
A.2.1. GENERAL DESCRIPTION OF EXISTING BUILDING

Building Description : RCC Frame Structural (In good Condition, but built before Seismic Code introduced in NEPAL)	Site Visit/ Visual Inspection/Site measurements
Location : Khusibhu, Naya Bazar	Site Visit/ Visual Inspection/Site measurements
Storey height : 2.7 m	Site Visit/ Visual Inspection/Site measurements
No. of Stories : 3 nos	Site Visit/ Visual Inspection/Site measurements
Column Size : 230 mm × 230 mm	Site Visit/ Visual Inspection/Site measurements
Beam Size : 230 mm × 350 mm	Site Visit/ Visual Inspection/Site measurements
Slab thickness : 125 mm	Site Visit/ Visual Inspection/Site measurements
Type of foundation : Isolated foundation	Site Visit/ Foundation Exploration

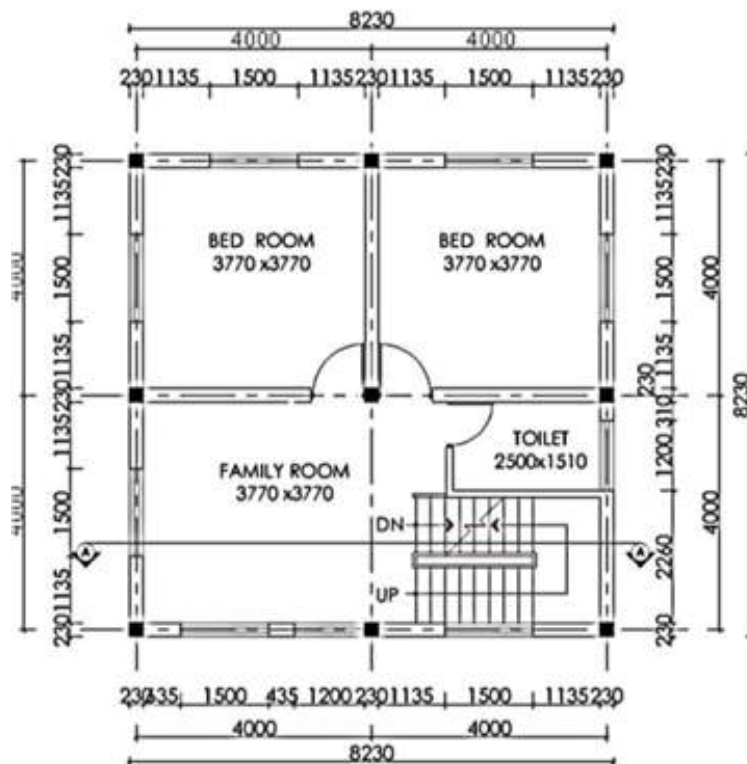
A.2.2. STRUCTURAL ASSESSMENT CHECKLIST

S.N.	CHECKS	REMARKS
1.	Load Path	The frame system provides a complete load path which transfers all inertial forces in the building to the foundation.
2.	Redundancy	There are two bays of frame in each direction.
3.	Geometry	The plan of the building is same in all stories.
4.	Weak Storey / Soft Storey	There is no weak / soft storey.
5.	Vertical Discontinuities	Vertical elements in the lateral force resisting system are continuous to the foundation.
6.	Mass	There is no change in effective mass in adjacent floors except at top floor.
7.	Torsion	The eccentricity of the building is not within the limit.
8.	Adjacent Buildings	There are no adjacent buildings.
9.	Short Column	No short column effect
10.	Deterioration of Concrete	No visible deterioration observed. No cracks were observed.

A.2.3. BUILDING DRAWINGS



GROUND FLOOR PLAN
AREA= 67.73 sq. mt



FIRST FLOOR PLAN
AREA= 67.73 sq. mt

Figure A-8 Building plan

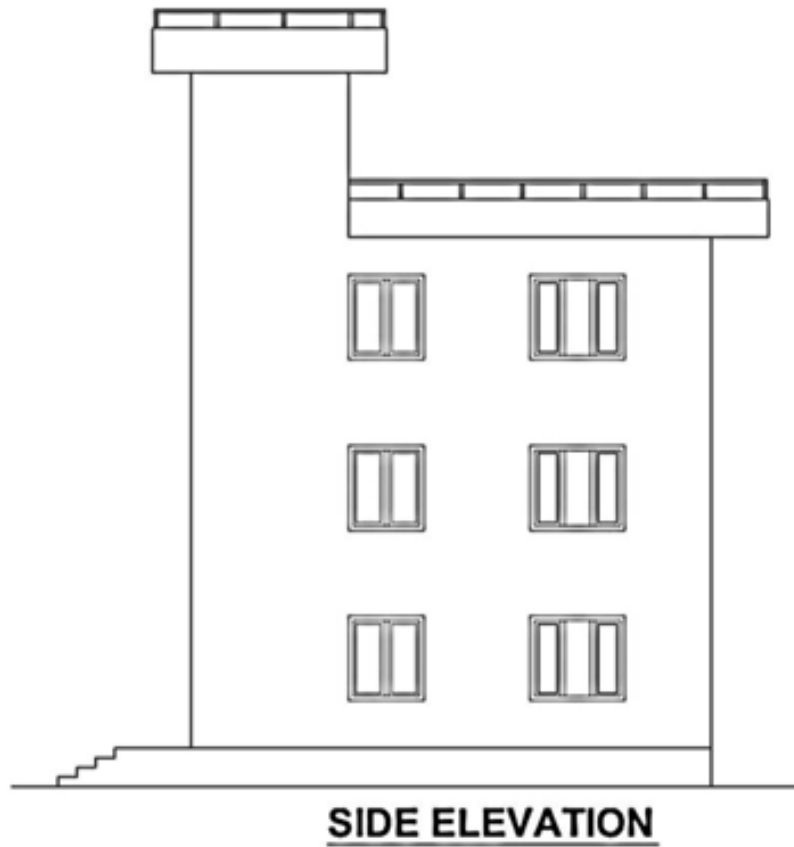
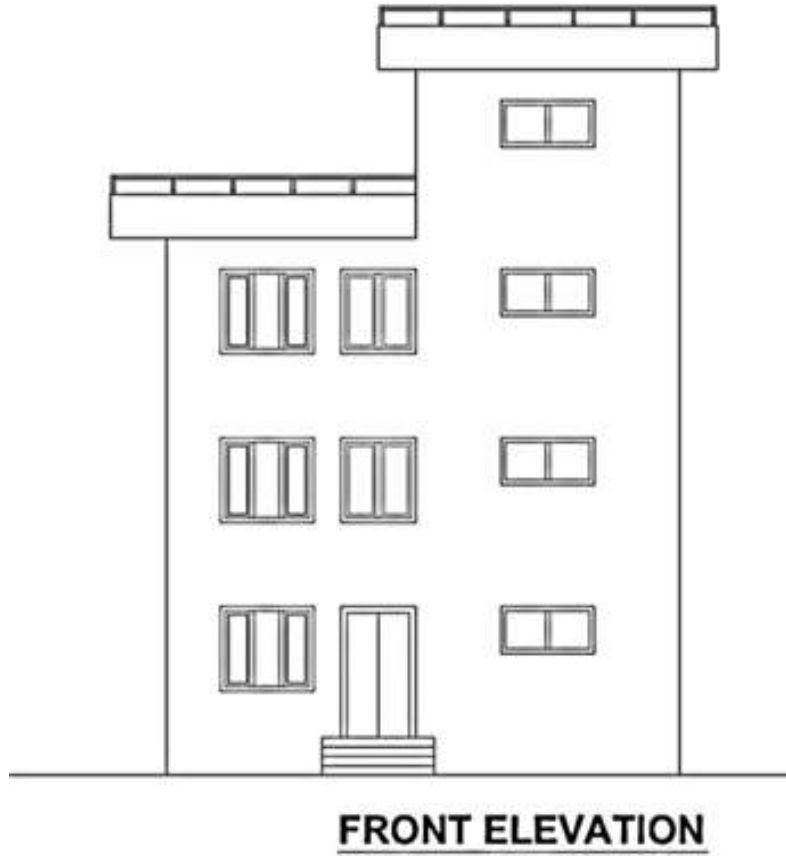
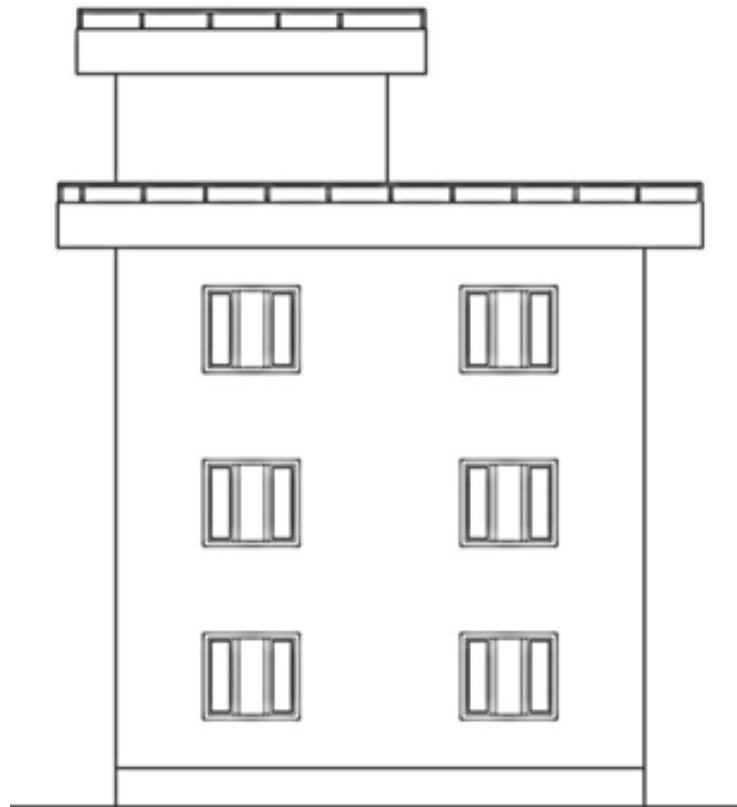
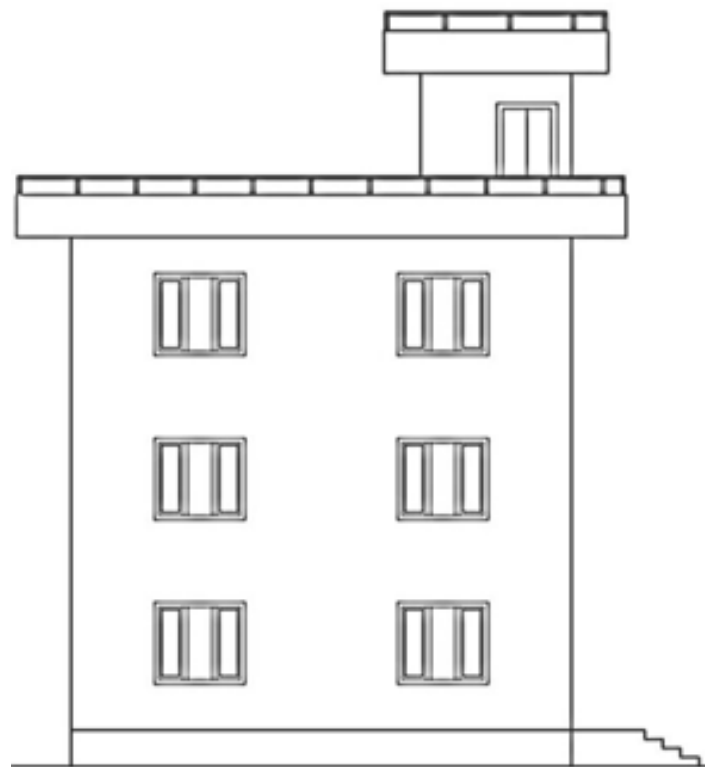


Figure A-9 Front and side elevation



BACK ELEVATION



SIDE ELEVATION

Figure A-10 Back and side elevation

A.2.4. STRUCTURAL DATA

Unit Weight of RCC	= 25 KN/m ³
Unit Weight of Brick Masonry	= 19.6 KN/m ³
Unit Weight of Plaster	= 20 KN/m ³
Unit Weight of Marble	= 26.7 KN/m ³

Live load:

For Floors	= 2.5 KN/m ² (Residential building)
	= 3.0 KN/m ² (Health Clinic)
For Roof	= 1.5 KN/m ²
Grade of Concrete	= M20 Site Visit/ Visual Inspection/Site Measurements
Grade of Steel	= Fe 415 Site Visit/ Visual Inspection/Site Measurements

(Stiffness of the Brick Masonry is not considered in the calculation)

A.2.5. LOAD CALCULATIONS

Dead Load :

1. For Different Floors:

Slab Load	= 0.125 × 25 = 3.125 KN/m ²
Ceiling Plaster Load	= 0.02 × 20 = 0.40 KN/m ²
Floor Finish Load	= 0.025 × 20 = 0.50 KN/m ²
Marble Floor Load	= 0.025 × 26.7 = 0.667 KN/m ²
Total Load	= 4.692 KN/m ²
	~ 4.70 KN/m ²

2. For Roof Floor:

Slab Load	= 0.125 × 25 = 3.125 KN/ m ²
Ceiling Plaster Load	= 0.02 × 20 = 0.40 KN/ m ²
Floor Finish Load	= 0.025 × 20 = 0.50 KN/ m ²
Mosaic Floor Load	= 0.025 × 20 = 0.50 KN/ m ²
Total Load	= 4.525 KN/m ²
	~ 4.50 KN/m ²

A.2.6. Case I: STRUCTURAL ANALYSIS OF BUILDING (As Residential Building)

A.2.6.1. Method I: As per NBC 105:1994

Assumptions:

a) Live Load Calculation:

Unit weight of brick work	= 19.6 KN/m ³
Live load	= 2.5 KN/m ²

LEVEL	FLOORS	FLOOR AREA sq.m	LL	0.25LL	Remarks
4	Third Floor	29.485	44.2275	11.056875	
3	Second Floor	90.33	270.99	67.7475	
2	First Floor	67.73	203.19	50.7975	
1	Ground Floor	67.73	203.19	50.7975	
Σ				180.39938	

Note: Quick check calculation based on FEMA 310 for the seismic evaluation of building under consideration

b) Lump Mass Calculation:

S.NO.	FLOORS	Total Dead Load (KN)	Total Live Load (KN)	Total Weight (KN)	Remarks
4	Third Floor	260.48	11.056875	271.54	
3	Second Floor	756.04	67.7475	823.79	
2	FirstFloor	649.82	50.7975	700.62	
1	Ground Floor	649.82	50.7975	700.62	
Σ				2496.56	

c) Calculation of Base Shear

The design horizontal seismic force coefficient, C_d shall be taken as :

As per NBC 105 $C_d = C Z I K$

Where, C is the basic seismic coefficient for the fundamental translational period in the direction under consideration.

Z = Seismic Zoning Factor

I = Importance Factor

K = Structural Performance Factor

The total design lateral force or Design Seismic Base Shear (V_B) along any principal direction is determined by the following expression :

$$V_B = C_d W_t$$

Where, C_d = The Design Horizontal Seismic Coefficient

W_t = Total of the gravity loads of the whole building

The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

As per NBC 105 $T_a = \frac{0.09h}{\sqrt{d}}$

Where, h = Height of Building in meter = 10.8m

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

dx = 8.23m

dz = 8.23m

$$T_{ax} = \frac{0.09h}{\sqrt{dx}}$$

= 0.338

$$T_{az} = \frac{0.09h}{\sqrt{dz}}$$

= 0.338

Therefore, C = 0.08 for medium soil (Ref: NBC 105:1994)

Seismic zoning factor for Lalitpur is, Z = 1.0

$$\begin{aligned}
 C_d &= C Z I K \\
 &= 0.08 \times 1 \times 1 \times 1 \\
 &= 0.08
 \end{aligned}$$

$$\begin{aligned}
 \text{Base shear} &= V = C_d W_i \\
 &= 0.08 \times 2468.34 \quad \mathbf{197.46KN}
 \end{aligned}$$

d) Distribution of Base Shear And Calculation Of Shear Stress In RC Columns

The horizontal seismic force at each level i shall be taken as :

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

$$\text{As per NBC105} \quad F_i = V \times \frac{W_i h_i}{\sum W_i h_i}$$

Where,

$$\begin{aligned}
 W_i &= \text{Proportion of } W_i \text{ contributed by level } i, \\
 h_i &= \text{Height of floor } i \text{ measured from base}
 \end{aligned}$$

Floor	Total weight W _i (KN)	Height h _i (m)	W _i h _i	$\frac{W_i h_i}{\sum W_i h_i}$	Q _i (KN) = V × $\frac{W_i h_i}{\sum W_i h_i}$	Storey Shear F _i (KN)
4.00	271.54	10.8	2932.63	0.19	37.51	37.51
3.00	812.50	8.1	6581.25	0.43	84.90	122.41
2.00	692.15	5.4	3737.61	0.24	47.39	169.8
1.00	692.15	2.7	1868.80	0.12	23.69	193.49
∑	2468.34		15120.29	1.00	370.25	

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings, 6.5.1)

Average Shearing stress in columns is given as

$$T_{col} = \frac{n_c}{n_c - n_f} \times \frac{V_j}{A_c} < \text{min of } 0.4\text{MPa and } 0.1\sqrt{f_{ck}} \quad (\text{Ref: IS 15988:2013 Clause 6.5.1})$$

$$0.1\sqrt{f_{ck}} = 0.45$$

For Ground Storey columns,

n_c = Total no. of Columns resisting lateral forces in the direction of loading

n_f = Total no. of frames in the direction of loading

A_c = Summation of the cross- section area of all columns in the storey under consideration

V_j = Maximum Storey shear at storey level 'j'

DCR = Demand Capacity Ratio

Storey	n _c	n _{fl}	n _{t2}	A _c	Storey Shears	Shear Stress		DCR		Remarks
						T col-x (MPa)	T col-z (MPa)	in x-dir	in z-dir	
4	4	2	2	0.211	37.51	0.35	0.35	0.87	0.87	Demand Capacity Ratio is not less than 1,
3	9	3	3	0.476	122.41	0.38	0.38	0.95	0.95	
2	9	3	3	0.476	169.8	0.53	0.53	1.32	1.32	
1	9	3	3	0.476	193.49	0.6	0.6	1.5	1.5	

$$T_{col} > \min \text{ of } 0.4 \text{ MPa and } 0.1 \sqrt{f_{ck}} = 0.45 \text{ MPa}$$

Hence the check is not satisfied

e) Axial Stress Check

Axial Stresses Due To Overturning Forces As Per FEMA 310

i) Axial stress in moment frames for x-direction loading

Axial force in columns of moment frames at base due to overturning forces, The axial stress of columns subjected to overturning forces F_o is given by

$$F_o = \frac{2 V_b}{3 n_f} \times \frac{H}{L} \quad \text{Fema 310 Clause 3.5.3.6}$$

V_b = Base shear x Load Factor

$$\begin{aligned} 370.25 \times 1.5 &= 555.375 \quad \text{KN} \\ A_c = \text{column area} &= 0.0529 \quad \text{Sq.m.} \\ H = \text{total height} &= 10.8 \quad \text{m} \\ L = \text{Length of the building} &= 8.00 \quad \text{m} \end{aligned}$$

$$\begin{aligned} &= \frac{2 V_b}{3 n_f} \times \frac{H}{L} \\ &= 166.61 \quad \text{KN} \end{aligned}$$

Axial Stress for x-direction loading,

$$\sigma = \frac{166.61 \times 1000}{0.05} = 3.33 \quad \text{MPa}$$

$$\begin{aligned} \sigma_{all} &= 0.25 f_{ck} \\ &= 5.00 \quad \text{MPa} \quad (\text{IS 15988:2013 Clause 6.5.4}) \end{aligned}$$

therefore $\sigma < \sigma_{all}$ OK

ii) Axial stress in moment frames for z-direction loading

Axial force in columns of moment frames at base due to overturning forces,
The axial stress of columns subjected to overturning forces F_o is given by

$$F_o = \frac{2 V_b}{3 n_f} \times \frac{H}{L}$$

V_b = Base shear x Load Factor

$$370.25 \times 1.5 = 555.375 \quad \text{KN}$$

A_c = Column area = 0.052 sq.m.
 H = Total height = 10.8 m
 L = Length of the building = 8.00 m

$$F_o = \frac{2 V_b}{3 n_f} \times \frac{H}{L} \quad \text{Fema 310 Clause 3.5.3.6}$$

$$= 166.61 \quad \text{KN}$$

Axial Stress for z-direction loading,

$$\sigma = \frac{166.61 \times 1000}{0.052} = 2.11 \quad \text{MPa}$$

(IS 15988:2013 Clause 6.5.4) $\sigma_{all} = 0.25 f_{ck} = 5.00 \text{ MPa}$
 therefore $\sigma < \sigma_{all}$ OK
Hence the check is satisfied

f) Checking shear capacity of beam

The shear reinforcement provided in the existing beam at support section is 3 TOR 16 Top and bottom

Where,

$b = 230$
 $D = 350$
 $d = 350 - 25 - 8 = 317$

Area of steel (A_{st}) = 4 tor 12 diameter = 452mm²

$f_{ck} = 20 \text{ N/mm}^2$

$f_y = 415 \text{ N/mm}^2$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times 452}{230 \times 317} = 0.619 \%$$

Using table 19 of IS 456: 2000, for M20 grade of concrete and $\frac{100 A_{st}}{bd} = 0.813$
 $\tau_c = 0.58 \text{ MPa}$

Stirrups are 2-legged 8 mm dia. @ 150mm C/C, hence from clause 40.4 of IS 456:2000

$$V_{us} = \frac{0.87 \times f_y \times A_{st} \times d}{S_v}$$

$$V_u = V_{us} + \tau_c bd$$

$$= \frac{0.87 \times 415 \times 2 \times 50.265 \times 317}{150} + 0.58 \times 230 \times 317$$

$$= 118.99 \text{ KN}$$

A.2.6.2. Method II: As per IS 1893:2002-Part 1

Assumptions:

Unit weight of brick work = 19.6 KN/m³

Live load = 2.5 KN/m²

a) LIVE LOAD CALCULATION

LEVEL	FLOORS	FLOOR AREA (Sq.m)	LL (KN/m ²)	0.25LL	Remarks
4	Third Floor	29.485	44.2275	11.057	
3	Second Floor	90.33	225.825	56.456	
2	First Floor	67.73	169.325	42.331	
1	Ground Floor	67.73	169.325	42.331	
Σ				152.18	

Note: Quick check calculation based on FEMA 310 for the seismic evaluation of building under consideration.

b) LUMP MASS CALCULATION

S.NO.	FLOORS	Total Dead Load (KN)	Total Live Load (KN)	Total Weight (KN)	Remarks
4	Third Floor	260.48	11.056875	271.54	
3	Second Floor	756.04	56.45625	812.50	
2	First Floor	649.82	42.33125	692.15	
1	Ground Floor	649.82	42.33125	692.15	
Σ				2468.34	

c) CALCULATION OF BASE SHEAR

Calculation of earthquake loads using Seismic coefficient method:

