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.

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.

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FOREWORD

Nepal is located between two active tectonic plates, Indian and Tibetan, where the Indian plate is sub-ducting 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 Julliand
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
fa	Axial compressive stress due to gravity
Fp	Design force for anchorage of walls to diaphragms
FRC	Fiber reinforced composite
fy	Lower bound yield strength of shear reinforcement
fye	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
Ke	Elastic lateral stiffness
Ki	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
Pw	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
Sa	Response spectrum acceleration
SXS	Spectral response acceleration for short period
t	Thickness of wall
Te	Effective fundamental period
Ti	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 RETROFITTING[1]

1.4.1 REPAIR

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

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

[1] *Adapted from IAEE 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 Categories[2]

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

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

2.2 DAMAGE TYPOLOGIES[3]

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

2.2.1 SHEAR/ DIAGONAL CRACKS

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

The severity of in-plane cracks 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.

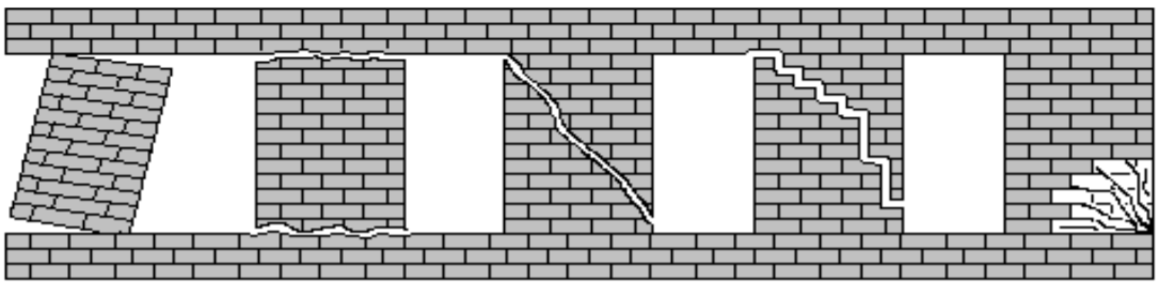
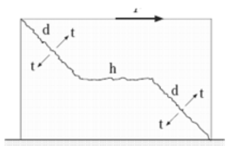
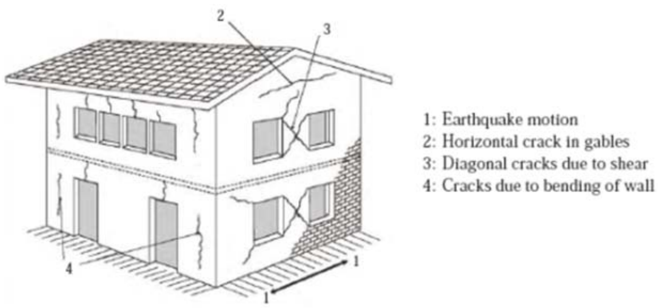


Figure 2-1 Different crack patterns of un-reinforced masonry pier (Source Yi, 2004)





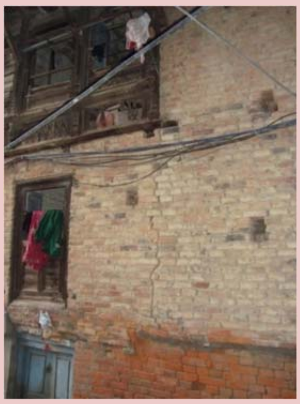
[3] *Adapted from Planning and Engineering Guidelines for the Seismic Retrofitting of Historic Adobe Structures, E. Leroy Tolles, Edna E. Kimbro, William S. Ginell*

Cracks and Damages	Remark
 *Damage at corners of openings, Indonesia*	
  *Heavily damaged single storey rubble masonry wall with concrete roof in Manukawa & Sukhpur, India Walls survived due to diaphragm action from roof. Cantilever beams embedded in walls also helped this.* *Note: Window openings are also not close to corners.*	*Ref: Repair and strengthening guide for earthquake damaged low rise domestic buildings in Gujarat, India*
  *1. Large window openings close to corners and short column failures* *2. Diagonal cracking at corner column caused by twisting of frame and short column failure.*	The picture also shows cracks at corner along with diagonal cracks. *Ref: Repair and strengthening guide for earthquake damaged low rise domestic buildings in Gujarat, India*
 *Infill panels to an reinforced concrete frame building acting as non-structural shear walls, provided stability to the overall frame – Bharasar, Gujrat*	*Ref: Repair and strengthening guide for earthquake damaged low rise domestic buildings in Gujarat, India*

Cracks and Damages	Remark
 *Uri town – Civil Defense Bldg.– Diagonal cracks in window openings and tilting of walls in Burnt Brick Cement Masonry wall, Kashmir, India.*	*Ref: Repair, Restoration and retrofitting of Masonry Buildings in EQ affected Areas of Jammu & Kashmir*
 *Shear crack in a building during Haiti earthquake* *Photo: Er. Hari Darshan Shrestha*	
  *Pakistan earthquake* *Photo: Er. Hari Darshan Shrestha*	
 *Diagonal crack seen in a local building in Kathmandu, Nepal* *Photo: Er. Subin Desar*	

2.2.2 VERTICAL CRACKS

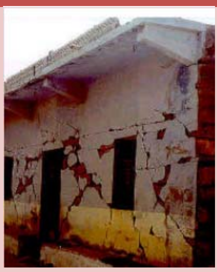
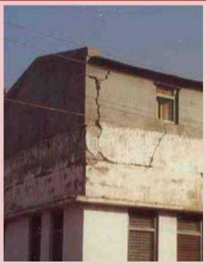
Vertical cracks often develop at corners during the interaction of perpendicular walls and are caused by flexure and tension due to out-of-plane movements. This type of damage can be particularly severe when vertical cracks occur on both faces, allowing collapse of the wall section at the corner.

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

Cracks and Damages	Remark
 *Side wall of a building damaged during Gorkha Earthquake, 2001*	

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
   *Modern cut-stone masonry building in Mirzapur, Gujrat, India*	*Ref: 2. Repair and strengthening guide for earthquake damaged lowrisedomestic buildings in Gujarat, India*
 *Uri town – Civil Defence Bldg.– Corner cracks and vertical cracks in Burnt Brick Cement Masonry wall, India*	*Ref: Repair, Restoration and retrofitting of Masonry Buildings in EQ affected Areas of Jammu & Kashmir*

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

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.

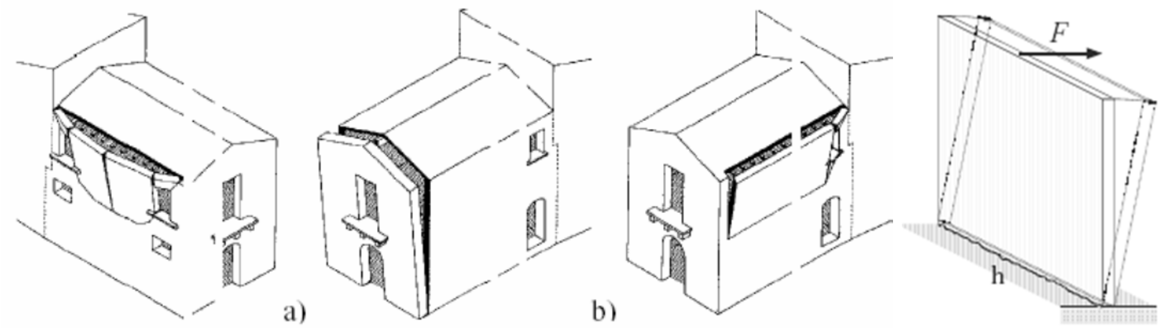


Figure 2-2 In-plane stiffness of wooden floor

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

Damages and Cracks	Remark
  	*Ref: In-plane stiffness of wooden floor*
  *Out of plane damage during Haiti earthquake* *(Photo: Hari Darshan Shrestha)*	
  *Out of plane damage during Haiti earthquake* *(Photo: Hari Darshan Shrestha)*	
  *(Photo: Subin Desar)*	

Damages and Cracks	Remark
  *Damages during Gorkha Earthquake, 2001*	

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
  *An example of bed joint can be seen in following pictures of buildings devastated after Haiti earthquake.* *(Photo: Er. Hari Darshan Shrestha)*	

2.2.6 TOE CRUSHING

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

Damages and Cracks	REMARK
 *Toe crushing defect can be seen in the picture beside, taken after Haiti earthquake.* *(Photo: Er. Hari Darshan Shrestha)*	
 *After an earthquake in Pakistan, a building can be seen with cracks and major defects at the toe.* *(Photo: Er. Hari Darshan Shrestha)*	

3. VULNERABILITY ASSESSMENT OF EXISTING BUILDINGS

Vulnerability assessment of buildings can be performed according to methodology prescribed in "SEISMIC VULNERABILITY EVALUATION GUIDELINE FOR PRIVATE AND PUBLIC BUILDINGS" developed by Ministry of Urban Development and Building Construction under Earthquake Risk Reduction and Recovery Preparedness Program for Nepal. For detailed Evaluation please refer Chapter 4 (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 structures[4]. These matters should receive careful consideration during the planning stage of retrofitting of masonry structures.