The design horizontal seismic coefficient,

$$A_h = \frac{ZIS_a}{2Rg}$$

Reference IS:1893:2002

Where Z = Zone Factor

I = Importance Factor

R = Response Reduction Factor

$\frac{s_a}{g}$ = Average Response Acceleration Coefficient

The total design lateral force or Design Seismic Base Shear (V_b) along any principal direction is determined by the following expression :

$$V_b = A_h W$$

Where,

A_h = The Design Horizontal Seismic Coefficient

W = Seismic weight of the building

The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$$T_a = \frac{0.09h}{\sqrt{d}} \quad \text{Ref : IS1893:2002}$$

Where,

h = Height of Building in meter = 10.80 m

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

$$dx = 8.23 \text{ m}$$

$$dz = 8.23 \text{ m}$$

$$T_{ax} = \frac{0.09h}{dx^{0.5}}$$

$$= 0.338 < 0.55$$

$$T_{az} = \frac{0.09h}{dz^{0.5}}$$

$$= 0.338 < 0.55$$

Therefore ,

$$\frac{s_a}{g} = 2.5 \text{ for medium soil (IS :1893(Part 1) : 2002)}$$

$$Z = 0.36 \text{ (For Seismic Zone V) (Refer IS 1893 (Part 1) :2002-table 2)}$$

$$I = 1.0 \text{ (For Residential Building) (Refer IS 1893 (Part 1) :2002-table 6)}$$

$$\frac{s_a}{g} = 2.5 \text{ (For Medium Soil) (Refer IS 1893 (Part 1):2002-Clause 6.4.5 and Fig.2)}$$

$$R = 3.0 \text{ (For Ordinary RC Moment Resisting Frame) (Refer IS 1893 (Part 1) :2002-table 7)}$$

The total design lateral force or design seismic base shear is given by,

$$A_h = \frac{ZIS_a}{2Rg}$$

$$= 0.36 * 1.0 * 2.5 / 2 * 3 = 0.15$$

$$\text{Base shear} = V_b = A_h W$$

$$= 0.15 \times 2468.34$$

$$= 370.251 \text{ KN}$$

d) **Distribution of Base Shear and Calculation of Shear Stress in RC Columns :**

Floor	Total weight W_i (KN)	Height h_i (m)	$W_i h_i^2$	$\frac{W_i h_i^2}{\sum W_i h_i^2}$	Q_i (KN)	Storey Shear V_i (KN)
4.00	271.54	10.8	31672.06	0.29	106.40	106.40
3.00	812.50	8.1	53307.88	0.48	179.09	285.49
2.00	692.15	5.4	20183.13	0.18	67.81	353.30
1.00	692.15	2.7	5045.78	0.05	16.95	370.25
\sum	2468.34		110208.85	1.00	370.25	

e) **SHEAR STRESS AT STOREY LEVEL :**

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of buildings 6.5.1)

The Total design lateral force or design seismic base shear is given by,

$$V_b = A_h W$$

Average Shearing stress in columns is given as (Ref: IS 15988:2013 Clause 6.5.1)

$$T_{col} = \left(\frac{n_c}{n_c - n_f} \right) \times \left(\frac{v_j}{A_c} \right) < \text{min. of } 0.4 \text{ Mpa and } 0.1 \sqrt{f_{ck}}$$

For Ground Storey columns,

n_c = Total No. of Columns resisting lateral forces in the direction of loading

n_f = Total No. of frames in the direction of loading

A_c = Summation of the cross- section area of all columns and shear wall in the storey under consideration

V_j = Maximum Storey Shear at storey level 'j'

f) **Shear Stress at Storey Levels**

Storey	n_c	n_{f1}	n_{f2}	A_c	Storey Shears (KN)	Shear Stress	
						$T_{col} 1$ (MPa)	$T_{col} 2$ (MPa)
4	4	2	2	0.211	106.40	1.01	1.01
3	9	3	3	0.476	285.49	0.90	0.90
2	9	3	3	0.476	353.30	1.11	1.11
1	9	3	3	0.476	370.25	1.17	1.17

$$T_{col} \gg \text{min of } 0.4 \text{MPa and } 0.1 \sqrt{f_{ck}} = 0.45 \text{MPa}$$

Hence, the check is not satisfied.

A.2.7. Case II: STRUCTURAL ANALYSIS OF THE BUILDING (After occupancy change to a Health Clinic)

A.2.7.1. Method I: As per NBC 105:1994

Major changes while converting Residential building into Health clinic

S.No	Description of Building	Live load(kN/m ²)	Importance Factor
1.	Residential	2.5	1
2.	Health Clinic	3	1.5

Seismic evaluation of building under consideration.

Assumptions:

Unit weight of brick work = 19.6 KN/m³

Live load = 3.0 KN/m²

a) LIVE LOAD CALCULATION

LEVEL	FLOORS	FLOOR AREA sq.m	LL	0.25LL	Remarks
4	Third Floor	29.485	44.2275	11.056875	
3	Second Floor	90.33	270.99	67.7475	
2	First Floor	67.73	203.19	50.7975	
1	Ground Floor	67.73	203.19	50.7975	
Σ				180.39938	

Note: Quick check calculation based on FEMA 310 for the seismic evaluation of building under consideration

b) LUMP MASS CALCULATION

S.NO.	FLOORS	Total Dead Load (KN)	Total Live Load (KN)	Total Weight (KN)	Remarks
4	Third Floor	260.48	11.056875	271.54	
3	Second Floorz	756.04	67.7475	823.79	
2	FirstFloor	649.82	50.7975	700.62	
1	Ground Floor	649.82	50.7975	700.62	
Σ				2496.56	

Calculation Of Base Shear (Using NBC 105:1994)

c) Design Horizontal Seismic Coefficient for the Seismic Coefficient Method

The design horizontal seismic force coefficient, C_d shall be taken as :

$$C_d = C Z I K$$

Where, C is the basic seismic coefficient for the fundamental translational period in the direction under consideration.

$$Z = \text{Seismic Zoning Factor} \quad I = \text{Importance Factor}$$
$$K = \text{Structural Performance Factor}$$

The total design lateral force or Design Seismic Base Shear (V_B) along any principal direction is determined by the following expression :

$$V_B = C_d W_t$$

Where, C_d = The Design Horizontal Seismic Coefficient

W_t = Total of the gravity loads of the whole building

The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$$\text{NBC:105:1994} \quad T_a = (0.09h)/\sqrt{d}$$

Where,

h = Height of Building in meter = 10.8m

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

d_x = 8.23m

d_z = 8.23m

T_{ax} = $(0.09h)/\sqrt{d_x}$ = 0.338

T_{az} = $(0.09h)/\sqrt{d_z}$ = 0.338

Therefore, $C = 0.08$ for medium soil

Seismic zoning factor for Lalitpur is, $Z = 1.0$

$$C_d = C Z I K$$

$$= 0.08 \times 1 \times 1.5 \times 1 = 0.12$$

$$\text{Base shear} = V = C_d W_t$$
$$= 0.12 \times 2468.34 = 296.20 \text{ KN}$$

d) Distribution Of Base Shear And Calculation Of Shear Stress In RC Columns

The horizontal seismic force at each level i shall be taken as :

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

$$F_i = V \times \frac{W_i h_i}{\sum W_i h_i}$$

Where,

W_i = Proportion of W_t contributed by level i ,

h_i = Height of floor i measured from base

Floor	Total weight W_i (KN)	Height h_i (m)	$W_i h_i$	$\frac{W_i h_i}{\sum W_i h_i}$	Q_i (KN) $= V \times \frac{W_i h_i}{\sum W_i h_i}$	Storey Shear F_i (KN)
4.00	271.54	10.8	2932.63	0.19	56.27	56.27
3.00	823.79	8.1	6672.69	0.43	127.36	183.63
2.00	700.62	5.4	3783.34	0.24	71.08	254.71
1.00	700.62	2.7	1891.67	0.12	35.54	290.25
Σ	2468.34		15280.33	1.00	290.25	

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings, 6.5.1)

Average Shearing stress in columns is given as

$$T_{col} = \frac{n_c}{n_c - n_f} \times \frac{V_j}{A_c} < \min \text{ of } 0.4 \text{ MPa and } 0.1 \sqrt{f_{ck}} \quad (\text{Ref: IS 15988:2013 Clause 6.5.1})$$

$$0.1 \sqrt{f_{ck}} = 0.45$$

For Ground Storey columns,

n_c = Total no. of Columns resisting lateral forces in the direction of loading

n_f = Total no. of frames in the direction of loading

A_c = Summation of the cross- section area of all columns in the storey under consideration

V_j = Maximum Storey shear at storey level 'j'

DCR = Demand Capacity Ratio

Floor	Total weight W_i (KN)	Height h_i (m)	$W_i h_i$	$\frac{W_i h_i}{\sum W_i h_i}$	Q_i (KN) $= V \times \frac{W_i h_i}{\sum W_i h_i}$	Storey Shear F_i (KN)
4.00	271.54	10.8	2932.63	0.19	56.27	56.27
3.00	823.79	8.1	6672.69	0.43	127.36	183.63
2.00	700.62	5.4	3783.34	0.24	71.08	254.71
1.00	700.62	2.7	1891.67	0.12	35.54	290.25
Σ	2468.34		15280.33	1.00	290.25	

$$T_{col} > \min \text{ of } 0.4 \text{ MPa and } 0.1 \sqrt{f_{ck}} = 0.45 \text{ MPa}$$

Hence the check is not satisfied

A.2.7.2. Method II: As per IS 1893:2002-Part 1

Assumptions:

Unit weight of brick work = 19.6 KN/m³

Live load = 3.0 KN/m²

a) **LIVE LOAD CALCULATION**

LEVEL	FLOORS	FLOOR AREA (Sq.m)	LL	0.25LL	Remarks
4	Third Floor	29.485	44.2275	11.056875	
3	Second Floor	90.33	270.99	67.7475	
2	FirstFloor	67.73	203.19	50.7975	
1	Ground Floor	67.73	203.19	50.7975	
Σ				180.39938	

Note: Quick check calculation based on FEMA 310 for the seismic evaluation of building under consideration

b) **LUMP MASS CALCULATION**

S.NO.	FLOORS	Total Dead Load (KN)	Total Live Load (KN)	Total Weight (KN)	Remarks
4	Third Floor	260.48	11.056875	271.54	
3	Second Floor	756.04	67.7475	823.79	
2	FirstFloor	649.82	50.7975	700.62	
1	Ground Floor	649.82	50.7975	700.62	
Σ				2496.56	

c) **CALCULATION OF BASE SHEAR**

The total design lateral force or design seismic base shear is given by
Based on IS 1893 (Part 1): 2002, Criteria for earthquake resistant design of structures,
Calculation of earthquake loads using Seismic coefficient method:

The design horizontal seismic coefficient, $A_h = \frac{ZIS_a}{2R_g}$

Where Z = Zone Factor

I = Importance Factor

R = Response Reduction Factor

S_a/g = Average Response Acceleration Coefficient

The total design lateral force or Design Seismic Base Shear (V_B) along any principal direction is determined by the following expression :

$$V_B = A_h W$$

Where,

A_h = The Design Horizontal Seismic Coefficient

W = Seismic weight of the building

The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$$T_a = \frac{0.09h}{d^{0.5}} \quad \text{Ref: IS:1893:2002(part1)}$$

Where,

h = Height of Building in meter = 10.80 m
d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

$$dx = 8.23 \text{ m}$$

$$dz = 8.23 \text{ m}$$

$$T_{ax} = \frac{0.09h}{dx^{0.5}}$$

$$= 0.338 < 0.55$$

$$T_{az} = \frac{0.09h}{dz^{0.5}}$$

$$= 0.338 < 0.55$$

Therefore,

Sa/g = 2.5 for medium soil (IS :1893(Part 1) : 2002

Z = 0.36 (For Seismic Zone V) (Refer IS 1893 (Part 1) :2002-table 2)

I = 1.50 (For Clinic Building) (Refer IS 1893 (Part 1) :2002-table 6)

Sa/g = 2.5 (For Medium Soil) (Refer IS 1893 (Part 1) :2002-Clause 6.4.5 and Fig.2)

R = 3.0 (For Ordinary RC Moment Resisting Frame)(Ref. IS 1893 (Part 1) :2002-table 7)

$$A_h = \frac{ZIS_a}{2Rg}$$

$$= 0.36 \times 1.5 \times 2.5 / 2 \times 3$$

$$= 0.225$$

$$\text{Base shear} = V_b = A_h W$$

$$= 0.225 \times 2496.56$$

$$= 561.726 \text{ KN}$$

d) Distribution of Base Shear and Calculation of Shear Stress in RC Columns :

Floor	Total weight W_i (KN)	Height h_i (m)	$W_i h_i^2$	$\frac{W_i h_i^2}{\sum W_i h_i^2}$	Q_i (KN)	Storey Shear V_i (KN)
4.00	271.54	10.8	31672.06	0.28	159.91	159.91
3.00	823.79	8.1	54048.70	0.49	272.88	432.79
2.00	700.62	5.4	20430.01	0.18	103.15	535.94
1.00	700.62	2.7	5107.50	0.05	25.79	561.73
Σ	2496.56		111258.27	1.00	561.73	

e) SHEAR STRESS AT STOREY LEVEL :

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of buildings 6.5.1)

The Total design lateral force or design seismic base shear is given by

$$V_b = A_h W$$

Average Shearing stress in columns is given as

(Ref: IS 15988:2013 Clause 6.5.1)

$$T_{col} = \left(\frac{n_c}{n_c - n_f} \right) \times \left(\frac{v_j}{A_c} \right) < \text{min. of } 0.4 \text{ Mpa and } 0.1 \sqrt{f_{ck}}$$

For Ground Storey columns,

n_c = Total No. of Columns resisting lateral forces in the direction of loading

n_f = Total No. of frames in the direction of loading

A_c = Summation of the cross- section area of all columns and shear wall in the storey under consideration

V_j = Maximum Storey Shear at storey level 'j'

f) Shear Stress at Storey Levels

Storey	n_c	n_{f1}	n_{f2}	A_c	Storey Shears (KN)	Shear Stress	
						T col 1(MPa)	T col 2(MPa)
4	4	2	2	0.211	159.91	1.52	1.52
3	9	3	3	0.476	432.79	1.36	1.36
2	9	3	3	0.476	535.94	1.69	1.69
1	9	3	3	0.476	561.73	1.77	1.77

$$T_{col} >> \text{min of } 0.4 \text{ MPa} \quad \text{and} \quad 0.1 \sqrt{f_{ck}} = 0.45 \text{ MPa}$$

Hence, the check is not satisfied.

Since columns are not safe, now checking for different categories as below:

g) Calculation of Shear Capacity of Colum Using Capacity design Method :

- Checking Shear Capacity of Center Column :
-

$$\text{Shear Capacity of column required} = 1.4 \frac{M_1 + m_1'}{h_{st}} \quad (\text{Ref:IS 13920:1993 Clause 7.3.4})$$

The longitudinal Beam size is equal to 230 × 350.

Reinforcement of Beam is equal to 3 TOR 16 top and bottom.

Where,

$$b = 230$$

$$D = 350$$

$$d = 350 - 25 - \frac{16}{2} = 317$$

The Moment Capacities are evaluated from STAADPro 2006, which is equal to 68.6 KN-m and 53.6 KN-m.

Shear force in Column corresponding to these moments :

(Ref:IS 13920:1993 Clause 7.3.4)

$$V_u = 1.4 \frac{M_1 + m'_1}{h_{st}}$$

$$= 1.4 \times \frac{68.6 - 53.6}{2.7}$$

$$= 63.36 \quad \text{KN}$$

Size of Column = 230 mm × 230 mm

Area of Steel (A_{st}) = 4 tor- 12 diameter

F_{ck} = 20 N/mm²

F_y = 415 N/mm²

From SP 16 Table 61

for P_t = 0.85% , τ = 0.585 N/mm²

Shear Capacity = 0.585 × 230 × 230 / 1000

$$= 30.94 \text{ kN}$$

Shear to be carried Stirrups V_{us} = 63.36 – 30.94

$$= 32.42 \text{ KN}$$

From SP 16 Table 62 :

Stirrups in the Column : Tor 8 Diameter @150 mm c/c

$$\frac{V_{us}}{d} = 2.42 \text{ KN/cm}$$

$$V_{us} = 2.42 \times 19.2 \text{ kN/cm}$$

$$= 46.5 \text{ KN} >> 32.42 \text{ KN}$$

Hence, the Check for shear tie is satisfied for central column.

h) Axial Stress Check:

1. The Axial Stress due to Gravity Loads as per FEMA 310

Permissible axial stress = $0.1f_c = 2 \text{ N/mm}^2$

The axial stress due to gravity loads in the center column of Ground Floor =

The axial stress due to gravity loads in column

$$= \text{Load on column(N)} / \text{Cross section Area of Column} = \frac{711.289 \times 1000}{230 \times 230}$$

$$= 13.446 \text{ N/mm}^2 > 2 \text{ N/mm}^2$$

Hence the check not satisfied.

2. Axial stresses due to overturning forces as per FEMA 310

2.1 Axial stress in moment frames for x-direction loading

Axial force in columns of moment frames at base due to overturning forces,

The axial stress of columns subjected to overturning forces F_o is given by

From Fema 310 clause 3.5.3.6

$$F_o = \frac{2}{3} \left(\frac{V_b}{n_f} \right) \times \left(\frac{H}{L} \right)$$

$$V_b = \text{Base shear} \times \text{Load Factor} \\ = 561.726 \times 1.5 = 842.59 \text{ KN}$$

$$A_c = \text{column area} = 0.0529 \text{ sq.m.}$$

$$H = \text{total height} = 10.8 \text{ M}$$

$$L = \text{Length of the building} = 8.00 \text{ M}$$

From Fema 310 clause 3.5.3.6

$$F_o = \frac{2}{3} \left(\frac{V_b}{n_f} \right) \times \left(\frac{H}{L} \right)$$

$$= 252.78 \text{ KN}$$

Axial Stress for x-direction loading,

$$\sigma = 252.78 / 0.05 = 4.78 \text{ MPa}$$

$$\sigma_{\text{all}} = 0.25 f_{ck} \text{ (Ref:IS 15988:2013 clause 6.5.4)} = 5.00 \text{ MPa}$$

Therefore $\sigma < \sigma_{\text{all}}$ OK

$$\text{DCR} = 0.334$$

Hence the check is satisfied.

2.2 Axial stress in moment frames for z-direction loading

Axial force in columns of moment frames at base due to overturning forces, The axial stress of columns subjected to overturning forces F_o is given by

From Fema 310 clause 3.5.3.6

$$F_o = \frac{2}{3} \left(\frac{V_b}{n_f} \right) \times \left(\frac{H}{L} \right)$$

$$V_b = \text{Base shear} \times \text{Load Factor} \\ = 561.726 \times 1.5 = 842.59 \text{ KN}$$

$$A_c = \text{column area} = 0.0529 \text{ sq.m.}$$

$$H = \text{total height} = 10.8 \text{ M}$$

$$L = \text{Length of the building} = 8.00 \text{ M}$$

From Fema 310 clause 3.5.3.6

$$F_o = \frac{2}{3} \left(\frac{V_b}{n_f} \right) \times \left(\frac{H}{L} \right)$$

$$= 252.78 \text{ KN}$$

Axial Stress for x-direction loading,

$$\sigma = 252.78 / 0.05 = 4.78 \text{ MPa}$$

$$\sigma_{\text{all}} = 0.25 f_{ck} \text{ (Ref:IS 15988:2013 clause 6.5.4)} = 5.00 \text{ MPa}$$

therefore $\sigma < \sigma_{\text{all}}$ OK

$$\text{DCR} = 0.334$$

Hence the check is satisfied.

i) Check for Out-of-Plane Stability of Brick Masonry Walls

Wall Type	Wall Thickness	Recommended Height/ Thickness ratio ($0.24 < S_x \leq 0.35$) (Ref: Fema 310 Table 4-2)	Actual Height/ Thickness ratio in building	Comments
Wall in ground storey	230 mm	1\8	$\frac{2700 - 350}{230} = 10.217$	Pass
Wall in upper stories	230 mm	16	$\frac{2700 - 350}{230} = 10.217$	Pass

Hence the check is satisfied.

A.2.8. RETROFITTING DRAWINGS

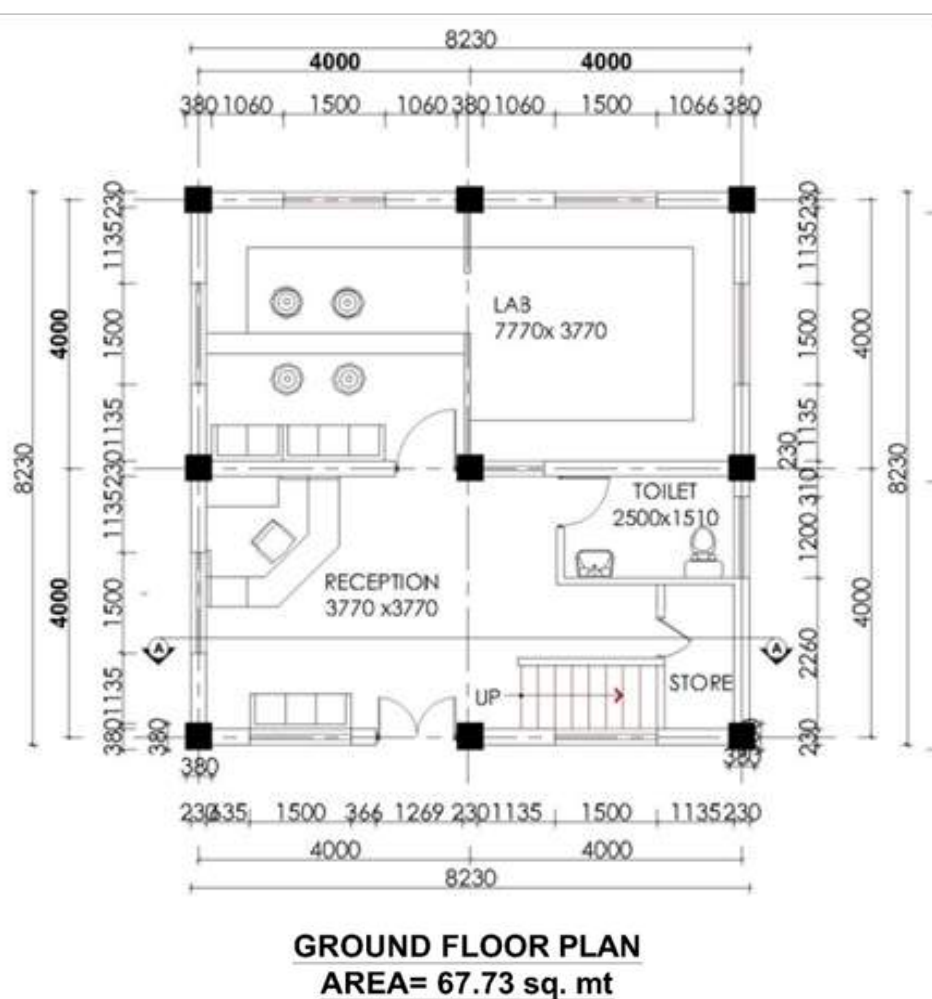


Figure A-11 Retrofitted ground floor plan

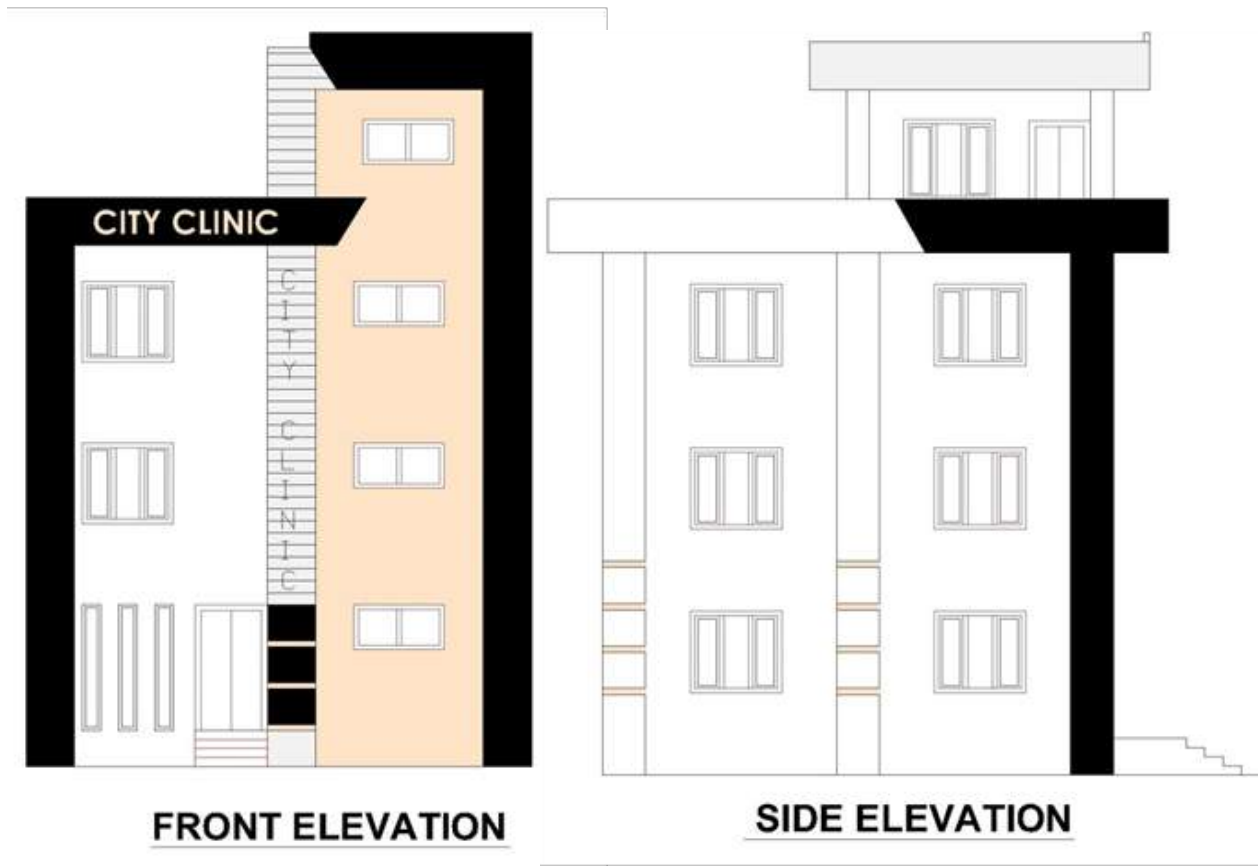


Figure A-13 Front and side elevation

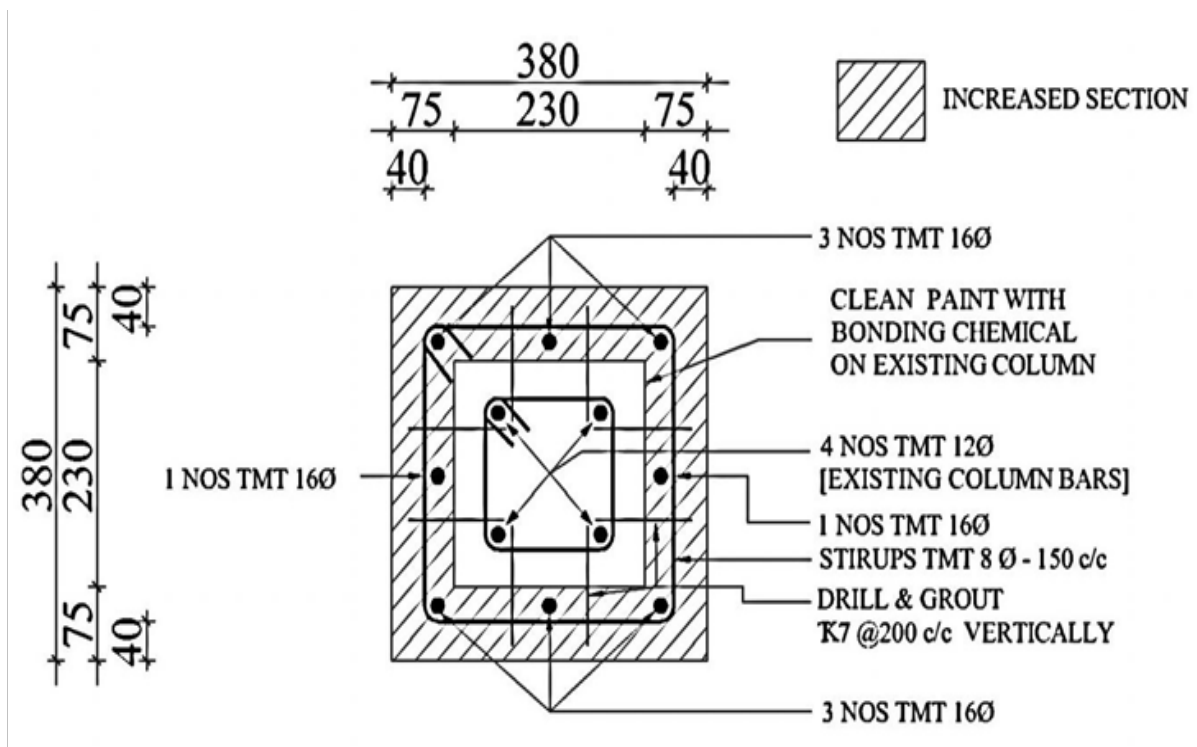


Figure A-14 Section of jacketed column C1

EXAMPLE 3: STRESS CHECK CALCULATION USING FEMA 310

The following is a sample of quick check calculations based on FEMA 310 for the seismic evaluation of building under consideration.

Assumptions:

Unit weight of RCC = 24 KN/m³; Unit weight of brick = 19 KN/m³
Live load = 3 KN/m²; Live load at roof level without access = 1.5 KN/m²
Weight of plaster and floor finish = 1.0 KN / m²

Weight of timber = 6.45 KN/m²

Lateral load is solely carried by frame elements. Stiffness of the walls is not considered.

- Grade of concrete = M20 for all structural elements
- Grade of steel = Fe 415

A.2.9. Calculation for Shear Stress check

Summary of lumped load calculation

LOAD	MULTIPLIER
Dead	1
Live	0.25

Total mass for seismic weight calculation $W = 9661.32$ KN

Calculation of base shear (Using IS 1893 (Part I):: 2002)

Based on IS 1893 (Part 1): 2002, Criteria for earthquake resistant design of structures, Calculation of earthquake loads using Seismic coefficient method:

The design horizontal seismic coefficient, $A_h = \frac{ZIS_a}{2R_g}$

Where Z = Zone factor
 I = Importance factor
 R = Response reduction factor

$\frac{S_a}{g}$ = average response acceleration coefficient

The approximate fundamental natural period of vibration (T_a) in seconds, of all other buildings, including moment-resisting frame buildings with brick infill panels, may be estimated by the empirical expression:

$$T_a = \frac{0.09h}{d^{0.5}}$$

Where,

h = Height of Building in meter and
 d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force
 I = 1 (from clause 6.4.2, IS 1893 (Part 1) 2002)
 Z = 0.36

$$A_h = \frac{ZIS_a}{2R_g}$$

$$T_a = \frac{0.09h}{dx^{0.5}}$$

$$= \frac{0.09 \times 8.64}{29.53^{0.5}} = 0.1431 \text{ sec}$$

$$T_{ay} = \frac{0.09h}{dy^{0.5}}$$

$$\frac{0.09 \times 8.64}{7.92^{0.5}} = 0.2763 \text{ sec}$$

$$\frac{S_a}{E} = 2.5 \text{ (from graph 1893 (part 1)-2002)}$$

$$R = 5$$

$$A_h = \frac{ZIS_a}{2R_g}$$

$$= \frac{0.36 \times 1 \times 2.5}{2 \times 5} = 0.09$$

The total design lateral force or design seismic base shear (V_B) along any principal direction is determined by the following expression

$$V_B = A_h W$$

Where,

A_h = The design horizontal seismic coefficient

W = Seismic weight of the building

$$\square V_B = 0.09 \times 9661.32 = 869.52 \text{ KN}$$

Distribution of base shear and calculation of storey shear

The design base shear (V_b) is distributed along the height of the building as per the following expression:

$$Q_i = V_b \times \frac{W_i h_i}{\sum W_i h_i}$$

Where

Q_i = Design lateral force at floor i

W_i = Seismic weight of floor i

h_i = Height of floor i measured from base

STORY	STOREY FORCE Q_i (KN)	STOREY SHEAR V_j (KN)
3FL	353.9	353.9
2FL	337.94	691.84
1FL	177.68	869.52

Level	Storey Shears		A_c	n_c	n_{f1}	n_{f2}	$n_c - n_{f1}$	$n_c - n_{f2}$	Shear Stress	
	V_j (KN)	V_j (lb)							(in ²)	V_{1avg} (psi)
3	353.9	79287.6	3991	41	11	4	30	37	13.6	11.01
2	691.84	154999.5	4158	41	11	4	30	37	25.47	20.65
1	869.52	194806.9	4747	45	11	5	34	40	27.16	23.08

Where,

A_c = Summation of the cross sectional area of all columns in the storey under consideration

n_c = Total no. of columns

n_f = Total no. of frames in the direction of loading

V_{avg} = Average shear stress (psi) in the columns of concrete frames

$$V_{avg} = \frac{1}{m} \left(\frac{n_c}{n_c - n_f} \right) \left(\frac{V_j}{A_c} \right) \quad (\text{Ref: Fema 310 Clause 3.5.3.2})$$

m = component modification factor = 2 for buildings being evaluated to the life safety performance level

f'_c = specified compressive strength of concrete = 20 N/mm²

The average induced shear stresses are less than the permissible value of 100 psi or $2\sqrt{f'_c}$ (107.59 psi).

Hence, safe.

Calculation of Shear capacity of column using capacity design method

Checking shear capacity of column (E-3)

$$\text{Shear capacity of column (E-3) required} = \frac{1.2(M^l + M^r)}{h_{st}} \quad (\text{Ref: IS13920:2013 Clause 7.3.4})$$

$$\text{Ultimate capacity of beam} = f_y \times A_{st} \times d_{eff} \quad (\text{Ref: IS 456:2000 ANNEX G})$$

Where,

$$d_{eff} = d \left\{ \frac{1 - (A_{st} - A_{sc}) f_y}{f_{ck} b d} \right\} \quad (\text{Ref: IS 456:2000 ANNEX G})$$

Calculation of M^l

$$A_{st} = 226 \text{ mm}^2 \quad (3-16; 3-12) \quad ; b = 230 \text{ mm} \quad ; d = 200 \text{ mm}$$

$$A_{sc} = 226 \text{ mm}^2 \quad ; f_{ck} = 20 \text{ N/mm}^2$$

$$f_y = 415 \text{ N/mm}^2 \quad ; P_t = 0.49 \%$$

$$d_{eff} = 200 \left\{ \frac{1 - (226 - 226) \times 415}{20 \times 230 \times 200} \right\}$$

$$d_{eff} = 200 \text{ mm}$$

$$\begin{aligned} \text{Beam moment capacity } M^l &= 415 \times 226 \times 200 \text{ Nmm} \\ &= 18.76 \text{ KN-m} = M^2 \end{aligned}$$

$$\begin{aligned} \text{Hence, Shear capacity of column (E-3) required} &= \frac{1.2(18.76+18.76)}{2.88} \\ &= 15.63 \text{ KN} \end{aligned}$$

$$P_t \text{ provided} = \frac{1206 \times 100}{230 \times 350} = 1.5 \%$$

From table 61, for $P_t = 1.5 \%$, M20 concrete $\tau_c = 0.72 \text{ N/mm}^2$

$$\text{Shear capacity of concrete section} = \frac{0.72 \times 230 \times 310}{1000} = 51.34 \text{ KN} > 15.63 \text{ KN}$$

Hence safe.

Checking shear capacity of column (D-3)

$$\text{Shear capacity of column (D-3) required} = \frac{1.2(M^l + M^r)}{h_{st}}$$

$$\text{Ultimate capacity of beam} = f_y \times A_{st} \times d_{\text{eff}}$$

Where,

$$d_{\text{eff}} = d \left\{ \frac{1 - (A_{st} - A_{sc})f_y}{f_{ck}bd} \right\} \text{ (Ref: IS 456:2000 ANNEX G)}$$

Calculation of M^l

$$\begin{aligned} A_{st} &= 427 \text{ mm}^2 \quad (3-16;3-12) & ; b &= 230 \text{ mm} & ; d &= 350 \text{ mm} \\ A_{sc} &= 226 \text{ mm}^2 & ; f_{ck} &= 20 \text{ N/mm}^2 \\ f_y &= 415 \text{ N/mm}^2 & ; P_t &= 0.53 \% \end{aligned}$$

$$\begin{aligned} d_{\text{eff}} &= 350 \left\{ \frac{1 - (427 - 226) \times 415}{20 \times 230 \times 350} \right\} \\ d_{\text{eff}} &= 332 \text{ mm} \end{aligned}$$

$$\begin{aligned} \text{Beam moment capacity } M^l &= 415 \times 427 \times 332 \text{ Nmm} \\ &= 58.83 \text{ KN-m} \end{aligned}$$

$$M^2 = 0$$

$$\text{Hence, Shear capacity of column (E-3) required} = \frac{1.2(58.83+0)}{2.88}$$

$$\begin{aligned} P_t \text{ provided} &= \frac{1206 \times 100}{230 \times 350} = 1.5 \% & & = 24.51 \text{ kN} \end{aligned}$$

From table 61, for $P_t = 1.5 \%$, M20 concrete $\tau_c = 0.72 \text{ N/mm}^2$

$$\text{Shear capacity of concrete section} = \frac{0.72 \times 230 \times 310}{1000} = 51.34 \text{ kN} > 24.51 \text{ kN}$$

Hence safe.

Axial Stress check

Axial stresses due to overturning forces as per FEMA 310

Permissible stress = 868 psi (0.3 f_c')

The axial stress of columns subjected to overturning forces P_{ot} is given by

$$P_{ot} = \left(\frac{1}{m}\right) \left(\frac{2}{3}\right) \left(\frac{Vh_n}{Ln_f}\right) \left(\frac{1}{A_c}\right)$$

Where,

n_f = Total no. of frames in the direction of loading = 5

V = Base shear = 869.52 KN = 194806.9 P

h_n = height (in feet) above the base to the roof level = 28.8ft

L = Total length of the frame (in feet) = 98.43ft.

m = component modification factor = 2

A_c = Summation of the cross sectional area of all columns in the storey under consideration = 4747 in²

P_{ot} = 0.8psi << 868 psi

Hence Safe

Check for torsion

Checking eccentricity between centre of mass and centre of stiffness at different floors

STORY	CENTER OF MASS		CENTER OF STIFFNESS		% ECCENTRICITY		WIDTH	
	X_m	Y_m	X_r	Y_r	E_x	E_y	L	B
ROOF	14.764	5.295	14.764	5.7	0.00	-4.19	29.528	9.673
2FL	14.764	5.297	14.764	5.643	0.00	-3.58	29.528	9.673
1FL	14.764	5.215	14.764	5.565	0.00	-3.62	29.528	9.673

Check for Strong column weak beam

Checking capacity of column E-3 at ground floor

$$\begin{aligned} \text{Ultimate capacity of beam} &= f_y \times A_{st} \times d_{eff} \\ \text{Beam moment capacity } M^l &= 415 \times 226 \times 200 \text{ N-mm} \\ &= 18.76 \text{ KN -m} = M^2 \end{aligned}$$

$$\text{Column moment capacity required} = 1.2 \times 18.76 = 22.51 \text{ KN -m}$$

$$\text{Column axial load} = 364 \text{ KN (factored)}$$

$$\frac{P_u}{f_{ck} b D} = \frac{364 \times 1000}{20 \times 230 \times 350} = 0.23$$

$$\frac{M_u}{f_{ck} b D^2} = \frac{22.51 \times 10^6}{20 \times 230 \times 350^2} = 0.04$$

Using SP – 16, P_t required is 0.15 % whereas P_t provided is 1.5 % .

Hence, the strong column-weak beam criteria meet.

Checking capacity of column D-3 at ground floor

$$\begin{aligned} \text{Ultimate capacity of beam} &= f_y \times A_{st} \times d_{\text{eff}} \\ \text{Beam moment capacity } M^l &= 415 \times 427 \times 332 \text{ Nmm} \\ &= 58.83 \text{ KN -m} = M^2 \end{aligned}$$

$$\begin{aligned} \text{Column moment capacity required} &= 1.2 \times 58.83 = 70.6 \text{ KN -m} \\ \text{Column axial load} &= 418 \text{ KN (factored)} \end{aligned}$$

$$= \frac{P_u}{f_{ck}bD} = \frac{418 \times 1000}{20 \times 230 \times 350} = 0.26 \quad \frac{M_u}{f_{ck}bD^2} = \frac{70.6 \times 10^6}{20 \times 230 \times 350^2} = 0.125$$

Using SP – 16 , P_t required is 1.6 % whereas P_t provided is 1.5. %

Hence, the strong-column weak-beam criterion does not meet.

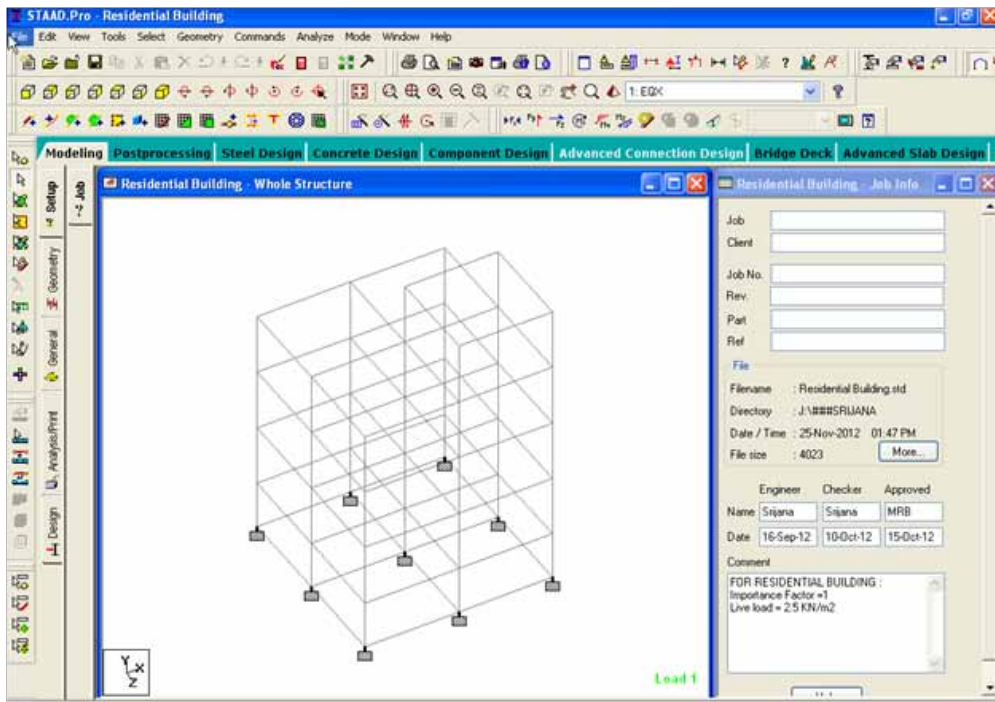
Check for out-of-plane stability of brick masonry walls

Wall type	Wall thickness	Recommended Height/ Thickness ratio ($0.24 < S_x \leq 0.35$) (Ref: Fema 310 Table 4-2)	Actual Height/ Thickness ratio in building	Comments
Wall in first storey,	230 mm	18	2530/230=11	Pass
	115 mm	18	2530/115 = 22	Fail
All other walls	230 mm	16	2530/230=11	Pass
	115mm	16	2530/115 = 22	Fail

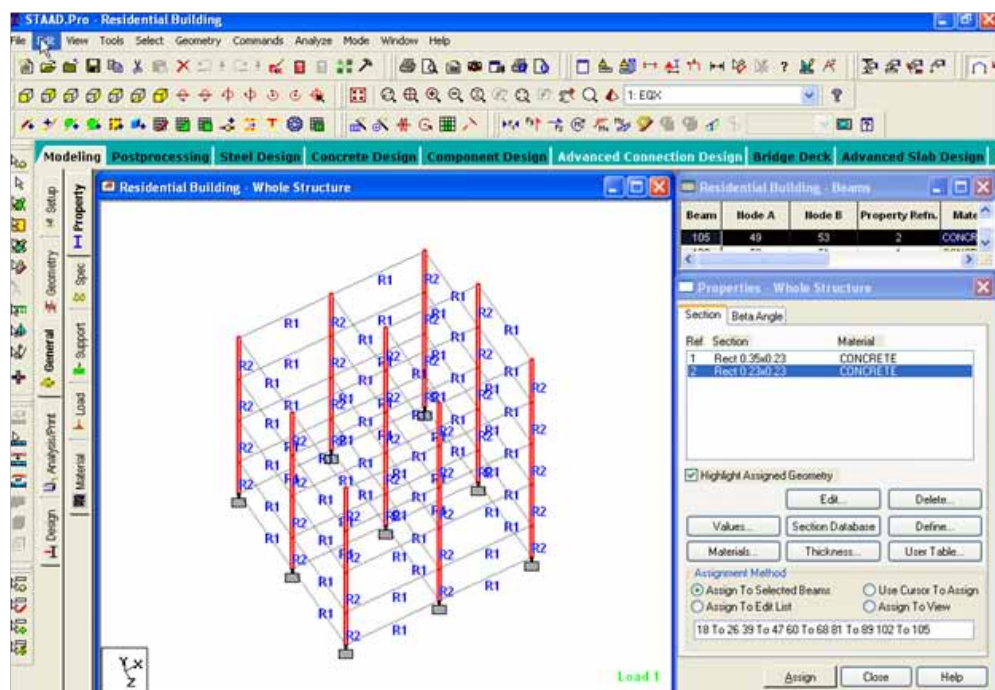
EXAMPLE 4: ANALYSIS OF BUILDING OF EXAMPLE 2 USING STRUCTURAL ANALYSIS PROGRAM-STAAD (STRENGTH BASED APPROACH)

Strength based Analysis of the structure with the given drawings was carried out for structural evaluation. The strength of the existing structure is evaluated and compared with the demand of the structure then structural members are retrofitted to fulfill the demand on the basis of strength.