4.1.1 BUILDING LIMITATION

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

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

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

4.1.2 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

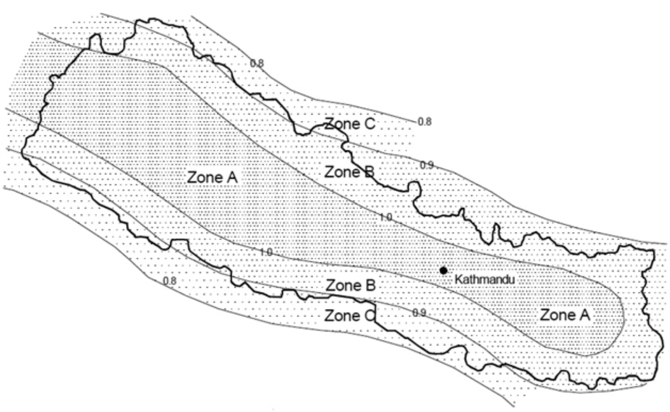


Figure 4-1 Seismic zones for masonry structures (NBC 109)

[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.

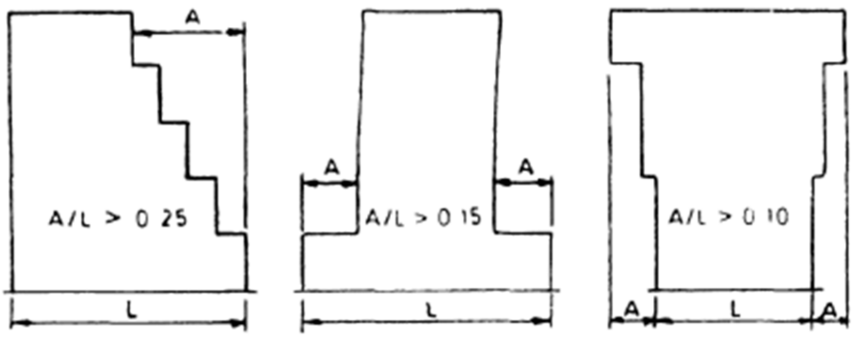


Figure 4-2 Vertical irregularities limit up to which treatment at discontinuity point or at element level is possible.

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.

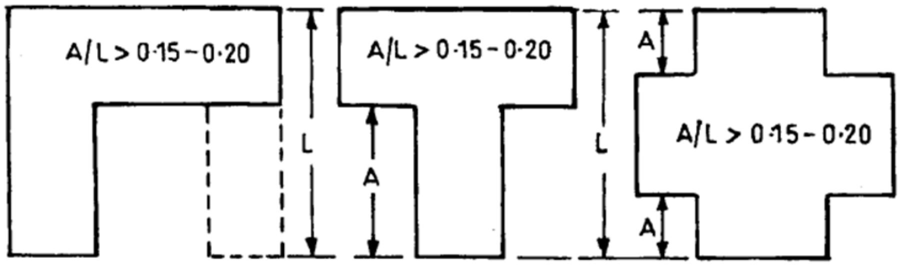


Figure 4-3Plan irregularity limits that require strengthening of overstressed elements and re-entrant corners

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.

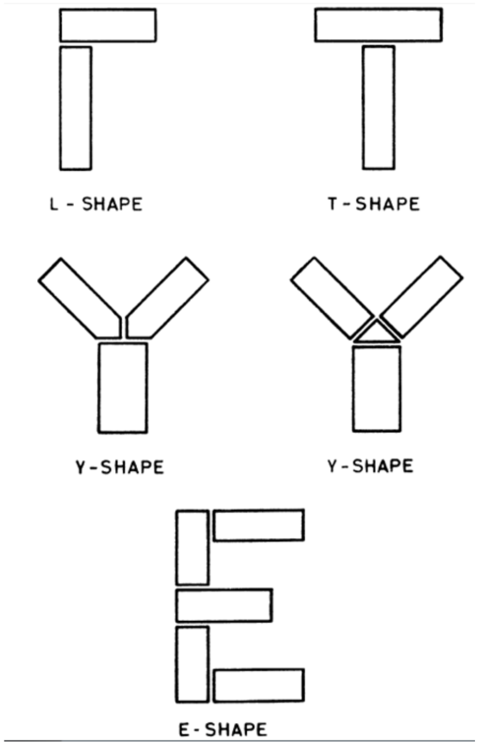
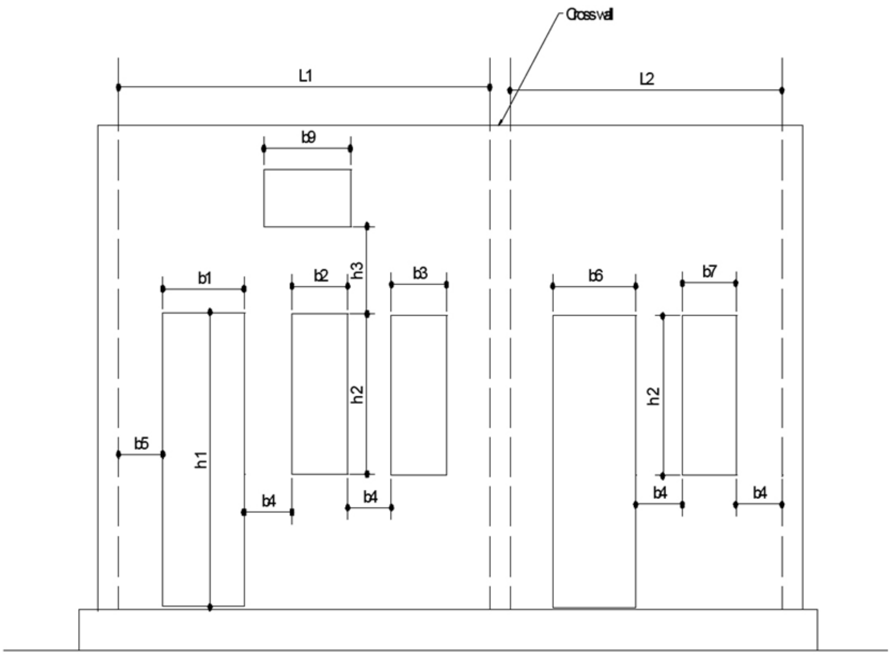


Figure 4-4 Recommended shapes for buildings with irregular plans

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 b2 and b9)

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.