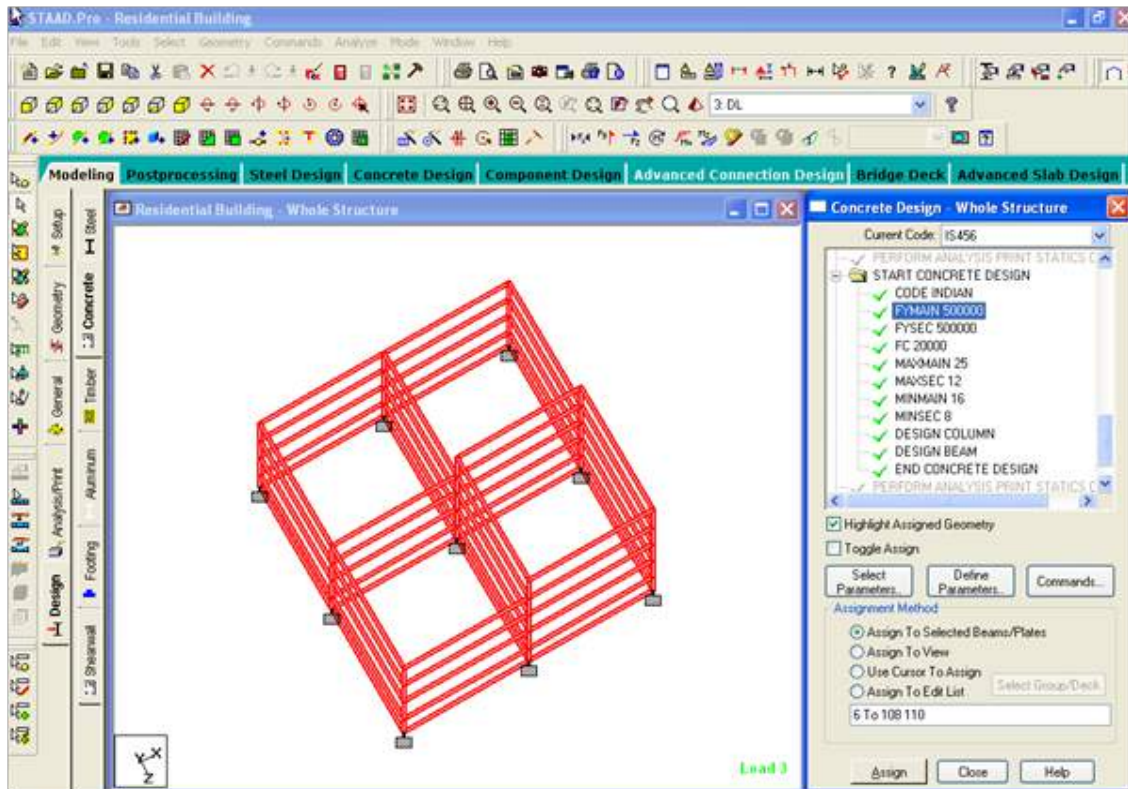
ANALYSIS IN STAAD



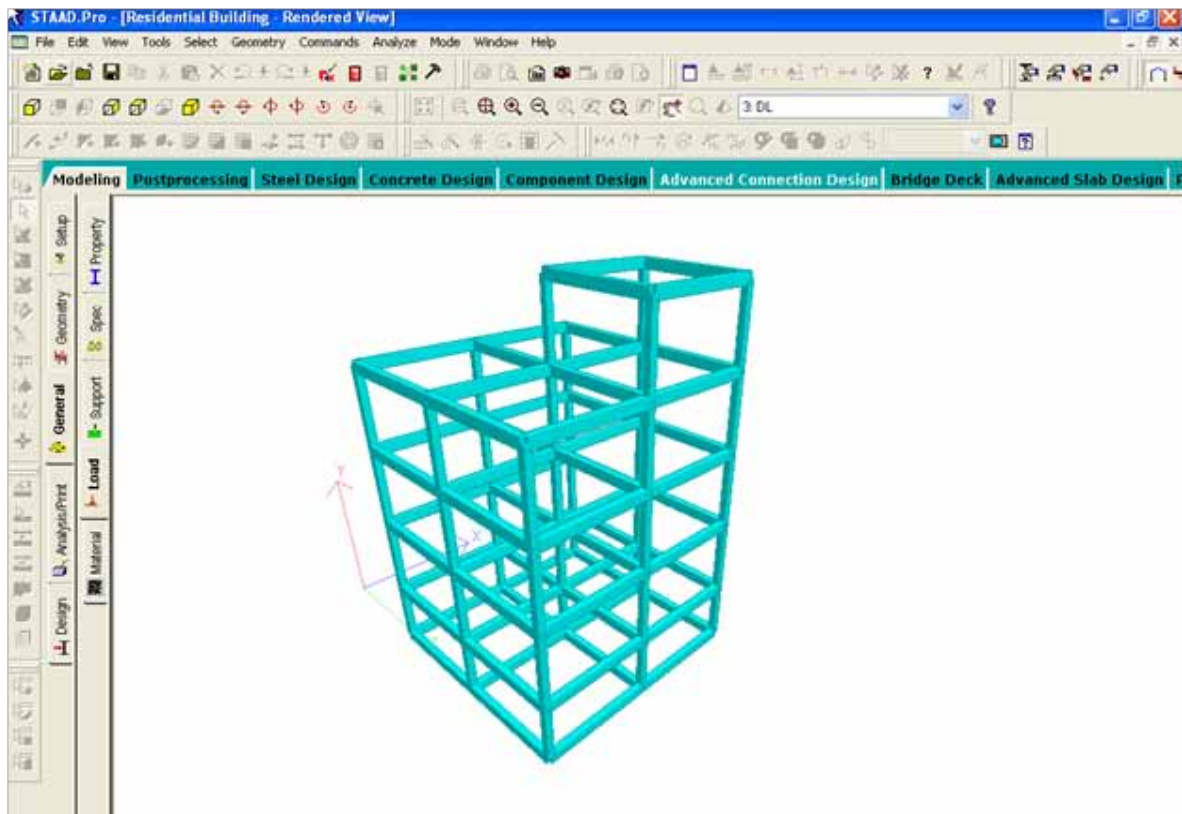
WIRE FRAME



COLUMN

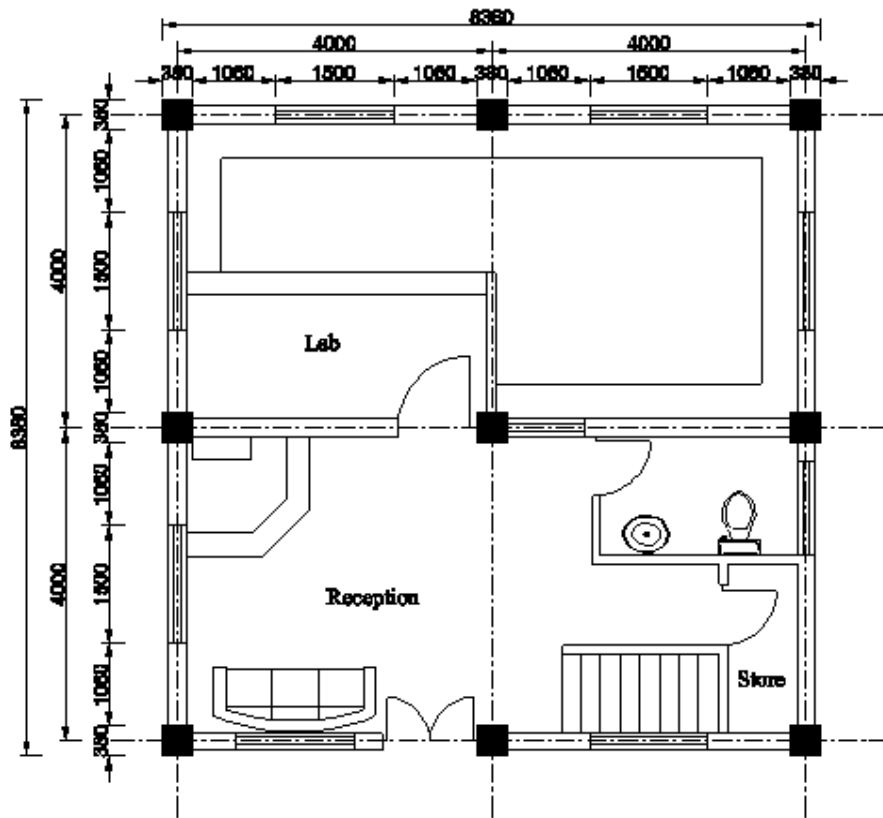


BEAM

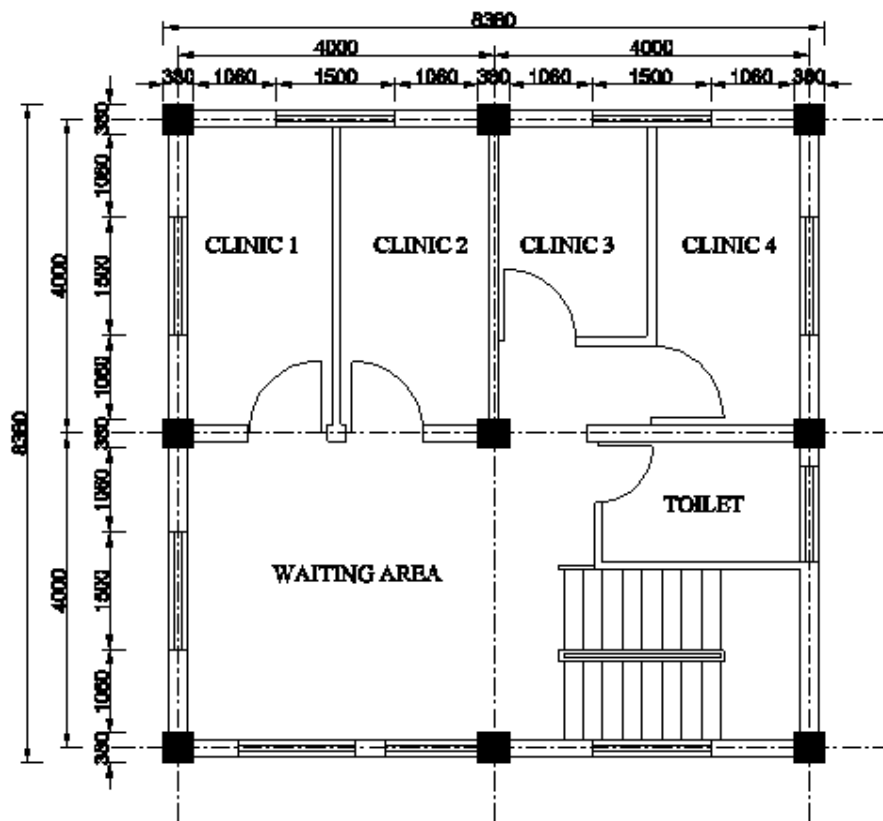


3D Model in Staad

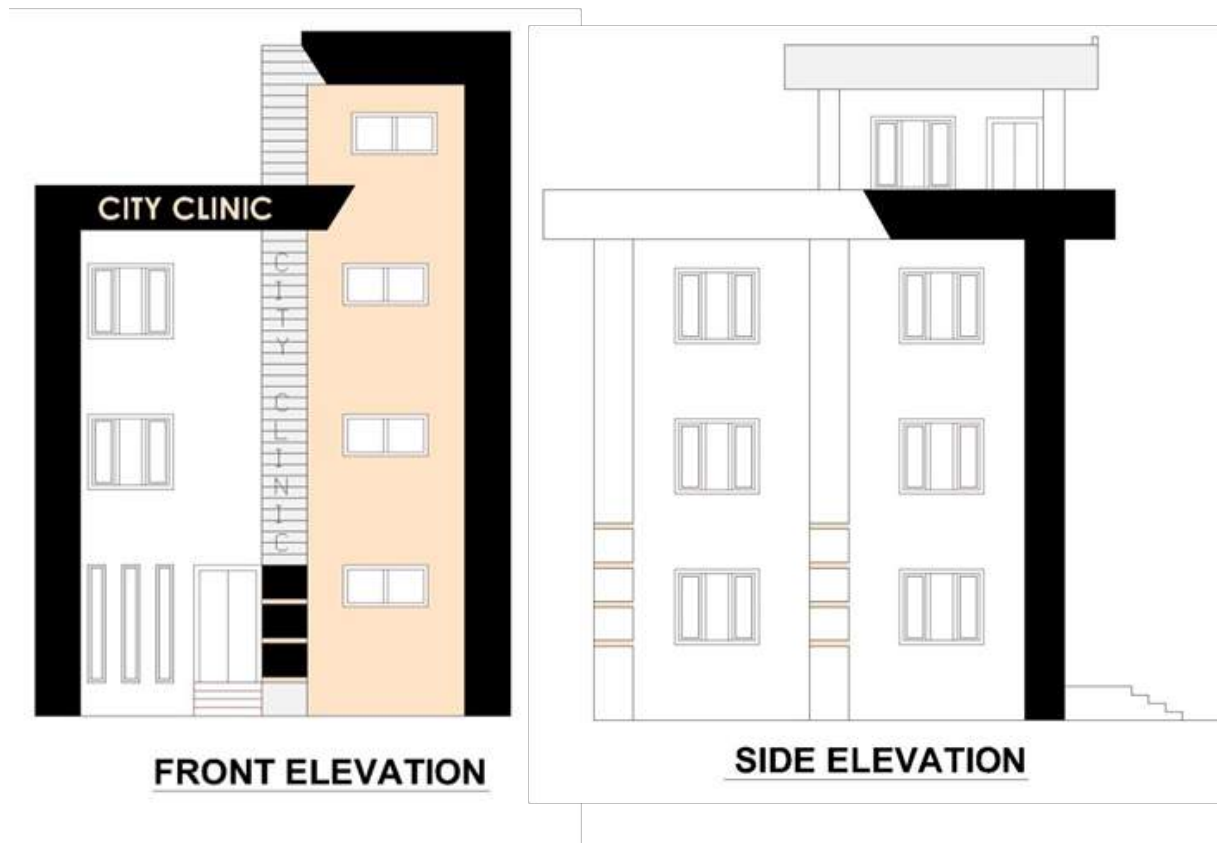
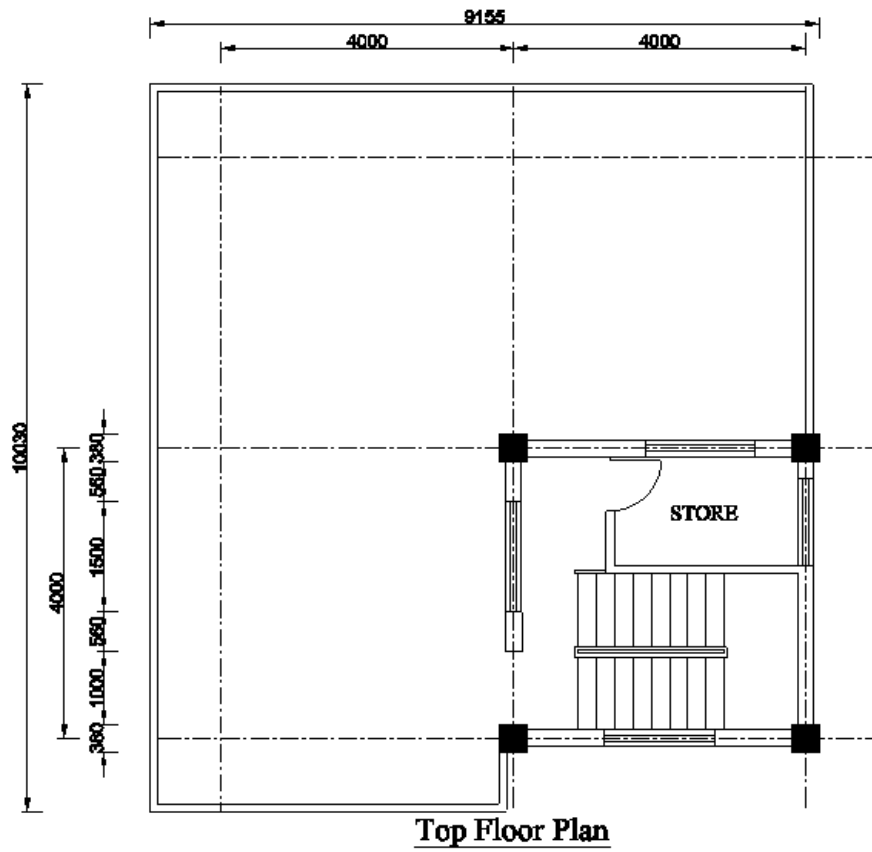
RETROFITTING DRAWINGS:

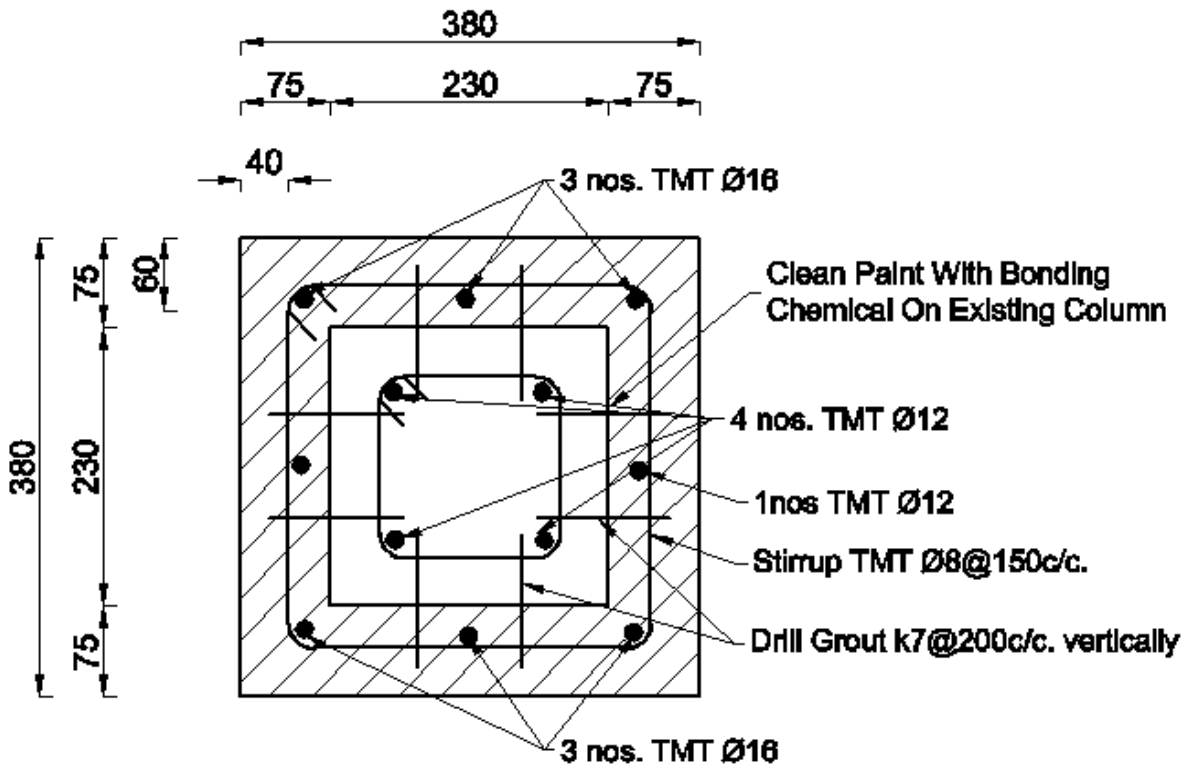


Ground Floor Plan



First Floor Plan





SECTION OF JACKETED COLUMN C1

EXAMPLE 5-A: PERFORMANCE ANALYSIS USING NON-LINEAR STATIC ANALYSIS (STATIC PUSHOVER ANALYSIS) OF BUILDING IN EXAMPLE -2, BEFORE RETROFITTING

Performance Analysis of the structure with the given drawings was carried out for structural performance evaluation. The performance evaluation of non-structural components and the combined performance is not evaluated. Static Pushover Analysis is used to evaluate the performance of the structure.

A.2.10. OUTPUT

The analysis is carried out for Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) as defined in IS 1893 (2002). The main results for both DBE and MCE are given below.

A.2.11. PERFORMANCE OF EXISTING STRUCTURE

The existing structure can't survive the Design Basis Earthquake as well as Maximum Considered Earthquake (MCE) since its capacity is low than the Demand.

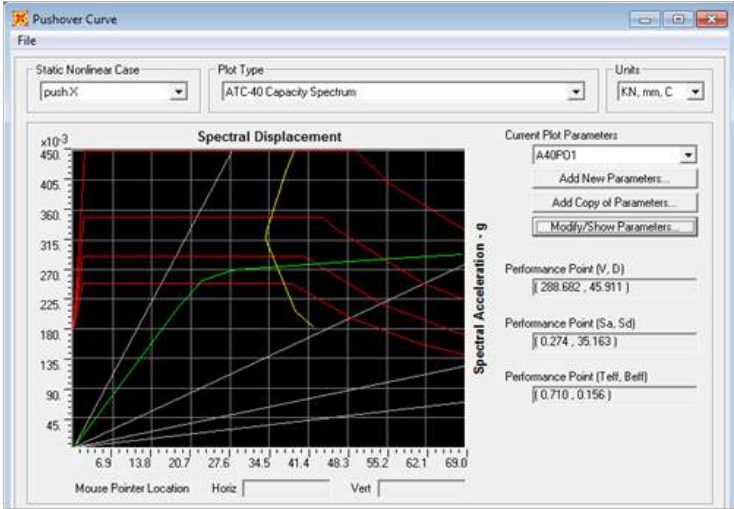


Figure A-15 Capacity Spectrum of Existing Structure at DBE

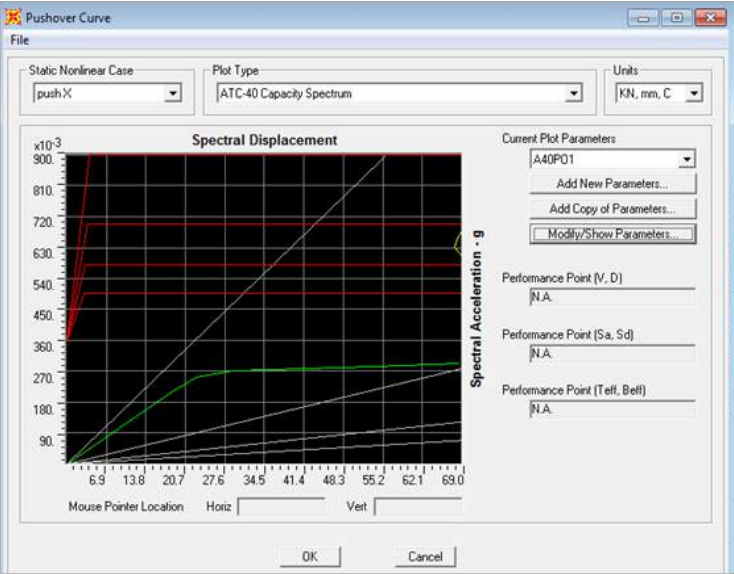
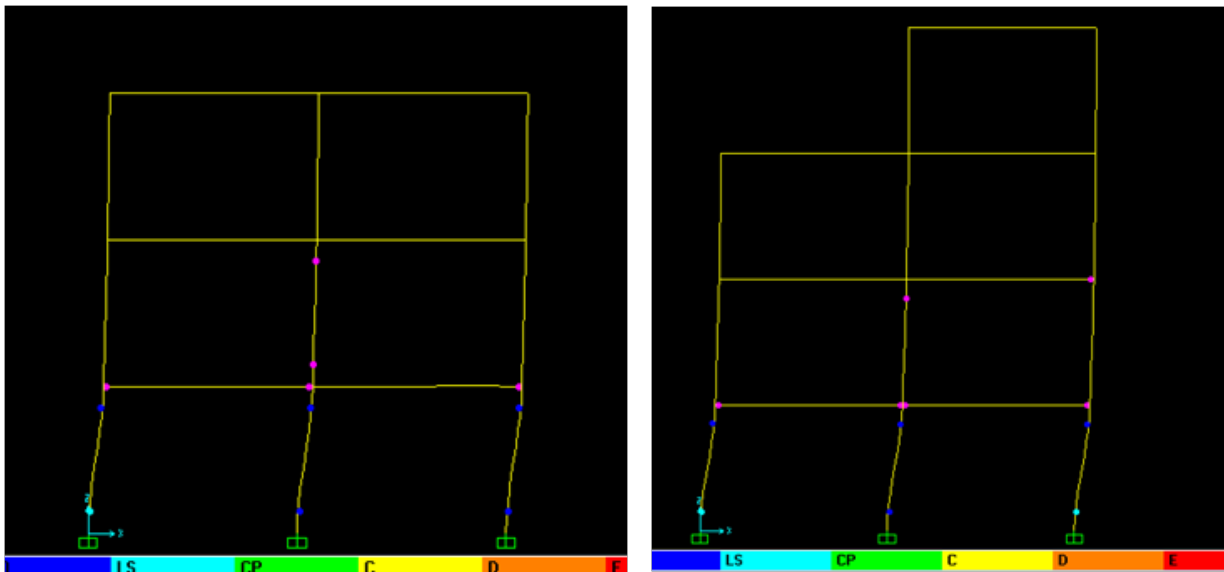


Figure A-16 Capacity Spectrum of Existing Structure

A.2.12. PLASTIC HINGES MECHANISM

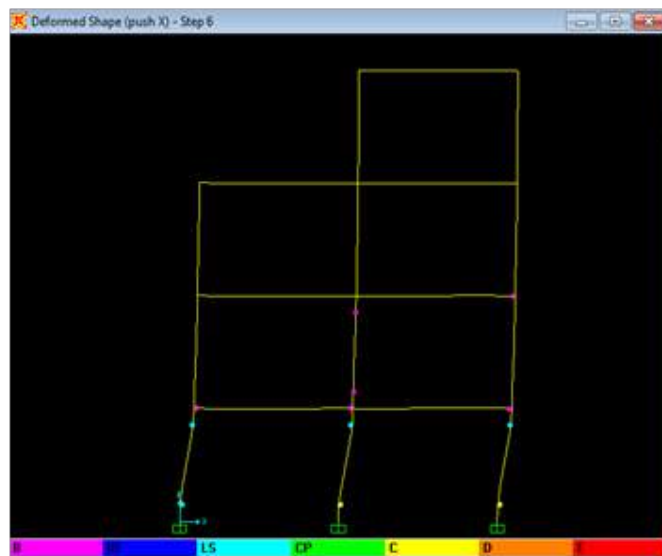
Plastic hinge formation for the building mechanisms have been obtained at different displacements levels. The hinging patterns are as shown in figures



Building I

Plastic hinge (push X) of Grid 1-1

Plastic hinge (push X) of Grid 2-2



Plastic hinge (push X) of Grid 3-3

A.2.13. FINDINGS AND RECOMMENDATION

The analysis of the building was carried out using the static analysis and push over analysis in SAP 2000 and the buildings are found to be unsafe.