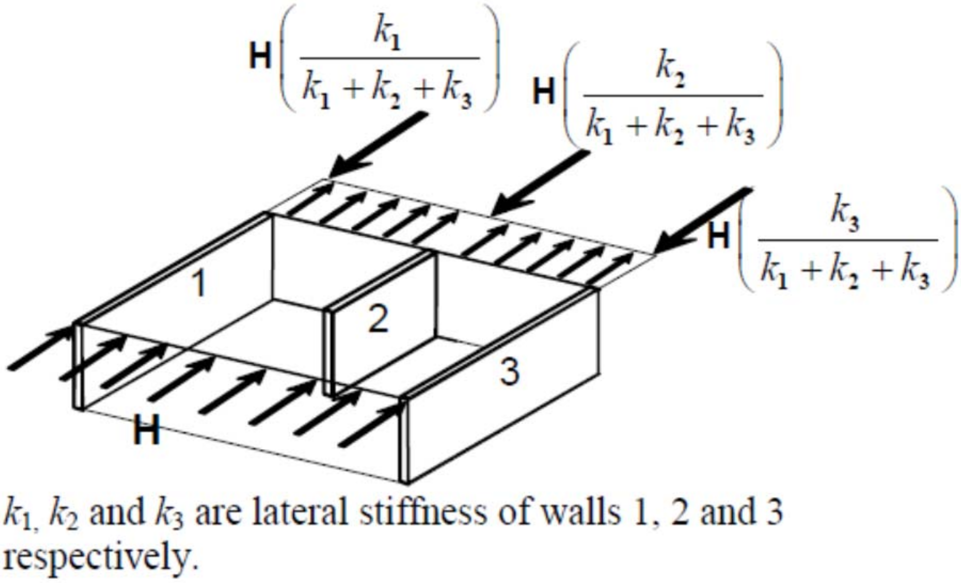


Figure 5-1 Proportional distribution of horizontal storey level force to individual walls for rigid diaphragm at floor level

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.

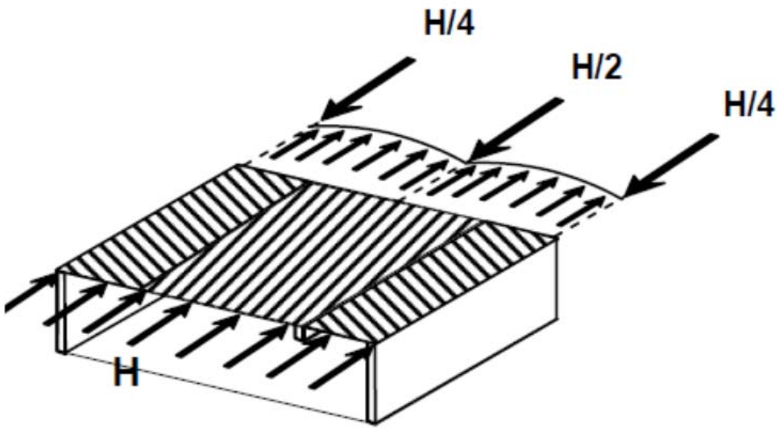


Figure 5-2 Proportional distribution of horizontal storey level force to individual walls for flexible diaphragm at floor level

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:

$V_a = v_a Dt$

where:

D = In plane length of masonry wall (mm)

t = thickness of wall (mm)

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

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

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 (mm^2)

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/mm^2 when in place shear test data is not available

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

For walls without openings:

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

For wall with openings:

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

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 4-2[5].

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

For walls with openings

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

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):

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

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):

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

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

$\mathbf{C_m}$ = 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):

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

Where:

K = Knowledge factor

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

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

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

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

Where:

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

Q_{CE} = Expected strength of component

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

The "m" factor is obtained from FEMA 356and 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 $\mathbf{T}_{sor}$ 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).

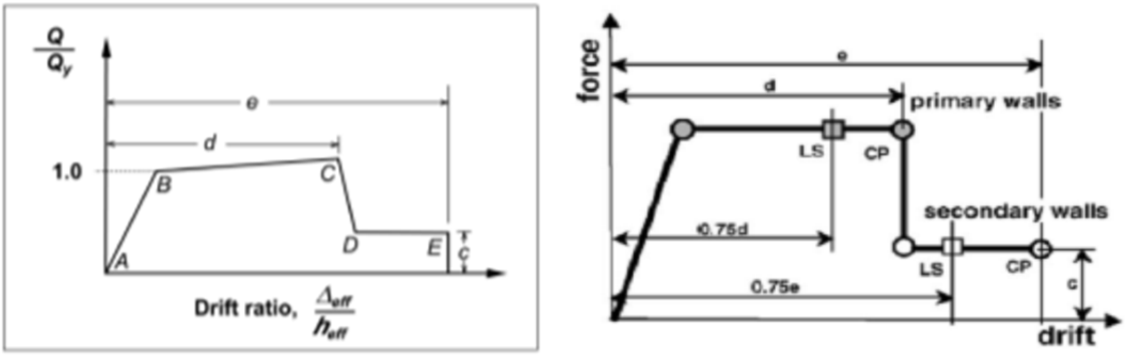


Figure 6- 1: Generalized Force-Deformation Relationship for Deformation Controlled Masonry Elements or Components (FEMA 356-2000)

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):

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

Where:

T_i = Elastic fundamental period calculated by elastic dynamic analysis

K_i = Elastic lateral stiffness

K_e = Effective lateral stiffness

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

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

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	0.8h_{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 WALLS[7]

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 below[8].

Property	Masonry Condition[1]		
	Good	Fair	Poor
Compressive Strength ($f^{1}m$)	900 psi	600 psi	300 psi
Elastic Modulus in Compression	$500f^{1}m$	$500f^{1}m$	$500f^{1}m$
Flexural Tensile Strength2	20 psi	10 psi	0
Shear Strength3			
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 (fme)	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 550fme where fme 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 $= \frac{1}{3}$ (C7-1)

$= \frac{h_{eff}}{3E_mI_g} + \frac{h_{eff}}{A_vG_m}$

where:

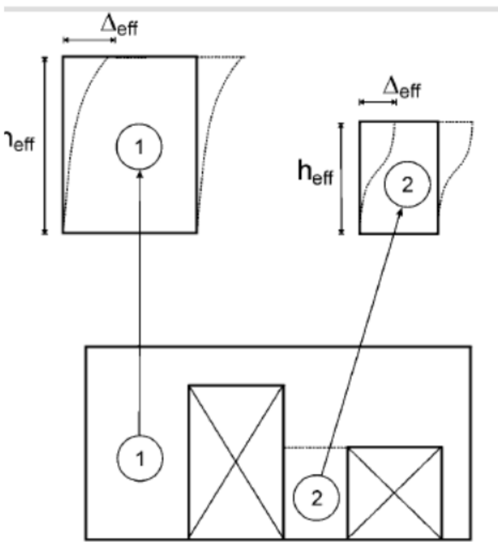
h_{eff} = wall height
A_v = Shear area
I_g = Moment of inertia for the gross section representing uncracked behacior
E_m = Masonry elastic modulus
G_m = Masonry shear modulus

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 $= \frac{1}{3}$ (C7-2)

$= \frac{h_{eff}}{12E_mI_g} + \frac{h_{eff}}{A_vG_m}$

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



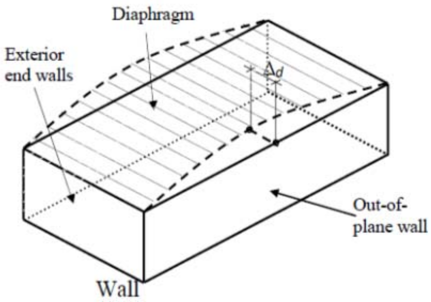
h_{eff} = The effective height of the component under consideration.
A = The differential displacement between the top and bottom of the component.

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

6.1.8 DEFLECTION CHECK OF DIAPHRAGM[9]

(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 wic flanged I beam with a large depth. The web of the I bean resists shear while the flange contributes resisting moment generated by the uniform later load.

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

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

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 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).

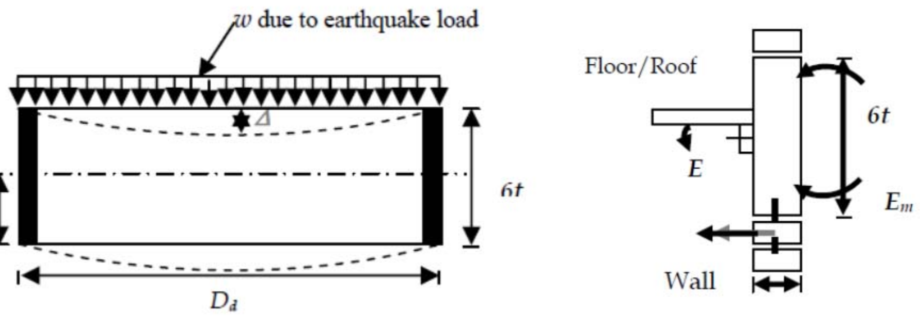


Figure C34(c) Diaphragm Deflection-I beam approximation

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)*

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

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

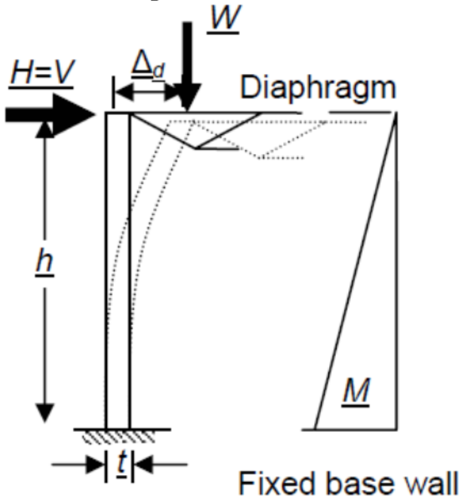
Where E_d and E_m are modulus of elasticity of diaphragm and masonry materials respectively.

(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.

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

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

Hence,

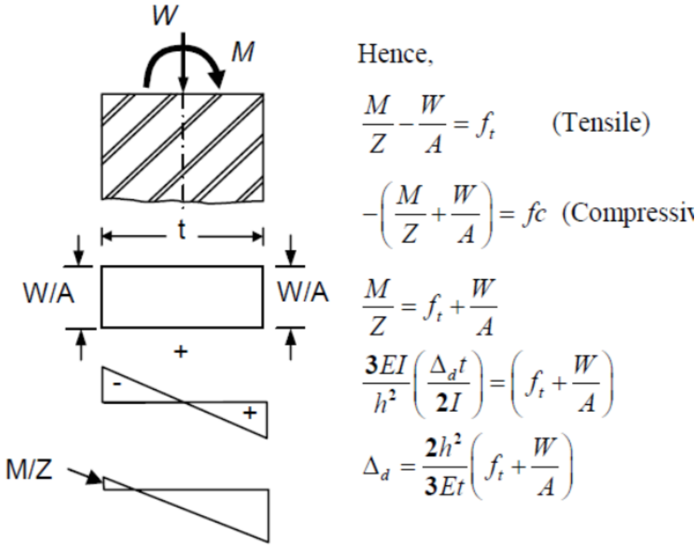
$$\frac{M}{Z} - \frac{W}{A} = f_t \qquad \text{(Tensile)}$$

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

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

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

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



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

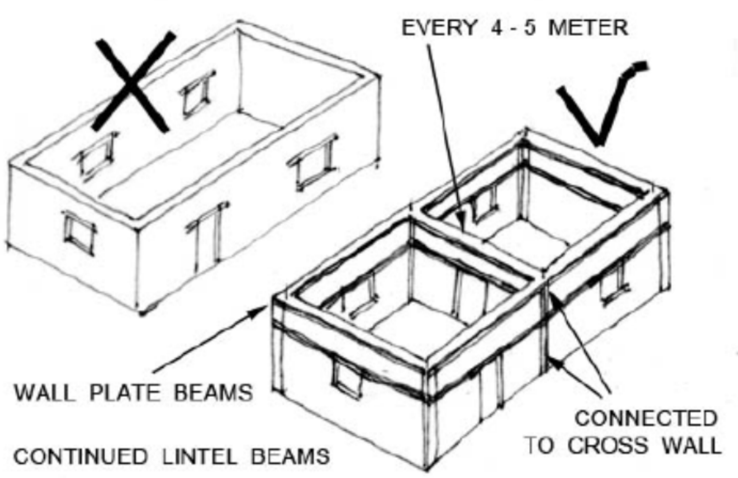


Figure 6- 2: Addition of cross-wall (Advies, 2012)

7 ALTERNATIVE APPROACH FOR ANALYSIS

7.1 PERFORMANCE BASED BEHAVIOR OF MASONRY[10]

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)

Higher performance
less loss

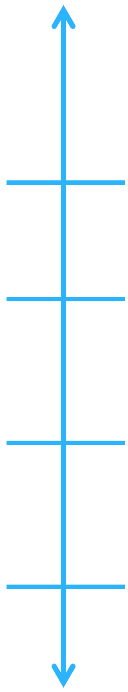
Expected Post-Earthquake Damage State

Operational (1-A)
Backup utility services maintain functions; very little damage (S1+NA)

Immediate Occupancy (1-B)
The building remains safe to occupy, any repairs are minor. (S1+NB)

Life Safety (3-C)
Structure remains stable and has significant reserve capacity, hazardous nonstructural damage is controlled. (S3+NC)

Collapse Prevention (5-E)
The building remains standing, but only barely; any other damage or loss is acceptable. (S5+NE)



Lower performance
more loss

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- 1shows 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):

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

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):

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

where,

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

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

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

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

7.2.1.2 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:

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

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

where,

A_n	=	Area of net mortared/grouted section
h_{eff}	=	Height to resultant of lateral force
L	=	Length of wall or pier
P_E	=	Expected axial compressive force due to gravity loads
v_{me}	=	Expected bed-joint sliding shear strength
V_{bjs}	=	Expected shear strength of wall or pier based on bed-joint sliding shear Strength
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).

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

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

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

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

7.2.2 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.

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

where,

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

Exceptions:

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

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

Structural Performance Level	X[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).

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

where,

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

i. Stiffness

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

ii. Strength

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

1. Test samples shall be extracted from an existing wall and subjected to minor-axis bending using the 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	$S_{x1} \leq 0.24g$	$0.24g < S_{x1} \leq 0.37$ g	$S_{x1} > 0.37g$
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 Ig shall be permitted. In either case, veneer wythes shall not be considered in the calculation of wall component properties. Stiffnesses for existing and new walls shall be assumed to be the same.

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

ii. Strength

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

iii. Flexural Strength of Walls and Piers

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

1. Stress in reinforcement below the expected yield strength, f_{ye}, shall be taken as the expected modulus of elasticity, E_{se}, times the steel strain. For reinforcement strains larger than those corresponding to the expected yield strength, the stress in the reinforcement shall be considered independent of strain and equal to the expected yield strength, f_{ye}.
2. Tensile strength of masonry shall be neglected when calculating the flexural strength of a reinforced masonry cross-section.
3. Flexural compressive stress in masonry shall be assumed to be distributed across an equivalent
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/Vdv, interpolation shall be used.

For M/Vdvless than 0.25:

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

For M/Vdvgreater than or equal to 1.00:

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

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)

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

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

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

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

where:

A_v = Area of shear reinforcement

s = Spacing of shear reinforcement

fy = Lower bound yield strength of shear reinforcement

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

v. Strength considerations for flanged walls

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

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

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

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

vi. Vertical compressive strength of walls and piers

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

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

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.			