- Demand base shear is 516.726KN but base shear obtain is less than required in both Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE)

Hence the building cannot be considered safe for the hospital purpose. It needs retrofitting measures to increase capacity for the earthquake safety.

EXAMPLE 5-B: PERFORMANCE ANALYSIS USING NON-LINEAR STATIC ANALYSIS (STATIC PUSHOVER ANALYSIS) OF BUILDING IN EXAMPLE -2 AFTER RETROFITTING

A.2.14. OUTPUT

The analysis is carried out for Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) as defined in IS 1893 (2002). The main results for both DBE and MCE are given below.

A.2.15. DESIGN BASIS EARTHQUAKE (DBE)

For Design Basis Earthquake (DBE), the performance point of retrofitted structure appears at Sa (spectral acceleration) equal to 0.382 and maximum roof deflection (D) is 0.525 inches as shown in the image below.

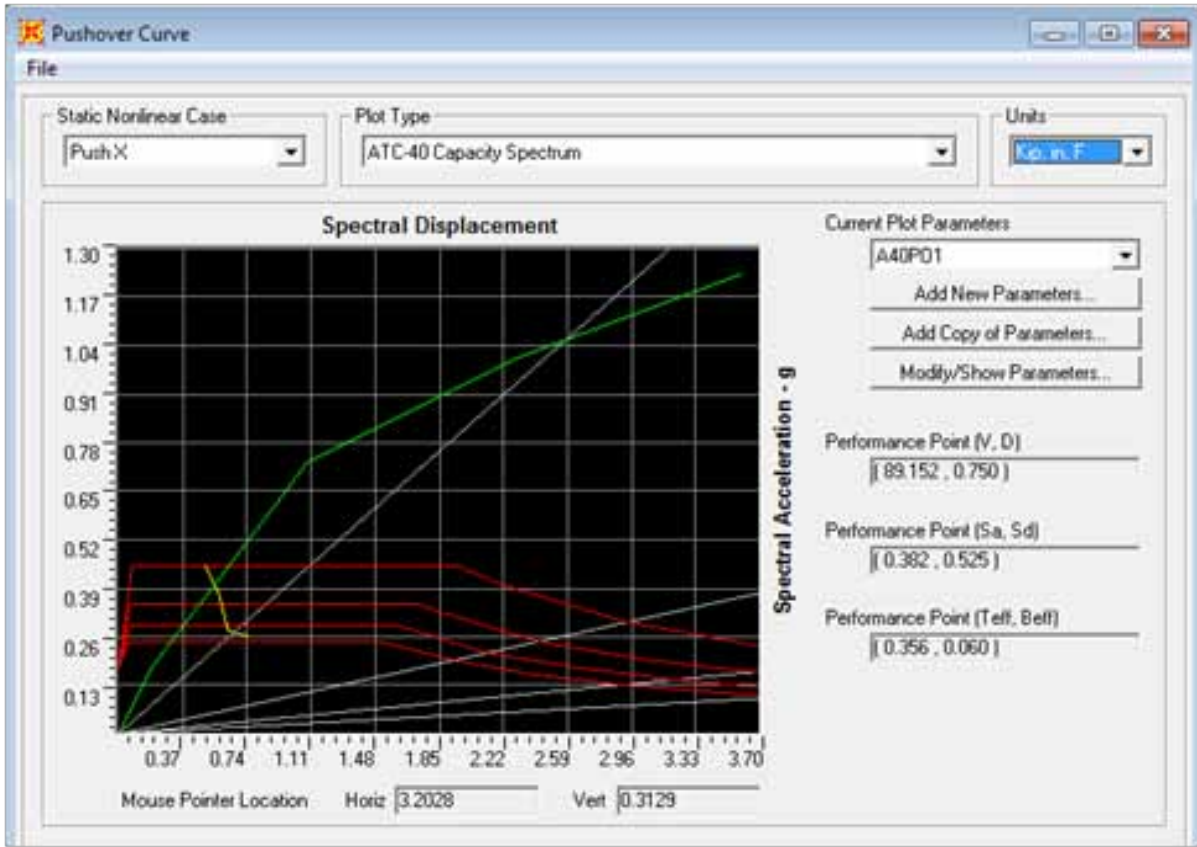


Figure A-17 Capacity Spectrum of Retrofitted Structure at DBE

A.2.16. Maximum Considered Earthquake (MCE)

For Maximum Considered Earthquake (MCE), the performance point is obtained at Base shear (V) and Roof Displacement (D) at 772.160 and 42.01mm, respectively. The spectral acceleration performance point is 0.744 which is higher than the spectral acceleration value of 0.36 given in IS 1893 codal provisions.

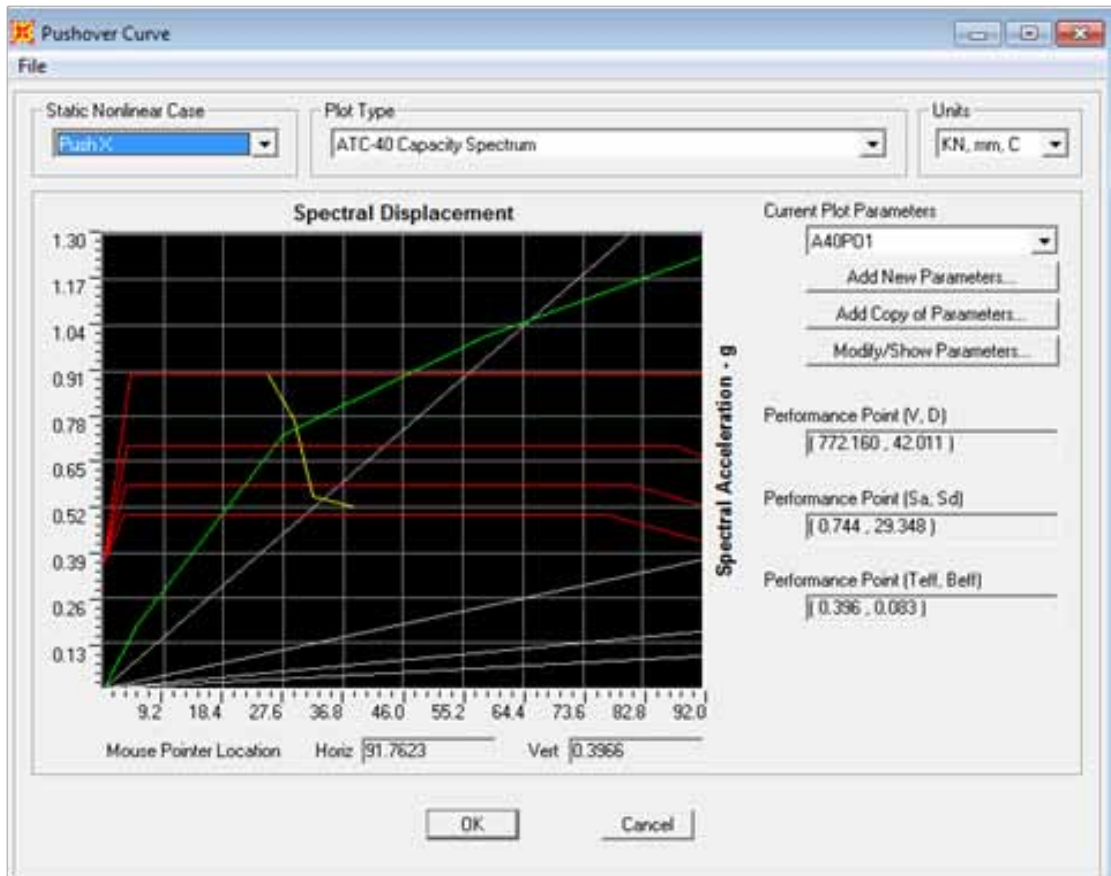
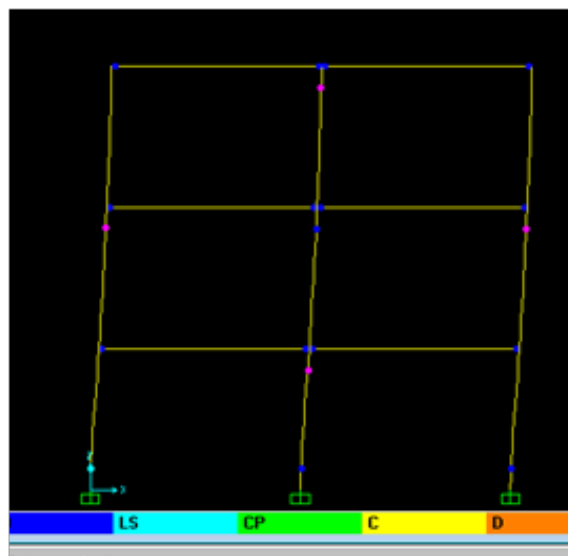


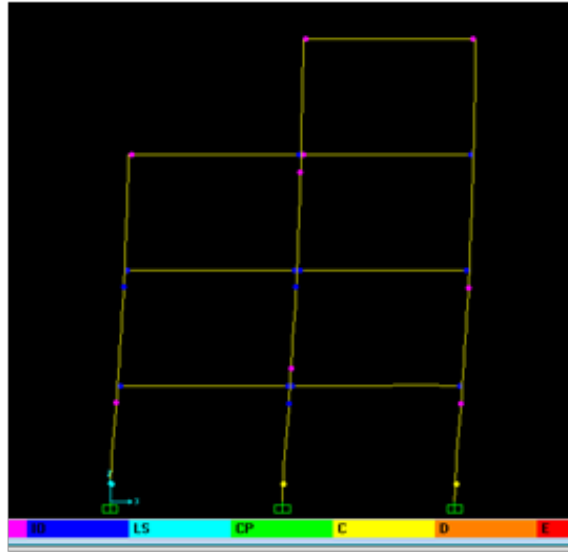
Figure A-18 Capacity Spectrum of Retrofit Structure at MCE

A.2.17. PLASTIC HINGES MECHANISM

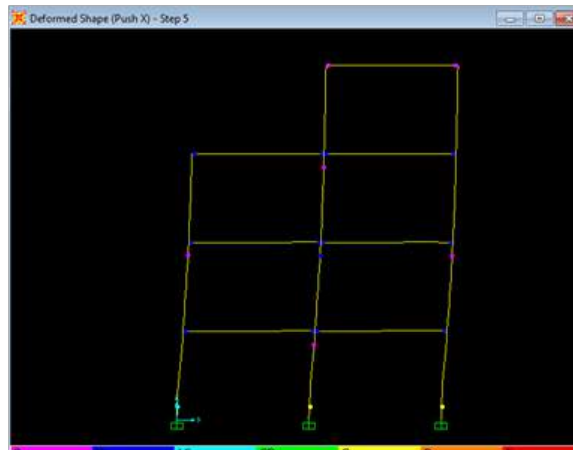
Plastic hinge formation for the building mechanisms have been obtained at different Displacement levels. The hinging patterns are as shown in figures:



Grid 1-1



Grid 2-2



Grid 3-3

A.2.18. CONCLUSION

Thus the analysis of the building after RC jacketing of columns was carried out by modeling the building in SAP 2000 and the results were found as mentioned above from the push over analysis. The building is found to be safer in Push over analysis.

Hence the retrofitted building can be considered safe for the hospital purpose.

From the performance analysis of the retrofitted building according to the drawings provided and field verification, the retrofitted building will be in damage control level. The performance objective for DBE is expected at Life Safety level and for MCE it's expected at Structural stability. As the performance of the building is realized at damaged control level (better performance than the expected performance), **the building is safe to use as per the requirements of IS 1893-2002.**

EXAMPLE 5-C: ANALYSIS AND RETROFITTING DESIGN OF SCHOOL BUILDING
A DETAIL EXAMPLE:
BUILDING DESCRIPTION

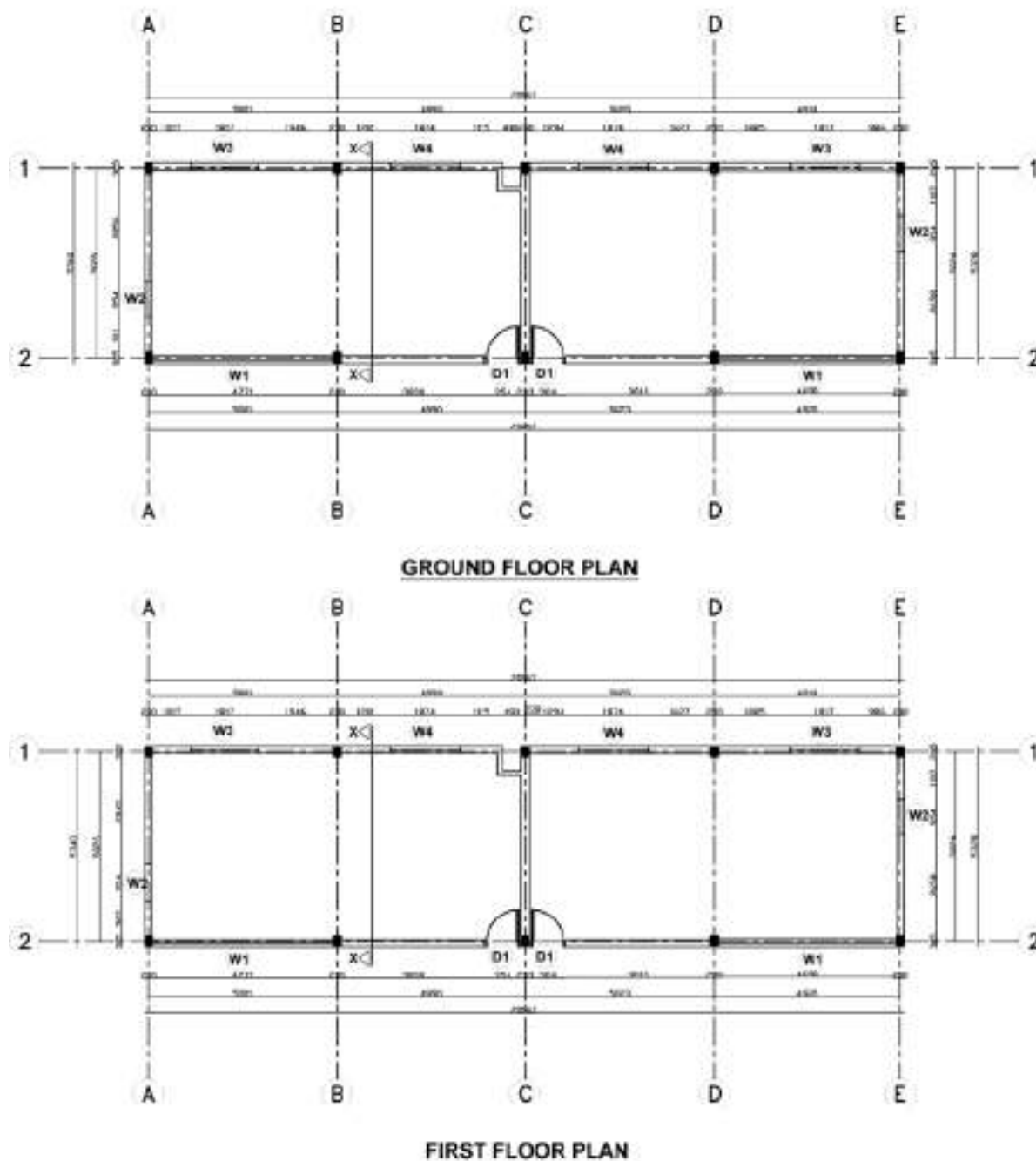


Figure A-19 Plan of the School Building

SUMMARY OF THE BUILDING

Shape of Building:	Rectangular
Building Dimension- Length (m):	19.949
-Width (m):	5.071
Plinth Area (sq.m):	101.16
Number of Storey:	2
Total Height of Building (m):	5.262
Building Type:	RC Framed Building

Wall Thickness (mm):	230
Joint Mortar:	Cement sand
Floor Finish :	Cement Punning
Type of Foundation:	RCC Isolated Foundation
Presence of- Lintel Band	No
-Sill Band	No

Structural Analysis & Retrofitting Design

The structural configuration of the building is Reinforced concrete 3-storied structure. The plan(Grid Lines) of the building is illustrated in Figure . The structural analysis of the building was done using linear static analysis with the help of structural analysis software named ETABS 2013 V 13.1.5.

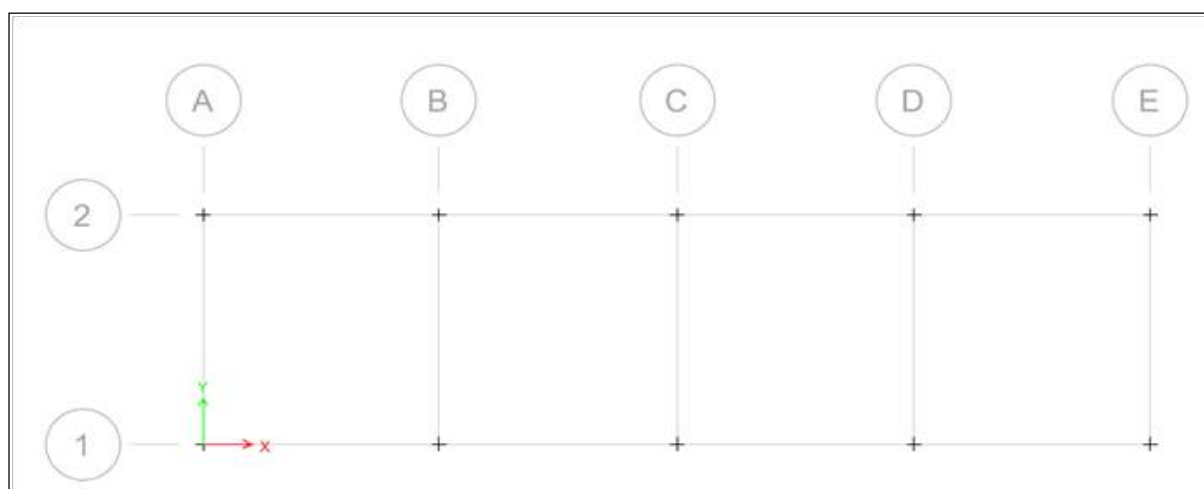


Figure A-20 Ground Floor Plan with Grid Lines of Analysis

GENERAL INPUT

Loads and Loading

The main types of loads considered for the design of building structure are vertical loads (dead and imposed load) and lateral load lateral loads (earthquake load).

a) Dead Loads:

The gravity loads due to self-weight of structural elements are determined considering the dimensions of elements and unit weights as per IS: 875 (part 1), 1987. The dead load is considered the weight of structural elements including walls, finishing work and all other permanent features in the building. The wall loads are calculated and applied on beams and slabs as per measured drawings

b) Live Loads:

The live load considered for various usage of space office, corridor, lobbies, parking and staircase are taken as per codal provision in IS: 875 (part 2), 1987.

c) Earthquake Loads:

Earthquake load is calculated using Seismic coefficient of equivalent static force analysis method for zone V according to the codal provision in IS: 1893 (part 1), 2002. The soil type is taken as III and Importance Factor is taken as 1.0. However only 70% of the lateral load is considered according to the cl. 5.4 of IS 15988: 2013 Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings - Guidelines

Types and Grades of Material

Concrete and reinforcement steel are main basic material for reinforced concrete structure. The concrete used in the building is of Grade M15, and steel is of Grade Fe 415.

Depth of foundation

The depth of foundation is mainly governed by factors such as scour depth and nature of subsoil strata to place foundation, basement requirement and other environmental factors. As there are no rivers in the immediate vicinity of the building site, chance of scouring is absent. The foundation depth is assumed as per general practices.

Structural Data

As per measured drawing and Non-Destructive tests carried out at the site following data were used for the structural analysis:

ID	Designation	Size(DXB)	Grade	Top Rebar	Bottom rebar
BEAM	B1	375x230	M20	3-20□	3-20□
ID	designation	size(DXB)	grade	rebar	
COLUMN	C1	230x350	M20	8-20□	
ID	designation	size(D)	grade	edge rebar	mid span rebar
SLAB	S-100		M15	X - 10 mmØ @ 100 mm C/C	X - 10mm Ø @ 100 mm C/C
				Y - 10mm Ø @ 100 mm C/C	Y - 10mm Ø @ 100 mm C/C

A.2.19 Modeling and structural Analysis

ETABS 2013 V 13 was used as a tool for modeling and analysis of the building. ETABS 2013 V 13 is the most sophisticated and user friendly series of computer programs. Creation modification of models, execution of analysis and checking and optimization of the design can be done through this single interface. Structural analysis program ETABS 2013 V 13 is used for modeling and structural analysis to check the member capacity of structure as per existing design and construction.

Analysis Approaches for Building

The structure was modeled as a three dimensional ordinary RC moment resisting frame of main structural member beam and column to determine the required strength of the structure. The effect of infill brick wall is not considered while analyzing structure. The gravity (dead and live) load applied in combination with lateral load (seismic load) as recommended by IS 1893 (part1) 2002 in analysis. The analysis was performed for various combinations as per IS 1893 (part1):2002.

The analysis of the building was done based upon the following two cases:

1. Case I: Considering existing building
2. Case II: Considering existing building after improvement of structural members (Retrofitting)

Case I: Considering existing building

Analysis Input

The Figure and Figure illustrate the 3d model and girds of the building for ETABS analysis respectively

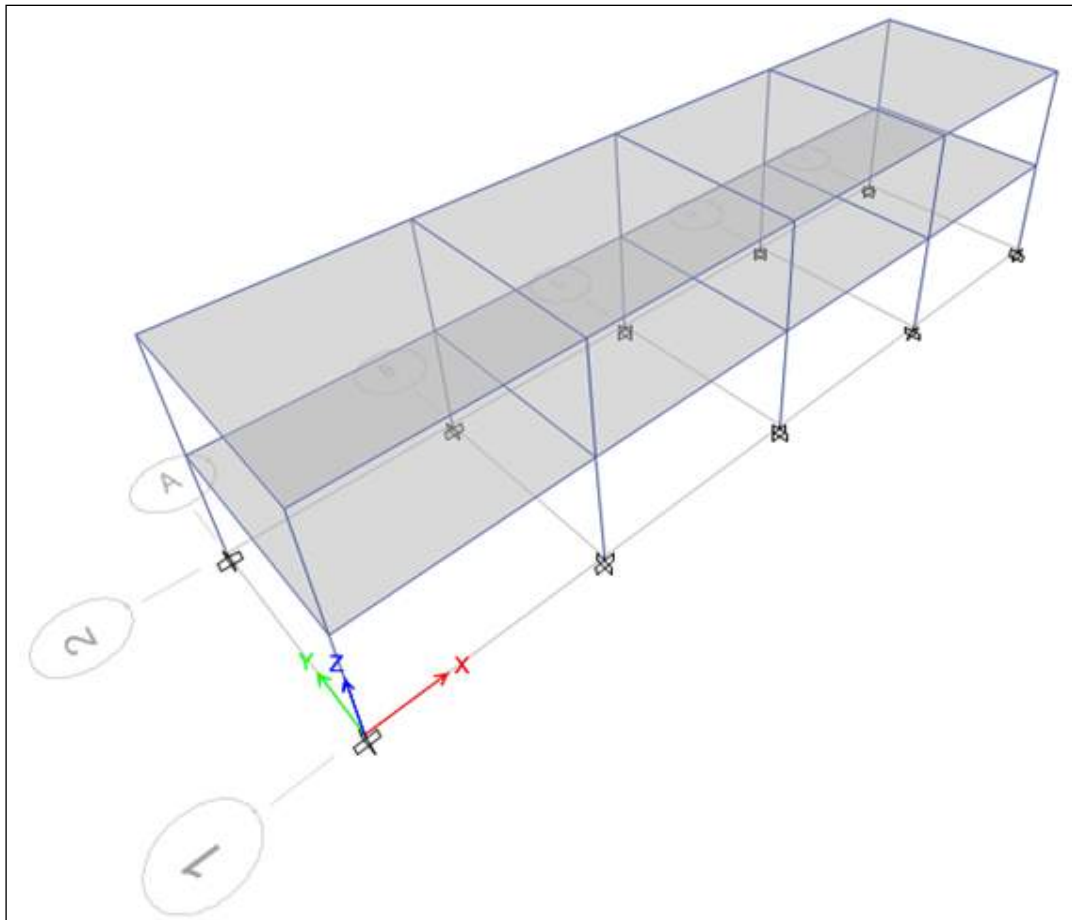


Figure A-21 3d model of the building with grid lines

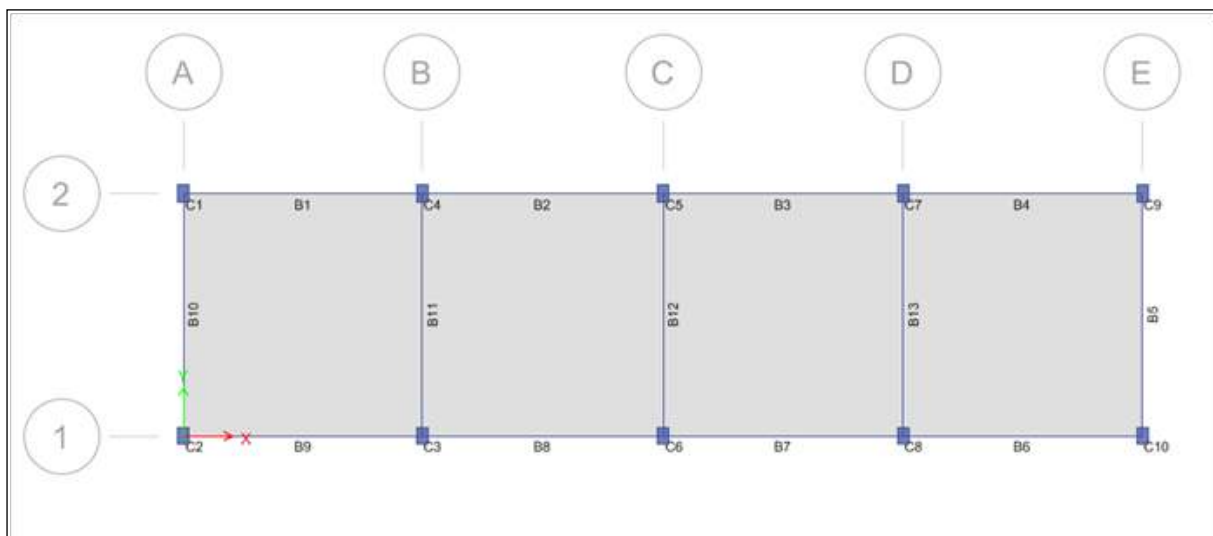


Figure A-22 Grid Lines with Beam and Column ID

The various parameters considered for analysis is presented below:

Zone factor,	Z	0.252
Importance Factor,	I	1.5
Reduction Factor	R	3
Damping	DAMP	0.05

Only 70% of the earthquake is considered here reducing zone factor to 0.252

Codal Calculation: As per IS 1893-(part1):2002

Clause 6.4.2: For any structure with $T < 0.1$, A_h shall not be less than $Z/2$

Cl.7.6.2, $T_x = 0.09h/\sqrt{D_x}$

h	=	5.262	m	D_x	=	19.49	m
D_y	=	5.07	m	T_x	=	0.1073	Sec
T_y	=	0.2103	Sec				

The building was analyzed as per fore-mentioned criteria and findings are shown below. The existing building was found to be safe in drift criteria. Also, given below is the table which compares existing reinforcements with the required values from analysis:

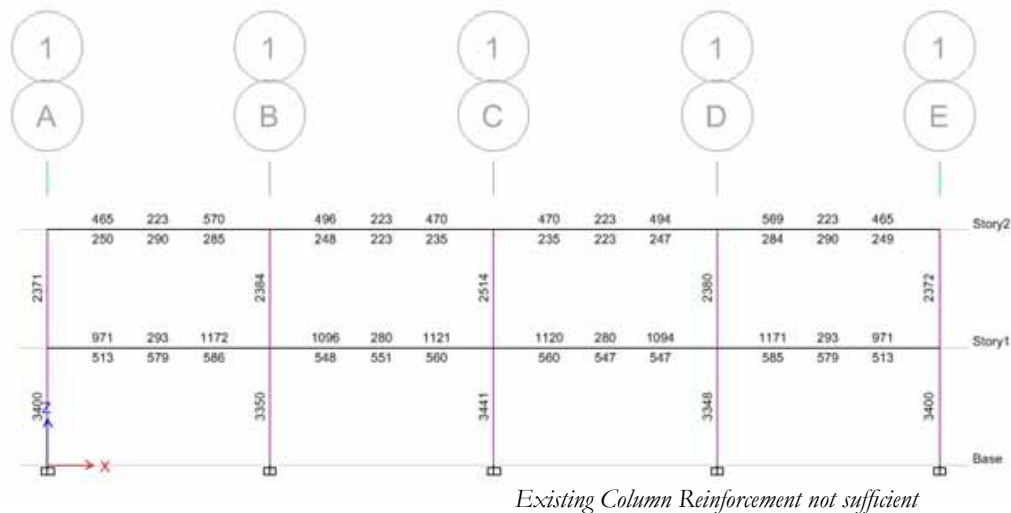


Figure A-23 Grid 1-1

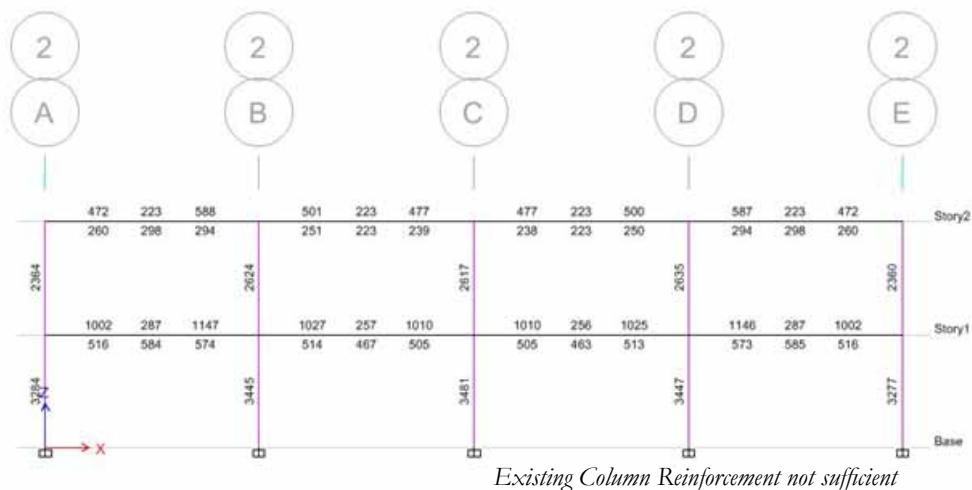


Figure A-24 Grid 2-2

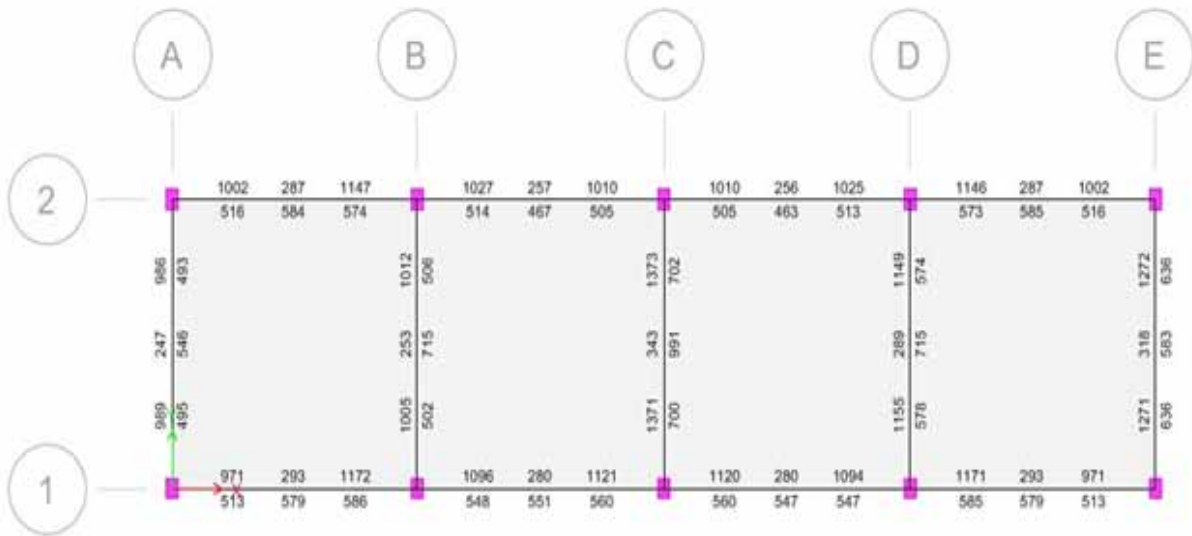


Figure A-25 Storey 1

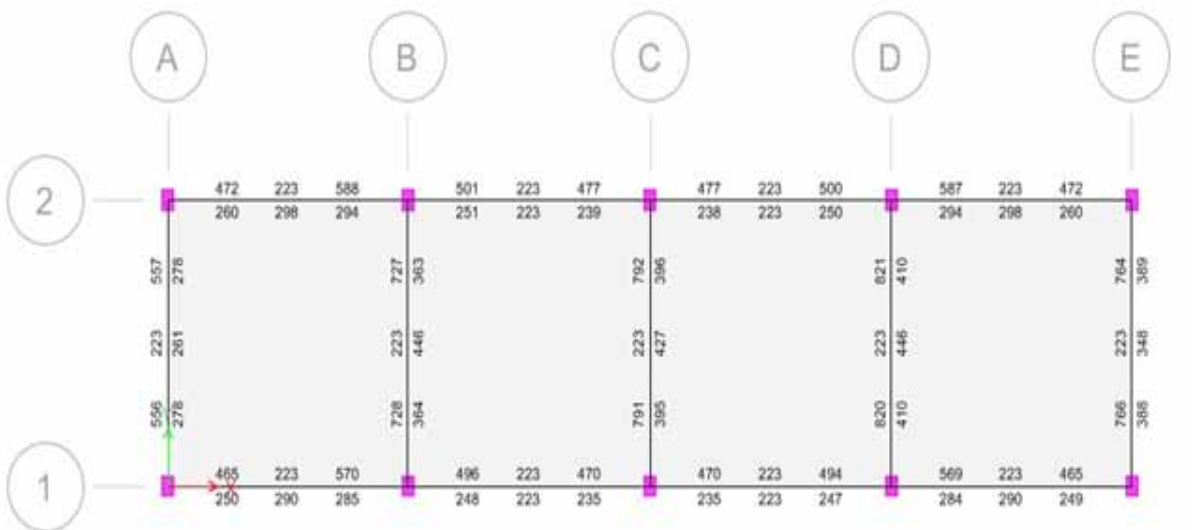


Figure A-26 Storey 2

It is observed that the size of the beam and columns is not appropriate for the existing structure. Also, the required reinforcements are seen to be on the higher side than the existing reinforcement area and thus the building requires some seismic strengthening as the overall condition of the building seemed to be vulnerable to earthquakes. The details of existing and required reinforcements for the beam is shown in Error! Reference source not found. whereas the details of existing and required reinforcements in columns in shown in Table 1.

Table1: Required and existing reinforcements in beams

TABLE: Concrete Beam Flexure Envelope			Indian code			
Label	Story	Section	As Top mm ²	As Bot mm ²	Area Existing	
					Top	Bottom
B6	Story2	B-230X375-M20	569	290	942	942
B7	Story2	B-230X375-M20	494	247	942	942
B8	Story2	B-230X375-M20	496	248	942	942
B9	Story2	B-230X375-M20	570	290	942	942
B5	Story1	B-230X375-M20	1272	636	942	942
B6	Story1	B-230X375-M20	1171	585	942	942
B7	Story1	B-230X375-M20	1120	560	942	942
B8	Story1	B-230X375-M20	1121	560	942	942
B9	Story1	B-230X375-M20	1172	586	942	942

Beam reinforcement lagging

Table 2: Required and existing reinforcements in columns

Label	Story	Section	P	M Major	M Minor	Required steel(IS)	Main Bar Existing	
			kN	kN-m	kN-m	sq. mm	area	%
C2	Story2	C-230*350-M20	76.6208	-41.7207	-29.3236	2371	2512	3.12%
C3	Story2	C-230*350-M20	111.6689	-42.5493	-28.5133	2384	2512	3.12%
C6	Story2	C-230*350-M20	108.9562	39.4353	-41.3975	2514	2512	3.12%
C8	Story2	C-230*350-M20	111.5674	42.4764	-28.5031	2380	2512	3.12%
C10	Story2	C-230*350-M20	76.6452	41.7427	-29.3268	2372	2512	3.12%
C2	Story1	C-230*350-M20	219.6013	-63.6746	-14.0951	3400	2512	3.12%
C3	Story1	C-230*350-M20	293.2828	-64.6745	-9.1219	3350	2512	3.12%
C6	Story1	C-230*350-M20	350.7592	63.1464	-14.7266	3441	2512	3.12%
C8	Story1	C-230*350-M20	292.9997	64.6451	-9.1174	3348	2512	3.12%
C10	Story1	C-230*350-M20	219.6563	63.6819	-14.0897	3400	2512	3.12%

Case II: Considering existing building after improvement of structural members (Retrofitting)

The results in the case I show that the building is not safe in existing conditions with regard to the lateral load considered and thus the building has to be retrofitted. This case will see the possibility of increment in the size of columns. The reinforcements in the existing building also seem to be on lesser side. Thus, nominal increase in the reinforcements are done while increasing the sizes of the frames whose reinforcements are found to be okay with the existing ones.

Analysis Input

The Figure A-27 illustrates the 3d model and girds of the building for ETABS analysis.

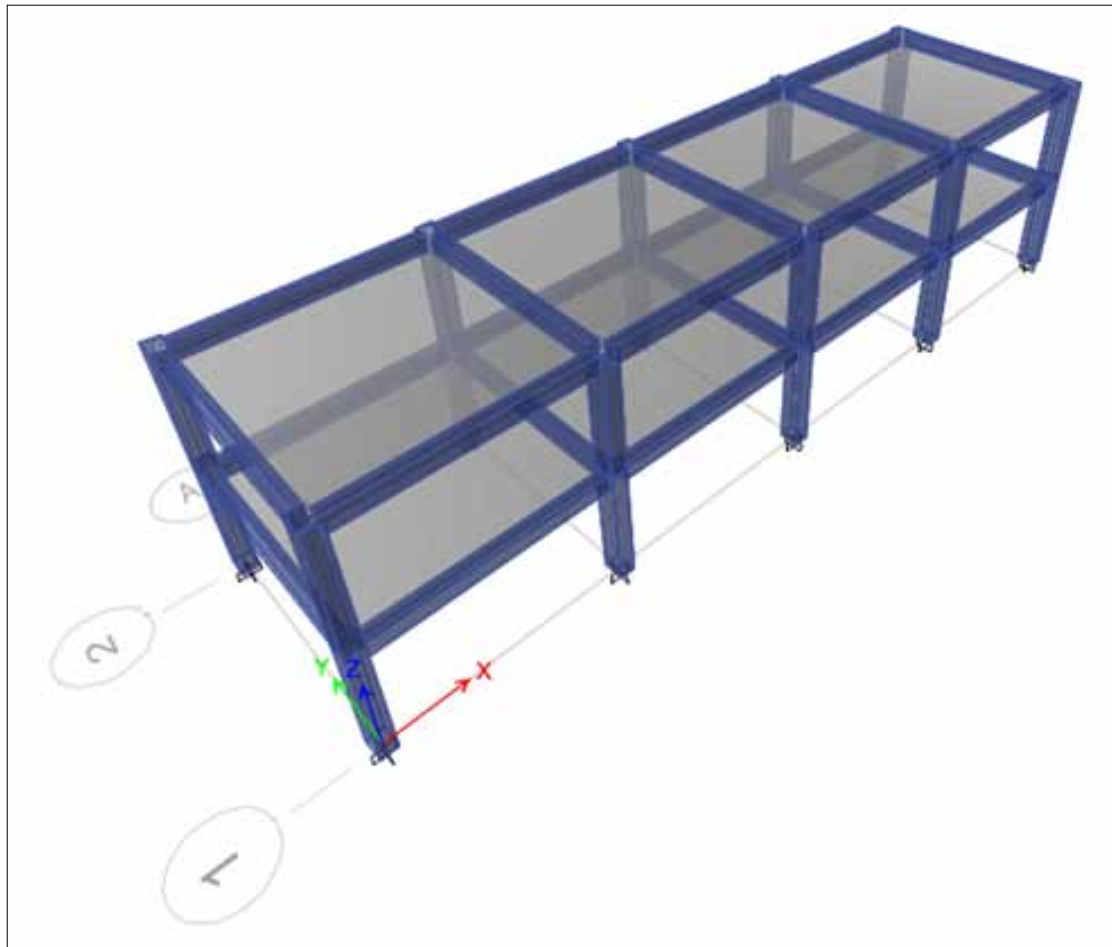


Figure A-27 3d model of the building with grid lines

The size of the structural members recommended after retrofitting is given below:

ID	Designation	size (DXB)	Grade	Top Rebar	Bottom Rebar
BEAM	B1	375x230	M20	3-20□	3-20□
ID	designation	size(DXB)	grade	rebar	
COLUMN	C1	430x550	M20	8-20□	
ID	designation	size(D)	grade	edge rebar	mid span rebar
SLAB	S-100		M15	X - 10 mmØ @ 100 mm C/C	X - 10mm Ø @ 100 mm C/C
				Y - 10mm Ø @ 100 mm C/C	Y - 10mm Ø @ 100 mm C/C

The beams and columns ID for the floor are shown in Figure 20:

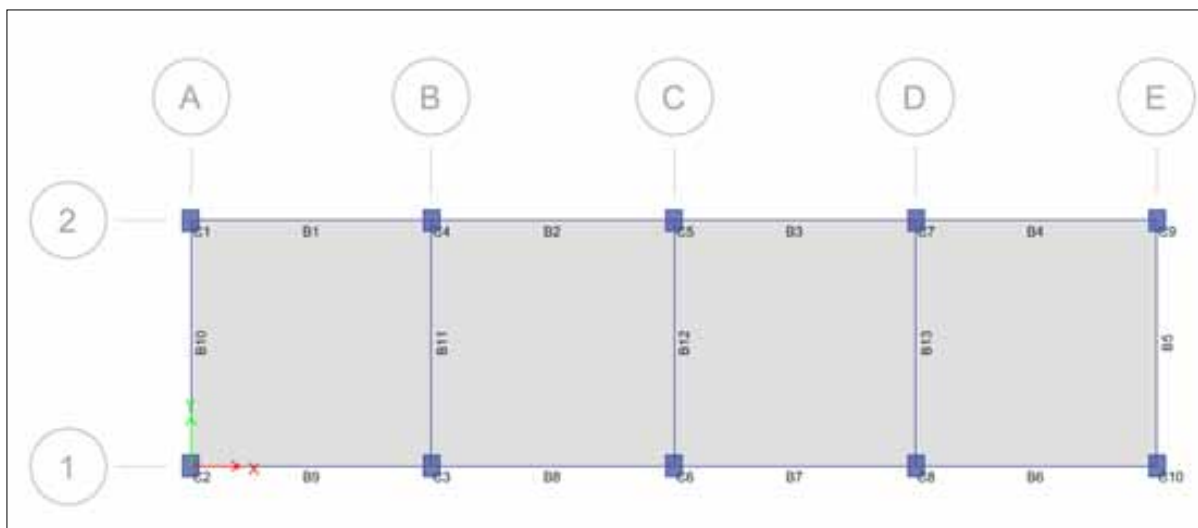


Figure A-28 Grid Lines with Beam and column ID

The details of existing and required reinforcements for the beam are shown in Table , whereas the details of existing and required reinforcements in columns in shown in

Table 3: Required and existing reinforcements in beams

TABLE: Concrete Beam Flexure Envelope							Area Provided	
Label	Story	Section	As Top mm ²	As Bot mm ²	Area Existing		top bar mm ²	bottom bar mm ²
					top	bottom		
B6	Story2	B-230X375-M20	567	283	942	942	Not Required	Not required
B7	Story2	B-230X375-M20	527	263	942	942	Not Required	Not required
B8	Story2	B-230X375-M20	528	264	942	942	Not Required	Not required
B9	Story2	B-230X375-M20	567	284	942	942	Not Required	Not required
B6	Story1	B-230X375-M20	851	514	942	942	Not Required	Not required
B7	Story1	B-230X375-M20	902	612	942	942	Not Required	Not required
B8	Story1	B-230X375-M20	904	617	942	942	Not Required	Not required
B9	Story1	B-230X375-M20	851	514	942	942	Not Required	Not required

Table 4: Required and existing reinforcements in columns

Label	Story	Section	P	M Major	M Minor	Required steel(IS)		main bar existing	Reinforcement to be added(As')
			kN	kN-mm	kN-mm	%	sq. mm	area	
C2	Story2	C-430*550-M20	79.2834	-29.4869	-34.7113	0.80%	1188	2512	Not required
C3	Story2	C-430*550-M20	132.834	-2.881	-41.9717	0.80%	1188	2512	Not required
C6	Story2	C-430*550-M20	130.842	2.6168	-57.0385	0.80%	1188	2512	Not required
C8	Story2	C-430*550-M20	132.687	2.7588	-41.9376	0.80%	1188	2512	Not required
C10	Story2	C-430*550-M20	93.5105	22.2236	-66.0542	0.80%	1193	2512	Not required
C2	Story1	C-430*550-M20	152.175	-81.0547	-11.7887	1.24%	1839	2512	Not required
C3	Story1	C-430*550-M20	182.102	-83.7444	-6.5989	1.21%	1802	2512	Not required
C6	Story1	C-430*550-M20	219.029	82.4681	-8.6342	1.12%	1665	2512	Not required
C8	Story1	C-430*550-M20	182.017	83.733	-6.5953	1.21%	1802	2512	Not required
C10	Story1	C-430*550-M20	161.564	3.2313	-128.5347	1.33%	1982	2512	Not required

Note: The amount of concrete and steel according to IS 15988:2013 Cl.8.5.1.1 to account for losses:

$$Ac' = (3/2) Ac \text{ and } As' = (4/3) As$$

Where,

Ac and As = actual concrete and steel resp. to be provided in the jacket.