Table7.4 Factors to Translate Lower-Bound Masonry Properties to Expected Strength Masonry Properties

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

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

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 IAEE Manual*

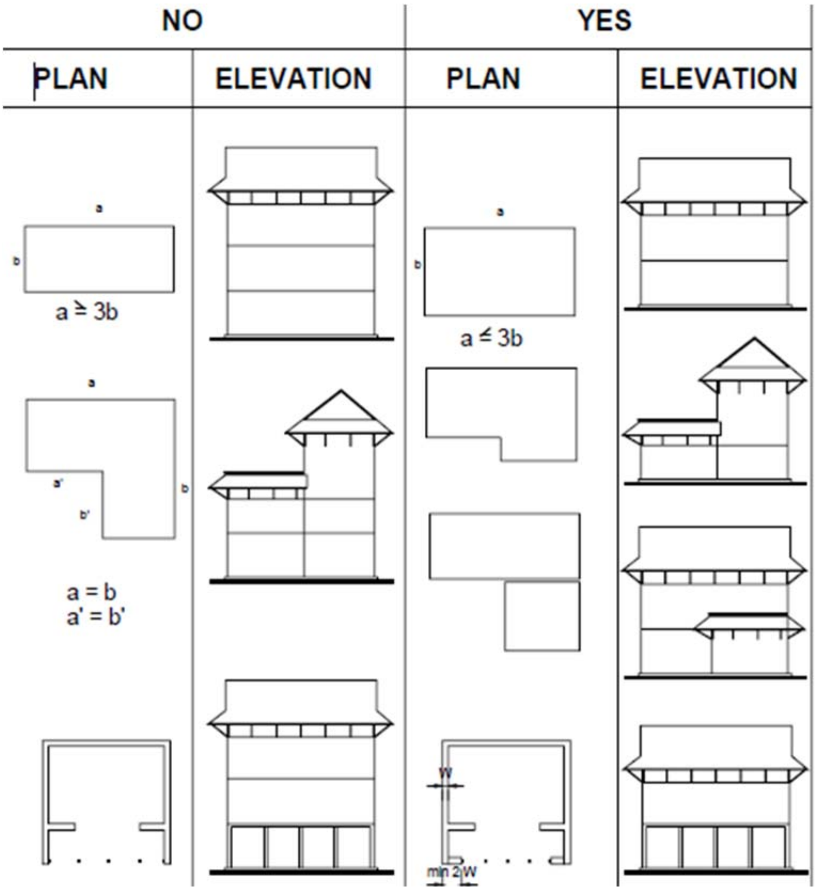
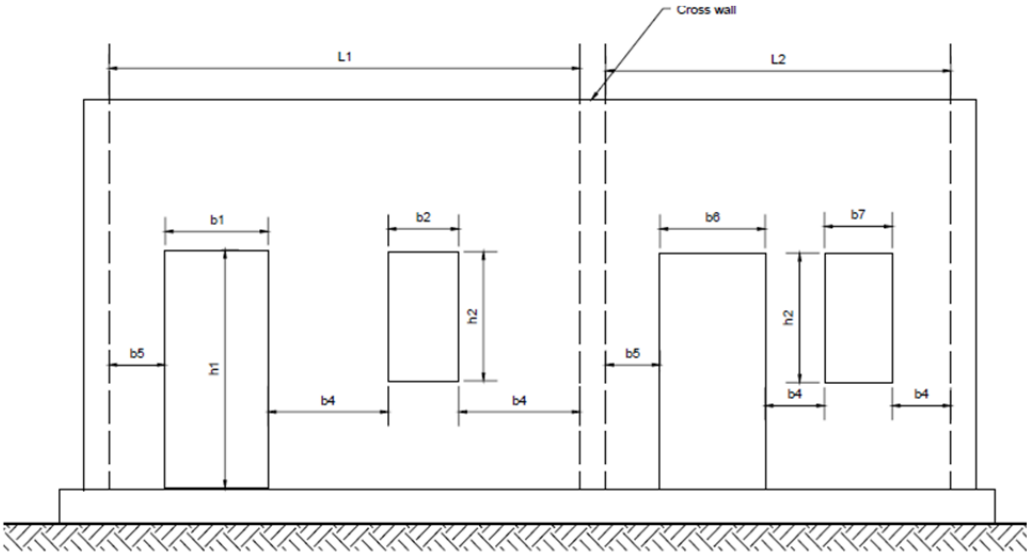


Figure 8-1 Recommended forms of buildings (Adapted from NBC 203)



RECOMMENDATION REGARDING OPENINGS IN LOAD BEARING WALLS

Note:

b1 + b2 < 0.3 L1 for one story, 0.25 L1 for one plus attic storeyed.
b6 + b7 < 0.3 L2 for one storey, 0.25 L2 for one plus attic storeyed, three storeyed
b4 ≥ 0.5h2 but not less than 600 mm
b5 ≥ 0.25 h1 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 as a way that the whole building performs as one unit in a box system. The in-plane rigidity provided by floor and roof (diaphragm) is a major factor in order to ensure box-system of the structure.

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 requirements[12].

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.

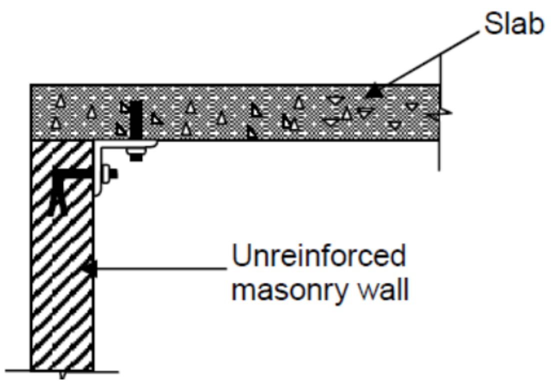


Figure 8-3 Anchorage of RCC slab with masonry wall

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*

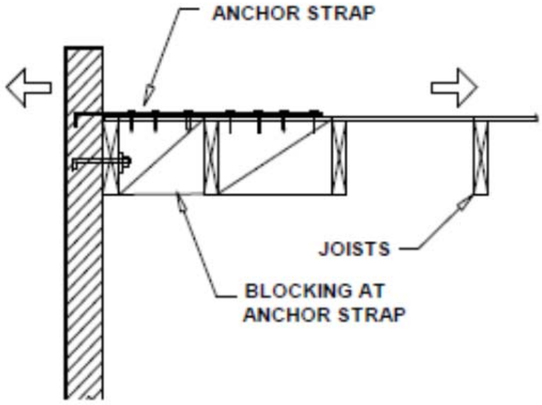


Figure 8-4 Wall anchorage

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.

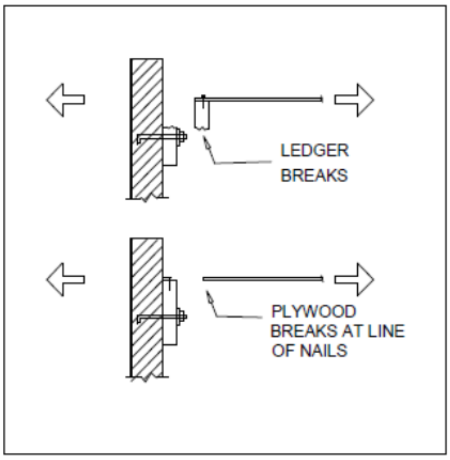


Figure 8-5 Connection relying in cross-grain of timber members induce cross-grain tension causing failure of the connection

Anchors shall be capable of development the maximum of[13]
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 SURFACE[14]

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.*

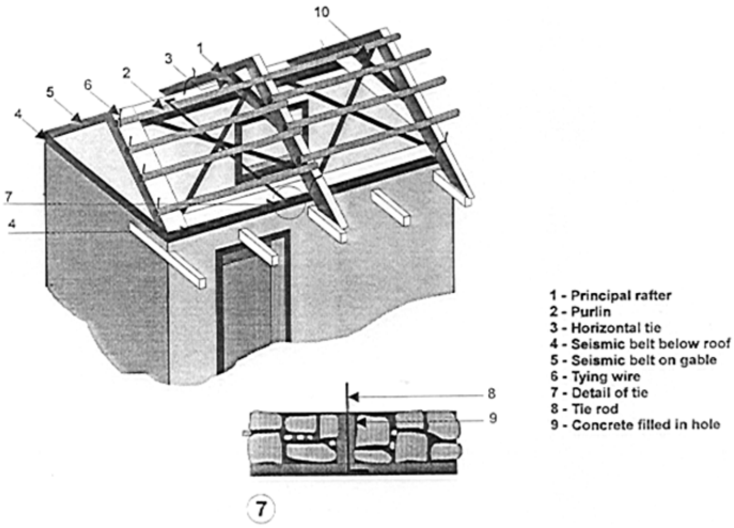


Figure 8-6 Stiffening of sloping roof structure

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*

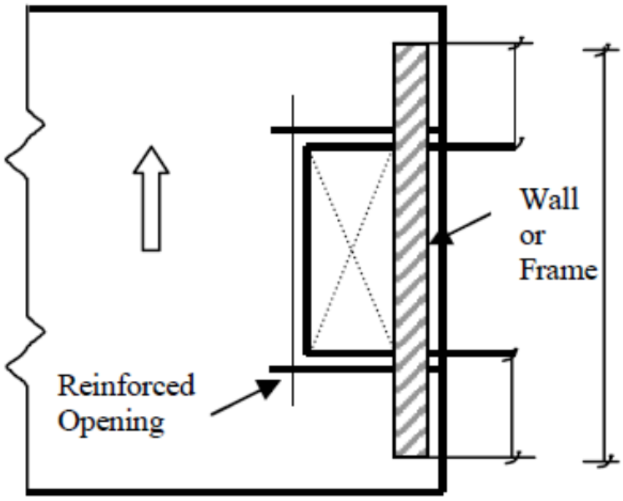


Figure 8-7 Opening adjacent to the masonry walls

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 mess with plaster on both faces of the wall
- PP Band with cement or mud plaster on both faces of the wall
- Gabion wire net with or without plaster on both faces of the wall
- The retrofitted walls must be safe against worst combination of lateral forces and designers shall check it before starting the construction.
- A sample calculation for strengthening the wall by using GI wire is shown in Annex.
- 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 OPENINGS[15]

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.

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

8.4.1.2 SEISMIC BELTS AROUND DOOR / WINDOW OPENING[16]

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.*

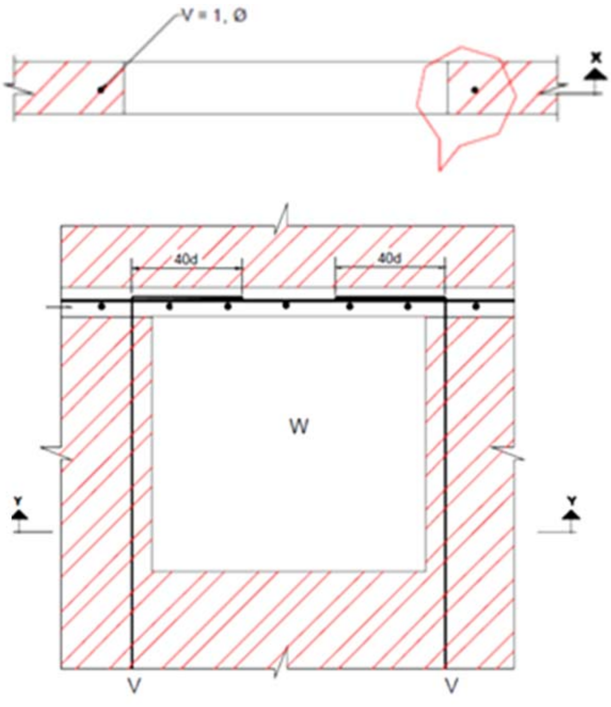


Figure 8-8 Reinforcement around opening in wall (Adapted from NBC 202)

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

8.5 STIFFENING THE FLAT WOODEN FLOOR / ROOF[17]

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 100mmwide 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).

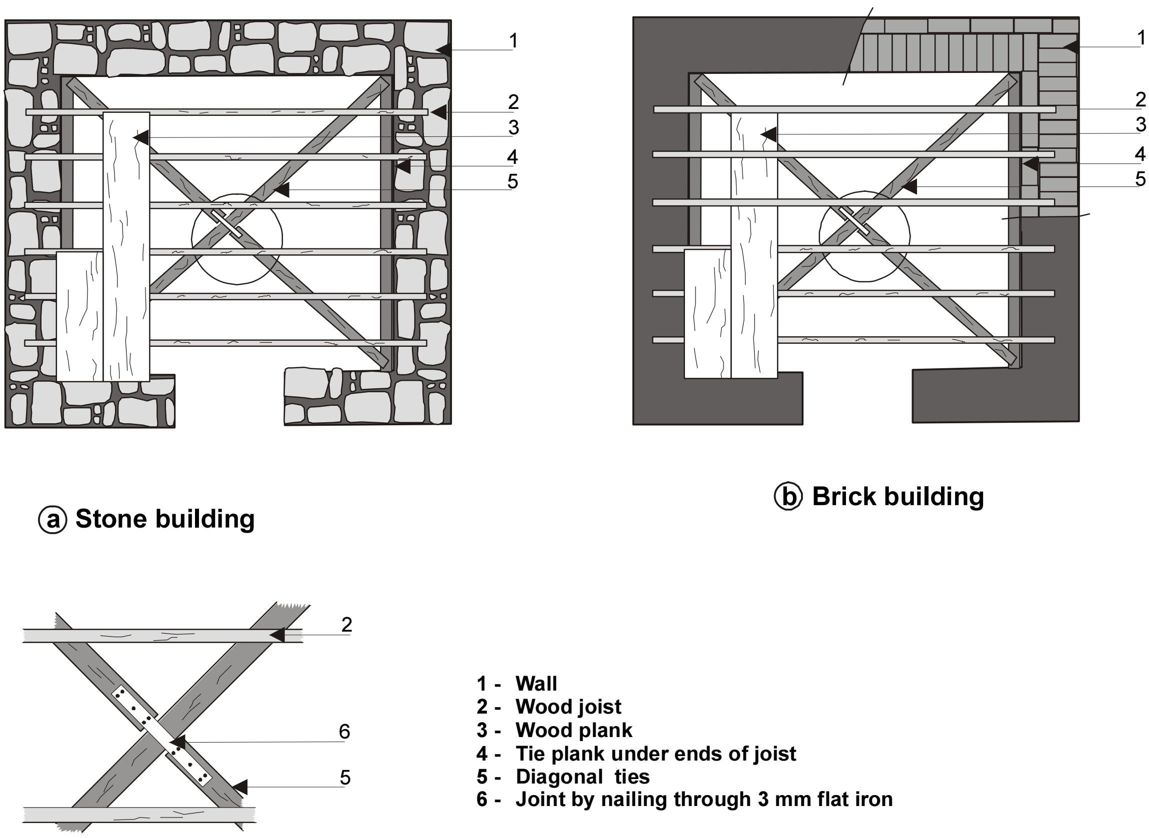


Figure 8-9 Stiffening flat wooden floor/ roof

8.6 SHEAR WALLS[18]

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.

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

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

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 criteria[19].

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.

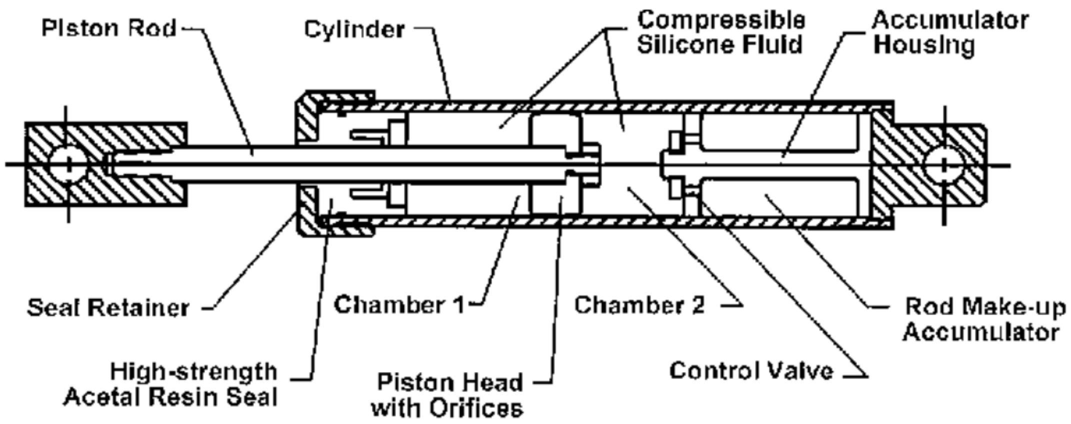


Figure 9-1 Typical Damper

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 ISOLATION[21]

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.