Ac' and As' = resp. concrete and steel values obtained for the jacket after deducting the existing concrete and steel from their respective required amount.

Hence minimum reinforcement (4-16 dia+ 4-12 dia) is added to the increased section of the column.

Similarly, spacing of ties to be provided in the jacket in order to avoid flexural shear failure of column and provide adequate confinement to the longitudinal steel along the jacket is given as:

$$s = (f_y / \sqrt{f_{ck}}) * (d_h^2 / t_j)$$

Where,

f_y = yield strength of steel

f_{ck} = cube strength of concrete

d_h = diameter of stirrups

t_j = thickness of jacket.

ANNEX B: CASE STUDY

B.1 ANALYSIS AND RETROFITTING DESIGN OF SDN 13 SYAMTALIRA ARUN

B.1.1 BUILDING DESCRIPTION

B.1.1.1 INTRODUCTION

This is a project under Save the Children project, with design and technical assistance from Syiah Kuala University. SDN 13 Syamtalira Arun is located at North Aceh. The school building consists of 2 rooms. The school has approximately 400 students. In general, the structural system before retrofitted was reinforced concrete frames with infill masonry walls.

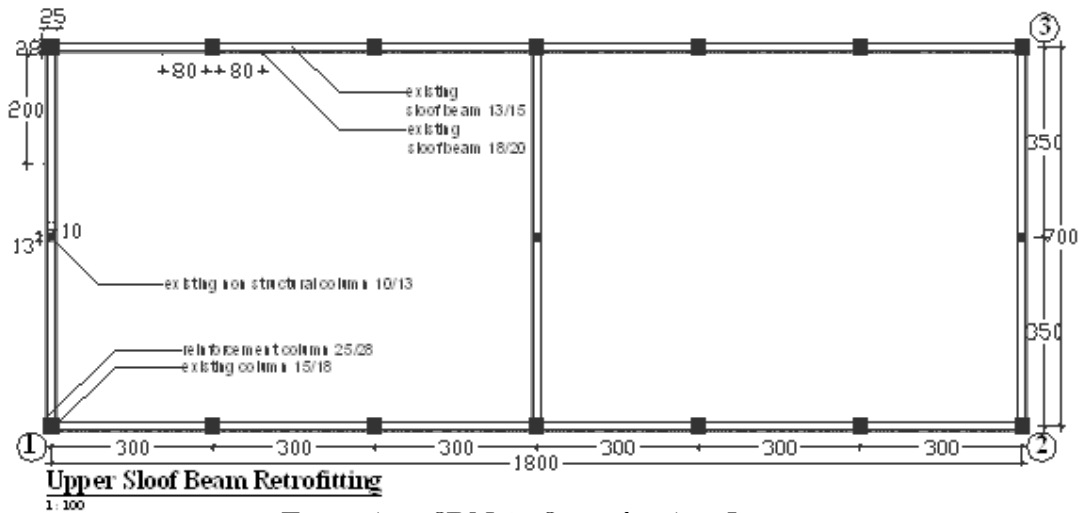


Figure A-30 SDN 13 Syamtalira Arun Layout

B.1.1.2 BACKGROUND

a. Status and Condition of Structure

From the initial survey, there were some major problems found in SDN 13 Syamtalira Arun, i.e:

1. Cracks on walls
2. Cracks on structural member
3. Poor workmanship
4. Poor quality construction

B.1.2 VULNERABILITY ASSESSMENT

(cracks observed, sizes of elements/walls and others)

a. Assessment

1) Visual Assessment

In the visual assessment, the following measures were conducted:

- Rapid visual inspection and assessment
- Collection of design and drawing
- Topographical information of site
- Site measurement of main structural member
- Inspection of cracks and location
- Judgment of the construction quality
- Evaluation of workmanship
- Inspection of material used and its quality



Figure A-31 Existing Condition of SDN 13 Syamtalira Arun





Figure A-32 Visual Assessment

2) Technical Assessment

Based on the results from the visual assessment, the technical assessment was conducted. In the technical assessment, some of the physical verification and partial/non-destructive tests were carried out, and the technical assessment measures included:

- Review and evaluation of design, specification & drawing
- Comparison of size, quality between design drawing and state of the structure in site
- Check with code provision, mainly size of main structural member, reinforcement bar

3) Results

Based on the assessment, the following problems were found:

- Defect on the design
- Not satisfied code requirement
- Not satisfied new Code requirement (new provision after tsunami)
- Insufficient size of Structural member
- Improper site for foundation in some case
- Poor quality of material - Not satisfied Specification
- Poor workmanship



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

Figure A-33 Technical Assessment

B.1.3. RETROFITTING DESIGN

a. Design Recommendation

Retrofitting strategy was decided based on the results of technical assessment. Due to the approach of open frame system (walls were not considered as lateral resisting elements), the retrofitting design required that structural element sizes (beams and columns) to be increased to provide larger load resistance capacity. Hence, the following design approaches were proposed:

1) Retrofitting on structural member

FOUNDATION

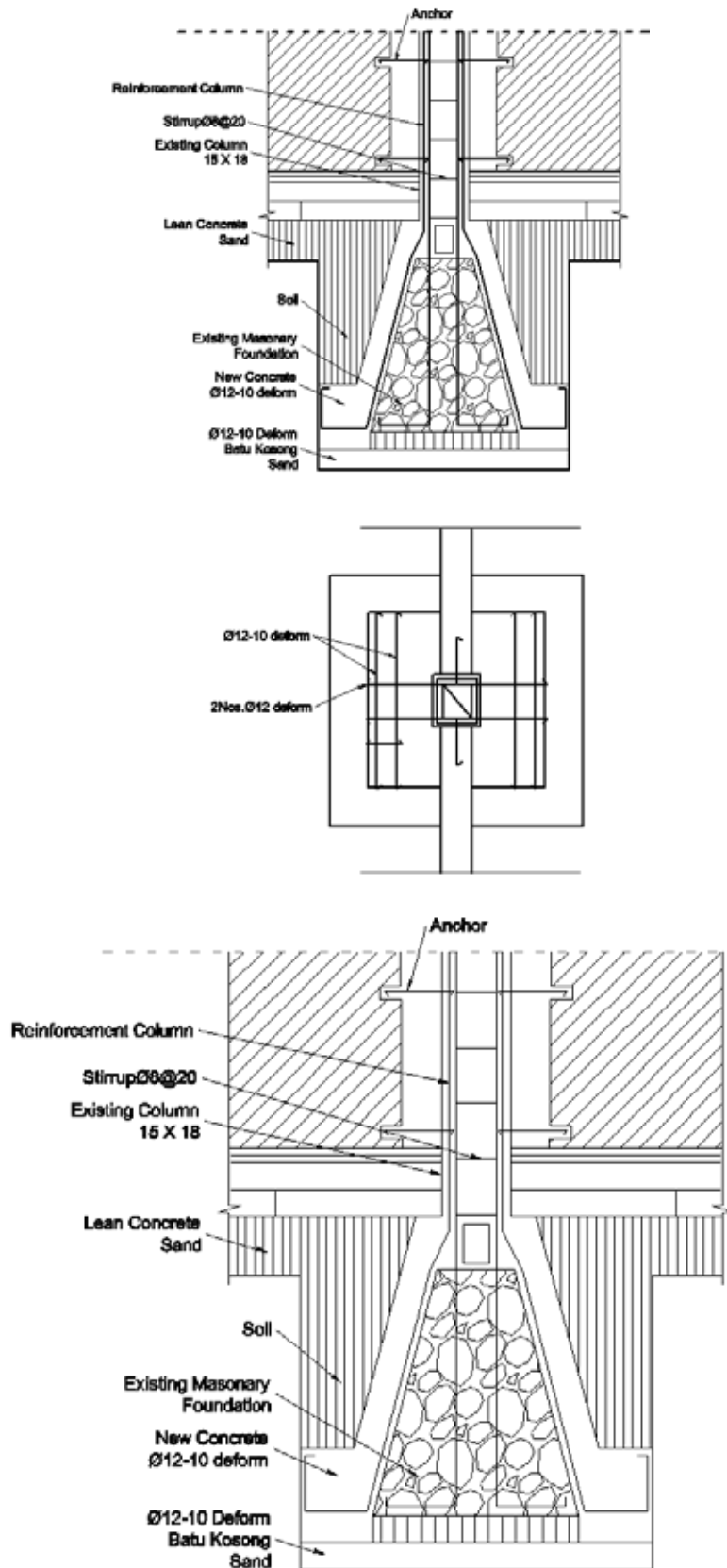
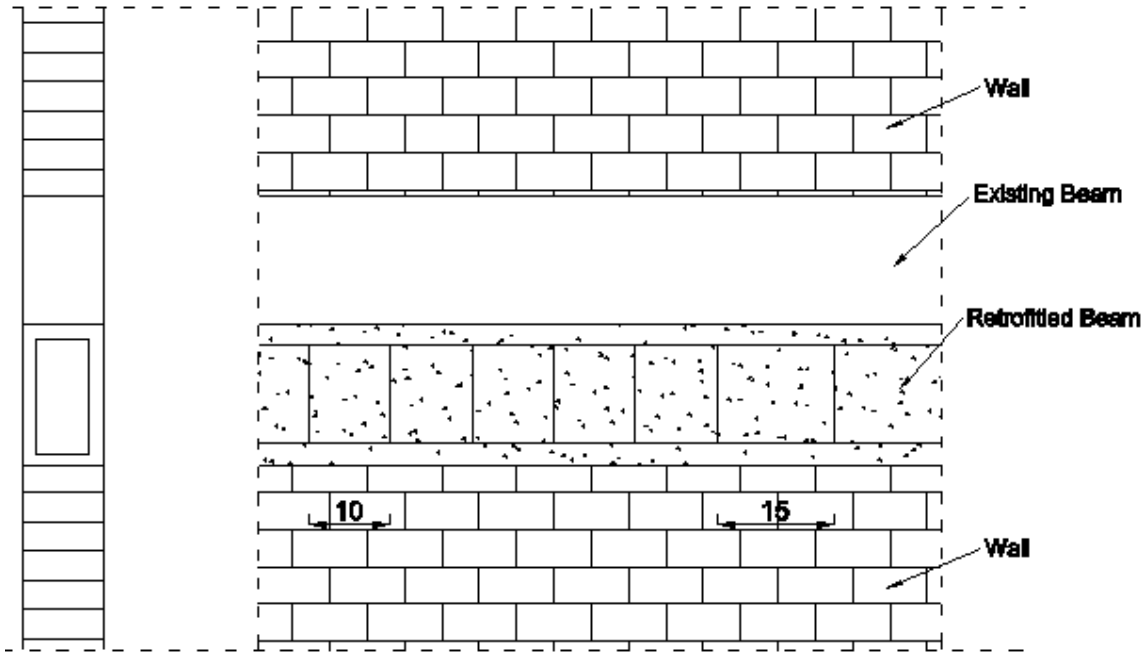


Figure A-34 Retrofitting of Foundation

BEAMS



Beam Retrofitting

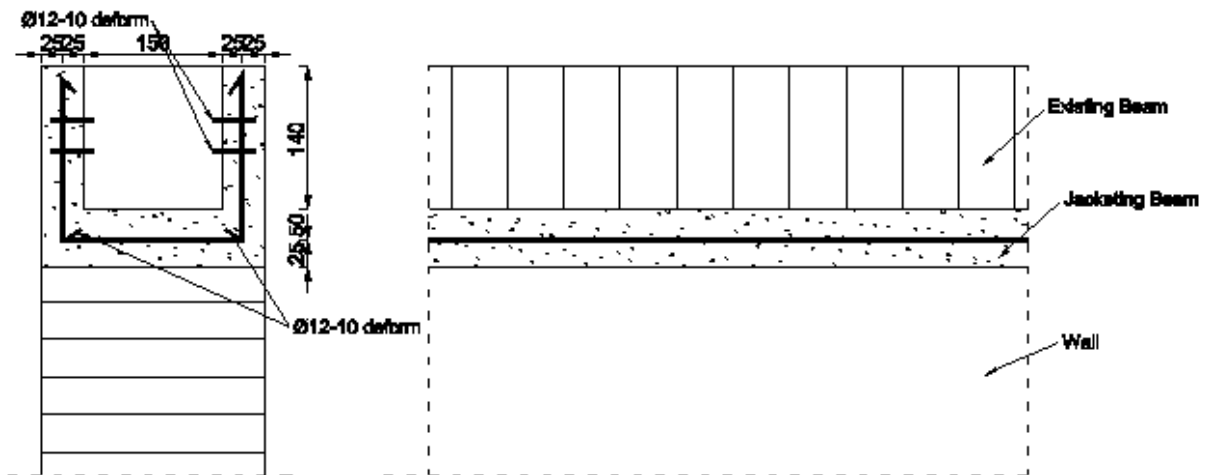


Figure A-35 Retrofitting of Beam

COLUMN

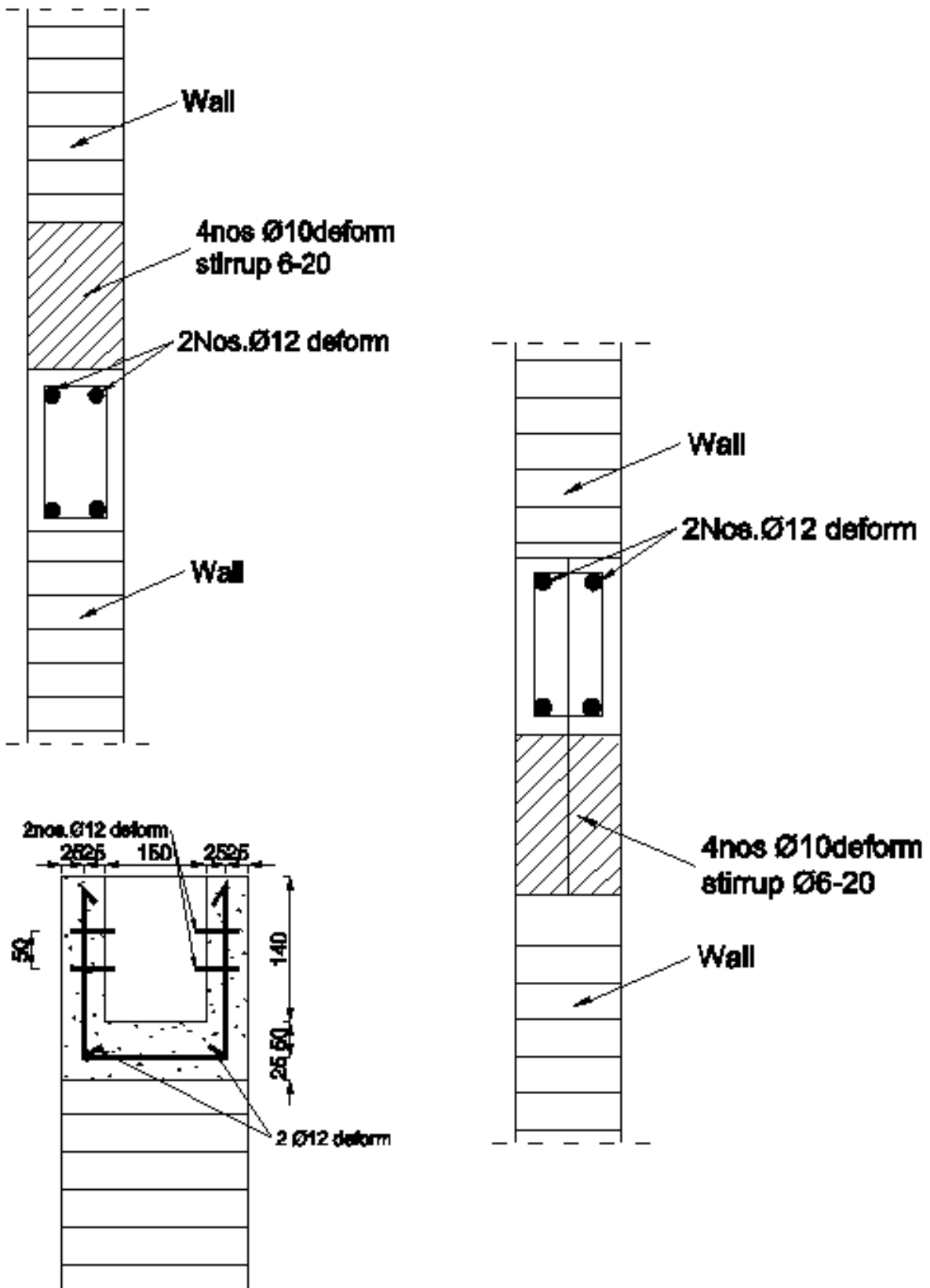


Figure A-36 Retrofitting of Beam Figure

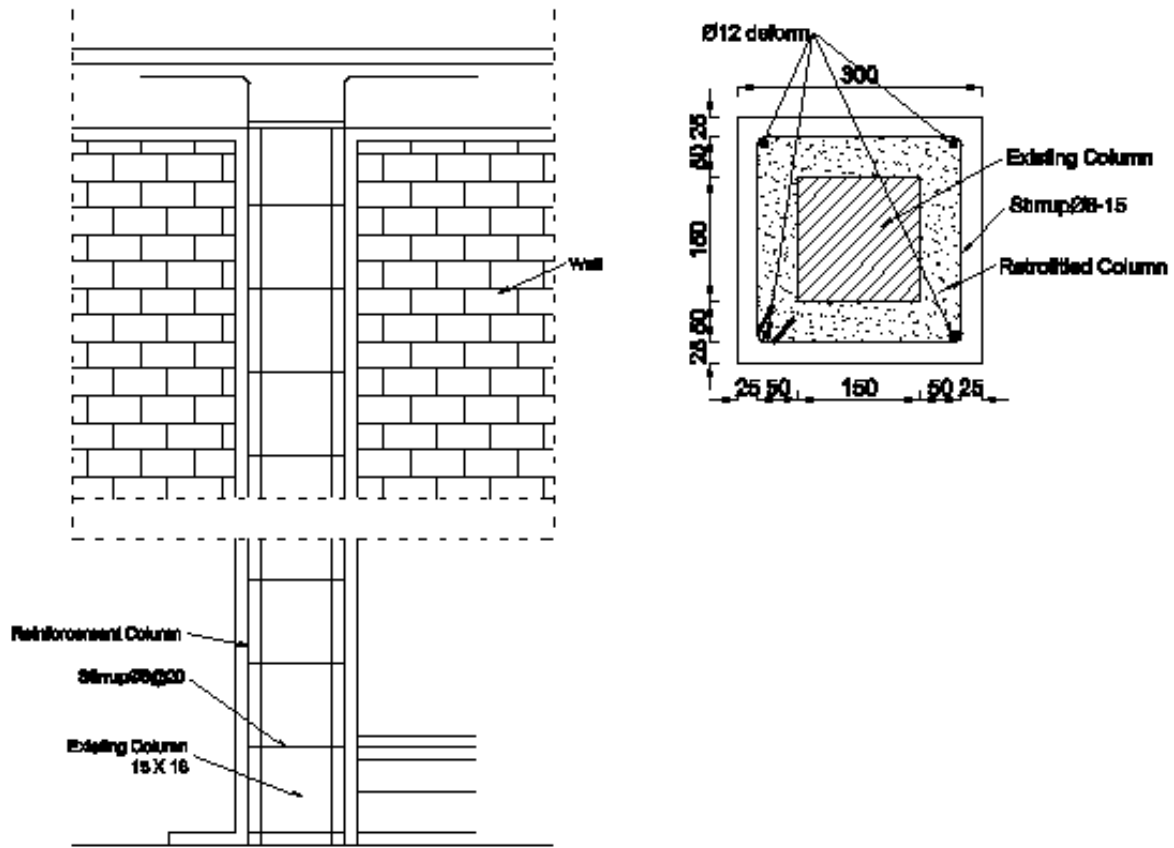


Figure A-37 Retrofitting of Column

2) Connection between Wall & Column

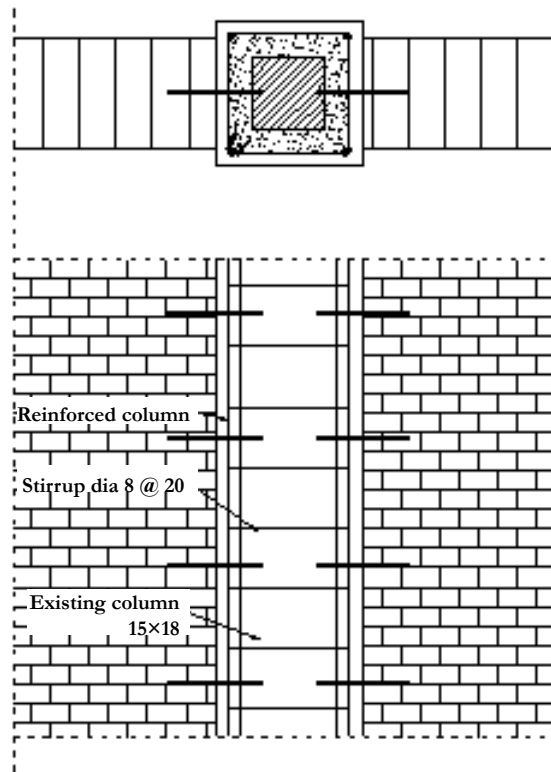


Figure A-38 Retrofitting between Wall and Column

- 3) Retaining structures to protect Foundation
- 4) Corrective measure on cracks