[20] *Two Activities- Base Isolation for Earthquake Resistance, TOTLE.*
[21] *Two 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 buildings displacement in the direction opposite the ground motion is actually due to inertia. In addition to displacing in a direction opposite to ground motion, the un-isolated building is deformed. If the deformation exceeds the constraints of the building design, the structure of the building will fail. This failure often occurs in the ground floor because most of the building's mass is above that level. Also many buildings have "soft" ground floors with many windows or unreinforced spaces for parking or lobbies.

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, space-craft, 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:

[22] *New and emerging technologies for retrofitting and repairs, DrGopalRai, CEO, R &MInternational 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).

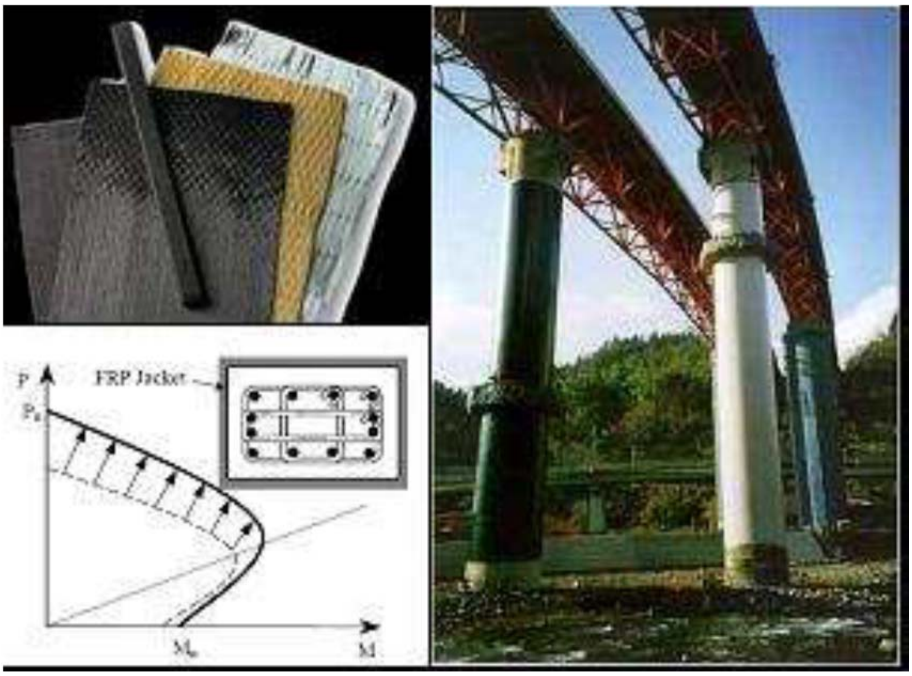


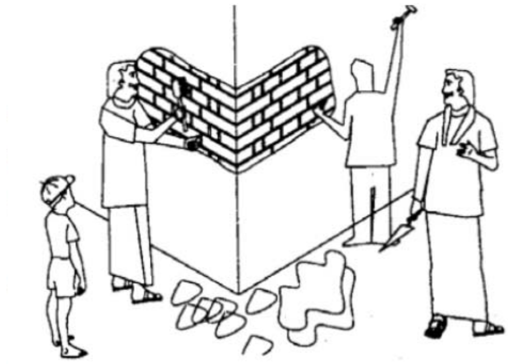
Figure 9-3 Different types and uses of Fiber wraps

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

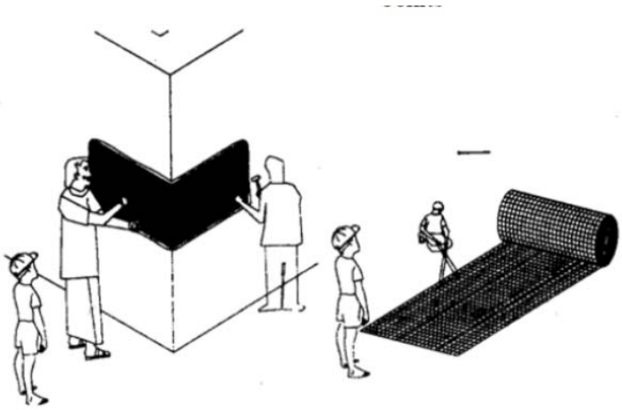
The welded wire mesh may be of 14 gauge wires @ 35 to 40 mm apart both ways. Provision of mesh on external or internal faces with an overlap of 30 cm at the corners will suffice for upto 3 m long walls. For longer walls, ferro-cement planting be provided on both faces.



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 near cement slurry is applied by a bruch 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 problem[23].

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 CRACKS[24]

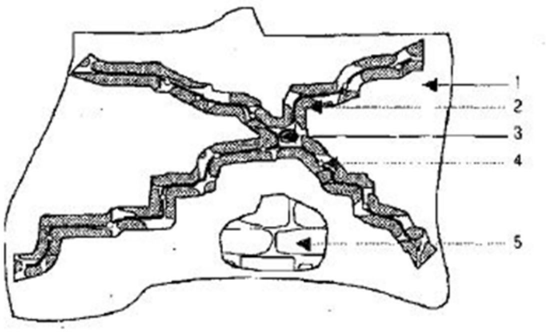
10.2.1.1 Minor and medium cracks (crack width 0.5 mm to 5.0mm)

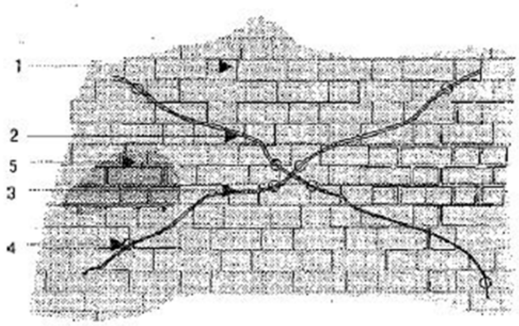
Material / Equipment required

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.

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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 Kachchh 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:

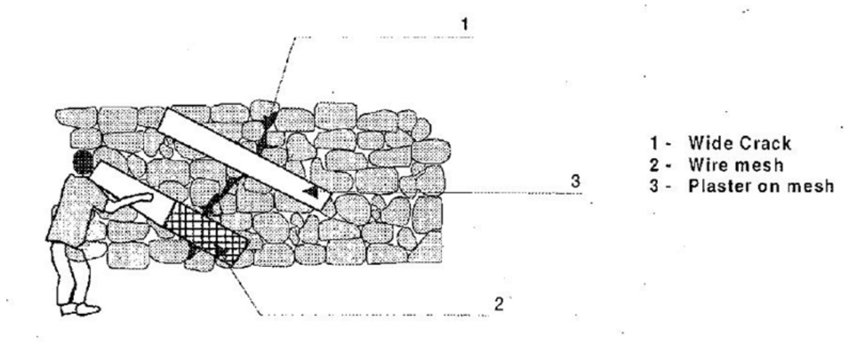

Figure 10-2 Fixing mesh across wide cracks

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

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 anchors[25]. 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 of epoxied on the wall surface, with quantity of 4-6 pieces per m^2.

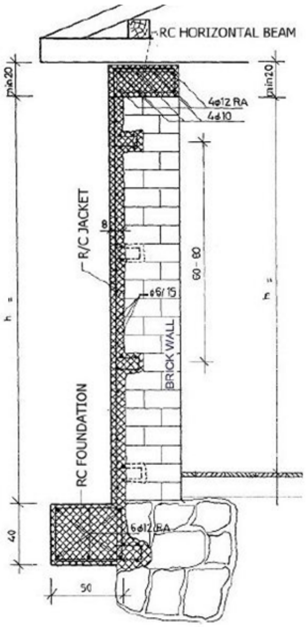

Figure 10-3 Application of RC coating

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.