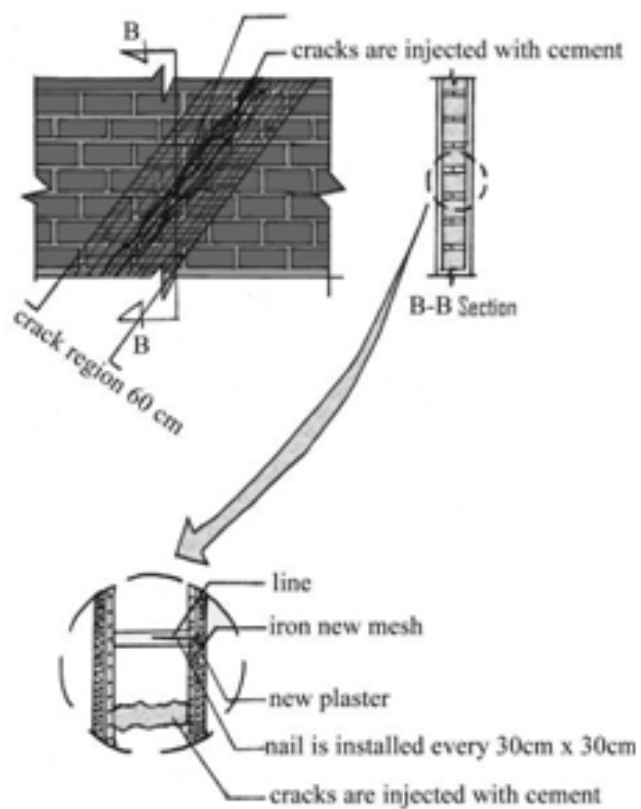


Figure A-39 Cracks Injection

b. Retrofitting Process



Figure A-40 Retrofitting of Foundation

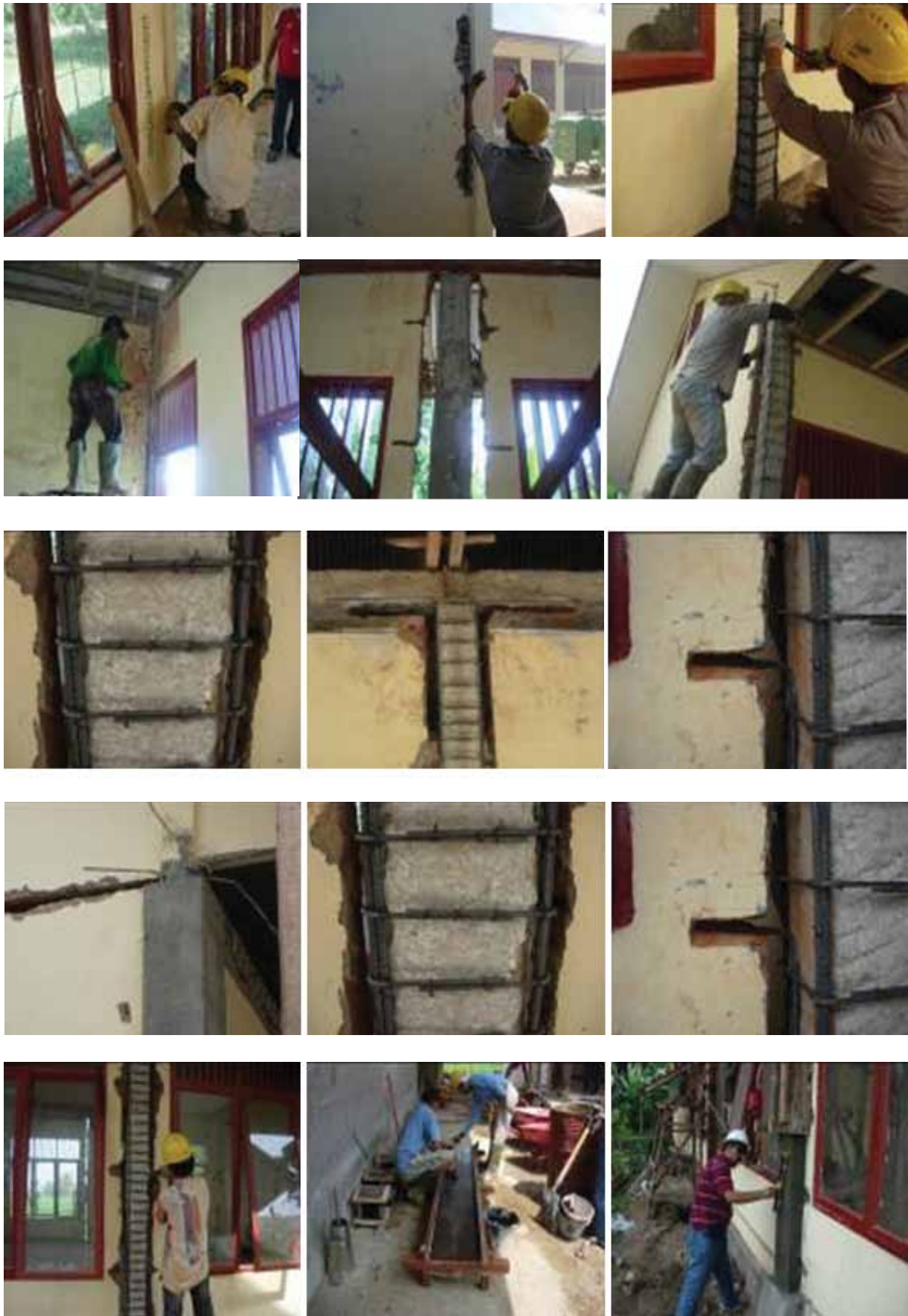


Figure A-41 Column Retrofitting



Figure A-42 Beam Retrofitting

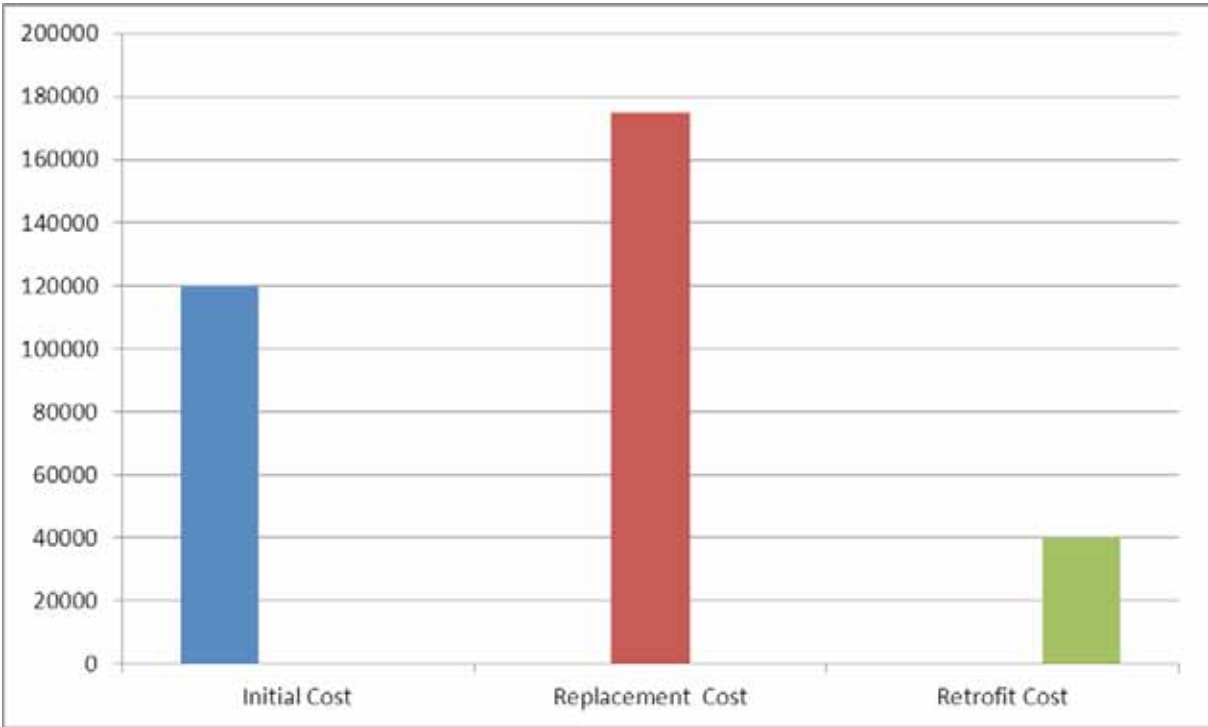
B.1.4. IMPLEMENTATION



Figure A-43 Retrofitted Structure

B.1.5. COST CALCULATION

Initial Construction Cost : US\$ 120,000
Replacement Cost : US\$ 175,000
Retrofit Cost : US\$ 40,000



ANNEX C:

CHECKING DIFFERENT VULNERABILITY FACTORS OF THE BUILDING

Each of the evaluation statements on this checklist shall be marked compliant (C), non-compliant (NC), or not applicable (N/A) or not known (NK) for a Tier 1 Evaluation. Compliant statements identify issues that are acceptable according to the criteria of FEMA 310, while non-compliant statements identify issues that require further investigation. Certain statements may not apply to the structures being evaluated. The evaluation of different statements is made and is noted by Underlined and Bold letter.

C.1. Building System

C NC N/A NK LOAD PATH: The structure shall contain one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation.

C NC N/A NK ADJACENT BUILDINGS: An adjacent building shall not be located next to the structure being evaluated closer than 4% of the height for Life Safety and Immediate Occupancy.

C NC N/A NK WEAK STORY: The strength of the lateral-force-resisting system in any story shall not be less than 80% of the strength in an adjacent story above or below for Life-Safety and Immediate Occupancy.

C NC N/A NK SOFT STORY: The stiffness of the lateral-force-resisting system in any story shall not be less than 70% of the stiffness in an adjacent story above or below or less than 80% of the average stiffness of the three stories above or below for Life-Safety and Immediate Occupancy.

C NC N/A NK GEOMETRY: There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30% in a story relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-story penthouses.

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

C NC N/A NK MASS: There shall be no change in effective mass more than 50% from one story to the next for Life Safety and Immediate Occupancy.

C NC N/A NK TORSION: The distance between the story center of mass and the story center of rigidity shall be less than 20% of the building width in either plan dimension for Life Safety and Immediate Occupancy.

Refer Annex 2 C: Check for torsion

C NC N/A NK DETERIORATION OF CONCRETE: There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force-resisting elements. No such deterioration observed

C.2. Lateral Force Resisting System

C NC N/A NK REDUNDANCY: The number of lines of moment frames in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. The number of bays of moment frames in each line shall be greater than or equal to 2 for Life Safety and 3 for Immediate Occupancy.

Meets the criteria

C ~~NC~~ N/A NK INTERFERING WALLS: All infill walls placed in moment frames shall be isolated from structural elements.

Infilled walls are attached to frames but not tied together

C NC N/A NK SHEAR STRESS CHECK: The shear stress in the concrete columns, calculated using the Quick Check procedure of Section 3.5.3.2, shall be less than 100 psi or $2\sqrt{f_c}$ for Life Safety and Immediate Occupancy.

Refer Annex 2A.3: Check for shear stress

C NC N/A NK AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces shall be less than $0.10f'_c$ for Life Safety and Immediate Occupancy. Alternatively, the axial stresses due to overturning forces alone, calculated using the Quick Check Procedure of Section 3.5.3.6, shall be less than $0.30f'_c$ for Life Safety and Immediate Occupancy.

Refer Annex 2B: Check for axial stress

C NC N/A NK FLAT SLAB FRAMES: The lateral-force-resisting system shall not be a frame consisting of columns and a flat slab/plate without beams.

Lateral force resisting system consists of columns and beams

C ~~NC~~ N/A NK SHORT CAPTIVE COLUMNS: There shall be no columns at a level with height/depth ratios less than 50% of the nominal height/depth ratio of the typical columns at that level for Life Safety and 75% for Immediate Occupancy.

Columns at the mid-landing of the staircases do not satisfy this criteria

C ~~NC~~ N/A NK NO SHEAR FAILURE: The shear capacity of frame members shall be able to develop the moment capacity at the top and bottom of the columns

Refer Annex 2A 4.2

C ~~NC~~ N/A NK STRONG COLUMN / WEAK BEAM: The sum of the moment capacity of the columns shall be 20% greater than that of the beams at frame joints.

Refer Annex 2D: Check for strong column weak beam

C NC N/A NK BEAM BARS: At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam. At least 25% of the longitudinal bars provided at the joints for either positive or negative moment shall be continuous throughout the length of the members for Life Safety and Immediate Occupancy.

Reference from structural drawing

C ~~NC~~ N/A NK COLUMN-BAR SPLICES: All columns bar lap splice lengths shall be greater than $35 d_b$ for Life Safety and $50 d_b$ for Immediate Occupancy and shall be enclosed by ties spaced at or less than $8 d_b$ for Life Safety and Immediate Occupancy.

C NC N/A NK BEAM-BAR SPLICES: The lap splices for longitudinal beam reinforcing shall not be located within $l_b / 4$ of the joints and shall not be located within the vicinity of potential plastic hinge locations.

C NC N/A NK COLUMN-TIE SPACING: Frame columns shall have ties spaced at or less than $d/4$ for Life Safety and Immediate Occupancy throughout their length and at or less than $8 d_b$ for Life Safety and Immediate Occupancy at all potential plastic hinge locations.

C NC N/A NK STIRRUP SPACING: All beams shall have stirrups spaced at or less than $d/2$ for Life Safety and Immediate Occupancy throughout their length. At potential plastic hinge locations stirrups shall be spaced at or less than the minimum of $8 d_b$ or $d/4$ but no less than 100mm for Life Safety and Immediate Occupancy.

As per structural drawing

C NC N/A NK JOINT REINFORCING: Beam-column joints shall have ties spaced at or less than $8d_b$ for Life Safety and Immediate Occupancy.

C NC N/A NK JOINT ECCENTRICITY: There shall be no eccentricities larger than 20% of the smallest column plan dimension between girder and column centerlines. This statement shall apply to the Immediate Occupancy Performance Level only.

C NC N/A NK STIRRUP AND TIE HOOKS: The beam stirrups and column ties shall be anchored into the member cores with hooks of 135° or more. This statement shall apply to the Immediate Occupancy Performance Level only.

As per structural drawing

C.3. Diaphragms

C NC N/A NK DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors. In wood buildings, the diaphragms shall not have expansion joints.

C NC N/A NK PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only.

CNC N/A NK DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing bars around all diaphragm openings larger than 50% of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only.

C.4. Connections

C NC N/A NK CONCRETE COLUMNS: All concrete columns shall be doweled into the foundation for Life Safety and the dowels shall be able to develop the tensile capacity of the column for Immediate Occupancy.

ANNEX D: MODIFIED MERCALLY INTENSITY SCALE (MMI Scale)

INTENSITY	DESCRIPTION OF EFFECT
I	<p>Very Weak Intensity</p> <ul style="list-style-type: none"> - Can only be noticed or felt by people who are in the right situation and circumstance - Furniture 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	<p>Slightly Weak Intensity</p> <ul style="list-style-type: none"> - Can be noticed or felt by people who are resting inside homes - Things that are hung on walls would slightly sway, shake or vibrate - The shaking or vibrations on water or liquid surfaces in containers would be highly noticeable
III	<p>Weak Intensity</p> <ul style="list-style-type: none"> - 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 has passed nearby - Things that are hung 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	<p>Slightly Strong Intensity</p> <ul style="list-style-type: none"> - 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 has passed nearby - Things that are hung 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	<p>Strong Intensity</p> <ul style="list-style-type: none"> - 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 hung on walls would sway, shake or vibrate much more strongly and intensely. Plates and glasses would also vibrate and shake much more 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 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	<p>Very Strong Intensity</p> <ul style="list-style-type: none"> - 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 flat tyres. - 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 vibration of plant or tree stem, branches and leaves would be strong and highly noticeable.

INTENSITY	DESCRIPTION OF EFFECT
VII	<p>Damaging Intensity</p> <ul style="list-style-type: none"> - 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 building 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	<p>Highly Damaging Intensity</p> <ul style="list-style-type: none"> - 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 may 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 opposition. 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 more and sway in all directions.
IX	<p>Destructive Intensity</p> <ul style="list-style-type: none"> - 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 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	<p>Extremely Destructive Intensity</p> <ul style="list-style-type: none"> - Overall extreme destruction and damage of all man-made structures - Widespread landslides, liquefaction, 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.
XI	<p>Devastative Intensity</p> <ul style="list-style-type: none"> • Severe damage even to well built buildings, bridges, water dams and railway lines; highways become useless; underground pipes destroyed.
XII	<p>Extremely Destructive Intensity (Landscape changes)</p> <ul style="list-style-type: none"> • Practically all structures above and below ground are greatly damaged or destroyed.

ANNEX E: EUROPEAN MACRO SEISMIC SCALE (EMS 98)

Classifications used in the European Macro seismic Scale (EMS)

*Differentiation of structures (buildings) into vulnerability classes
(Vulnerability Table)*

Type of Structure		Vulnerability Class					
		A	B	C	D	E	F
MASONRY	rubble stone, fieldstone	○					
	adobe (earth brick)	○	—				
	simple stone	—	○				
	massive stone		—	○	—		
	unreinforced, with manufactured stone units	—	○	—			
	unreinforced, with RC floors reinforced or confined		—	○	—		
REINFORCED CONCRETE (RC)	frame without earthquake-resistant design (ERD)	—		○	—		
	frame with moderate level of ERD		—	○	—		
	frame with high level of ERD			—	○	—	
	walls without ERD		—	○	—		
	walls with moderate level of ERD			—	○	—	
	walls with high level of ERD				—	○	—
STEEL	steel structures			—	○	—	
WOOD	timber structures		—	○	—		

○ most likely vulnerability class; — probable range;
.....range of less probable, exceptional cases

ANNEX F: EQUIPMENT USED IN RETROFITTING

Some of the major equipment used in retrofitting are:

1. Schmidt hammer

A Schmidt hammer, also known as a Swiss hammer or a rebound hammer, is a device to measure the elastic properties or strength of concrete or rock, mainly surface hardness and penetration resistance.

It was invented by Ernst Schmidt, a Swiss engineer.

The hammer measures the rebound of a spring-loaded mass impacting against the surface of the sample. The test hammer will hit the concrete at a defined energy. Its rebound is dependent on the hardness of the concrete and is measured by the test equipment. By reference to the conversion chart, the rebound value can be used to determine the compressive strength. When conducting the test the hammer should be held at right angles to the surface which in turn should be flat and smooth. The rebound reading will be affected by the orientation of the hammer, when used in a vertical position (on the underside of a suspended slab for example) gravity will increase the rebound distance of the mass and vice versa for a test conducted on a floor slab. The Schmidt hammer is an arbitrary scale ranging from 10 to 100. Schmidt hammers are available from their original manufacturers in several different energy ranges. These include: (i) Type L-0.735 Nm impact energy, (ii) Type N-2.207 Nm impact energy; and (iii) Type M-29.43 Nm impact energy.

The test is also sensitive to other factors:

- Local variation in the sample. To minimize this it is recommended to take a selection of readings and take an average value.
- Water content of the sample, a saturated material will give different results from a dry one.

Prior to testing, the Schmidt hammer should be calibrated using a calibration test anvil supplied by the manufacturer for that purpose. 12 readings should be taken, dropping the highest and lowest, and then take the average of the ten remaining. Using this method of testing is classed as indirect as it does not give a direct measurement of the strength of the material. It simply gives an indication based on surface properties; it is only suitable for making comparisons between samples.

2. Bar Scanner

To determine the position, depth and diameter of rebar can be problematic in everyday construction work. The bar scanner was developed is a portable, quick and simple-to-operate system that solves all these problems and many more:

- Finding a secure drilling point for drilling or coring work
- Carrying out structural analyses quickly and exactly in a non-destructive manner
- Determining coverage over the entire surface of a structure

3. Grouting Machine

Pressure grouting involves injecting a grout material into generally isolated pore or void space of which neither the configuration nor volume are known, and is often referred to simply as grouting. The grout may be a cementitious, resinous, or solution chemical mixture. The greatest use of pressure grouting is to improve geomaterials (soil and rock). The purpose of grouting can be either to strengthen or reduce water flow through a formation. It is also used to correct faults in concrete and masonry structures. Since first usage in the 19th century, grouting has been performed on the foundation of virtually every one of the world's large dams, in order to reduce the amount of

leakage through the rock, and sometimes to strengthen the foundation to support the weight of the overlying structure, be it of concrete, earth, or rock fill. Although very specialized, pressure grouting is an essential construction procedure that is practiced by specialist contractors and engineers around the world.

4. Drilling Machine

A drill is a tool fitted with a cutting tool attachment or driving tool attachment, usually a drill bit or driver bit, used for boring holes in various materials or fastening various materials together with the use of fasteners. The attachment is gripped by a chuck at one end of the drill and rotated while pressed against the target material. The tip, and sometimes edges, of the cutting tool does the work of cutting into the target material. This may be slicing off thin shavings (twist drills or auger bits), grinding off small particles (oil drilling), crushing and removing pieces of the workpiece (SDS masonry drill), counter sinking, counter boring, or other operations.

Drills are commonly used in woodworking, metalworking, construction and do-it-yourself projects. Specially designed drills are also used in medicine, space missions and other applications. Drills are available with a wide variety of performance characteristics, such as power and capacity.

ANNEX G: OVERVIEW OF SOME DAMAGED RC BUILDINGS AND ITS CAUSE



Strong beam weak column



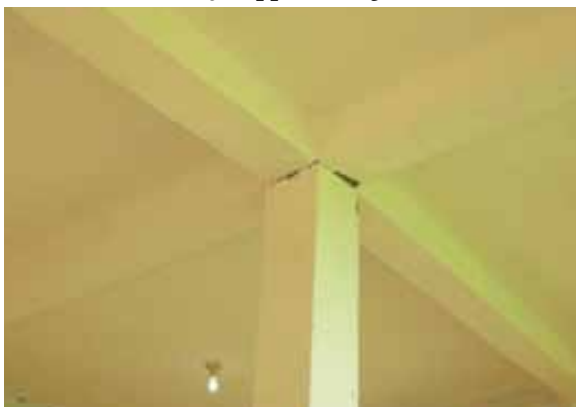
Weak Beam- column Joint



Soft Upper-Storey



Soft Ground Storey



Weak Beam - Column Joint



Unequal settlement



Due to confinement



Weak storey

Figure A-44 Photos of different Damage patterns (Source: Dr. Hari Darshan Shrestha)

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SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL

RCC STRUCTURES