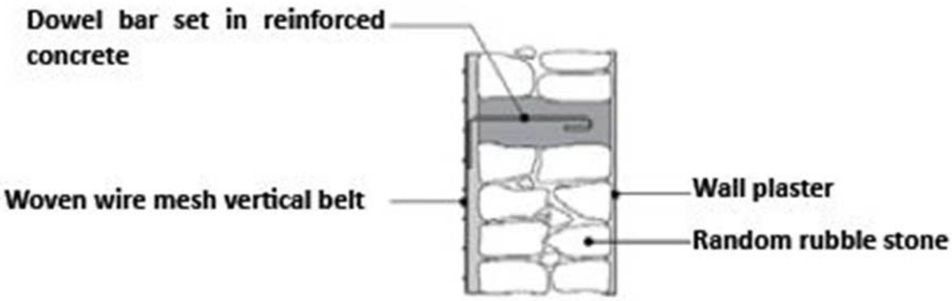

Figure 10-4 Single-sided jacketing showing steel dowels

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)

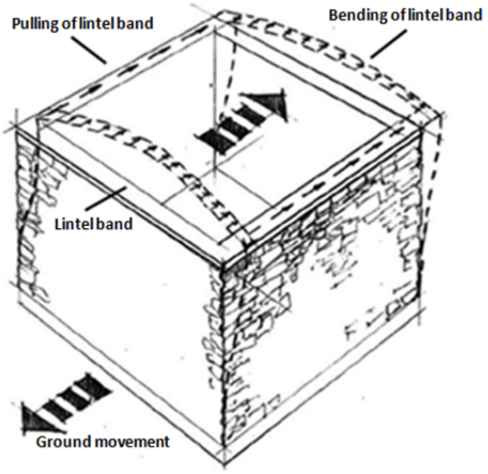

Figure 10-6 Pulling and bending of a lintel band in a stone masonry building (adapted from: Murty 2005)

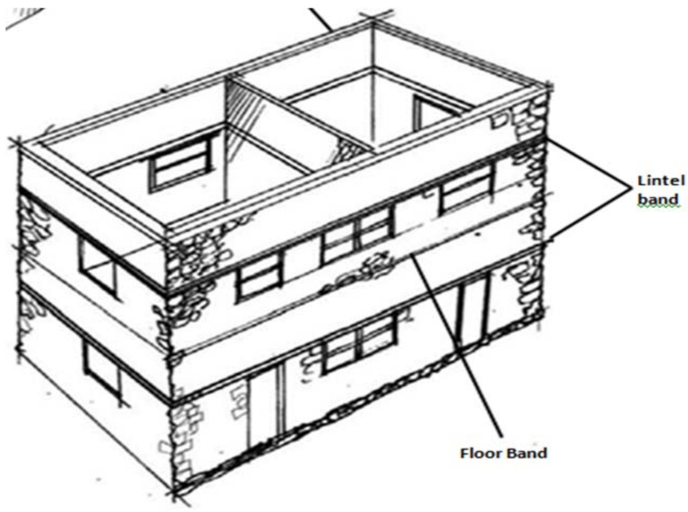

Figure 10-7 Location of seismic bands in a stone masonry building (roof omitted for clarity) (adapted from: UNCRD 2003)

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.

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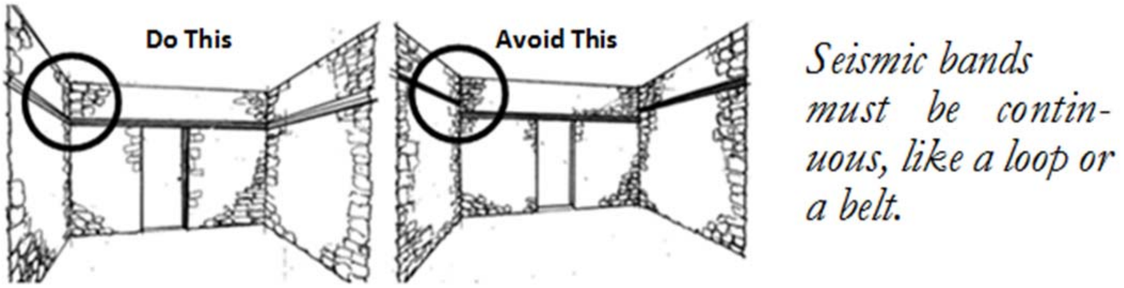

Figure 10-8 Seismic bands should always be continuous; an offset in elevation is not acceptable (adapted from: GOM 1998)

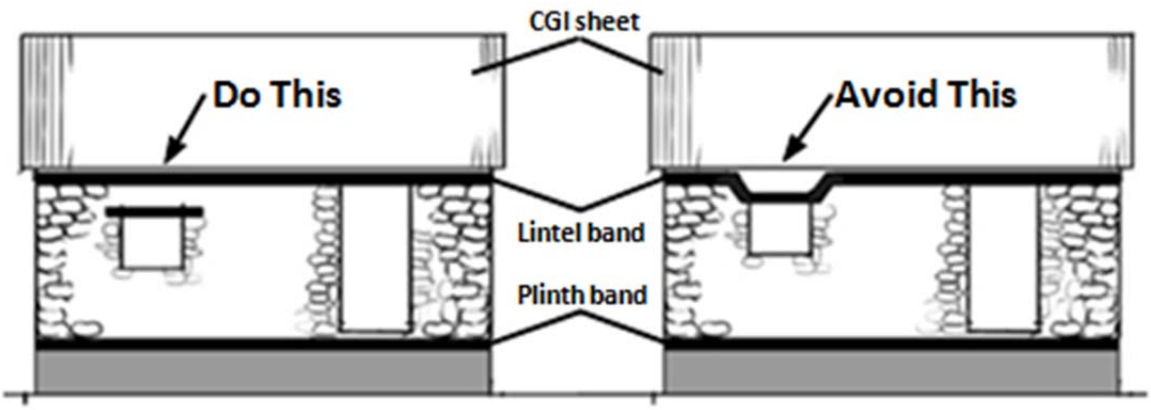

Figure 10-9 RC seismic bands should always remain level without any dips or changes in height (adapted from: GOM 1998)

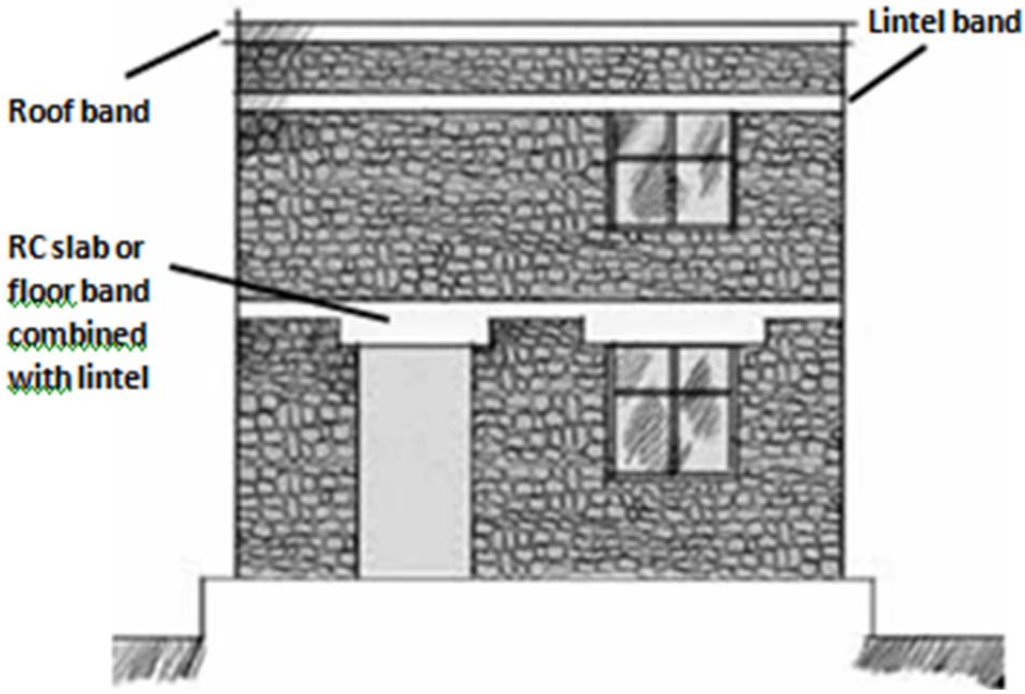

Figure 10-10 Merging RC floor and lintel bands

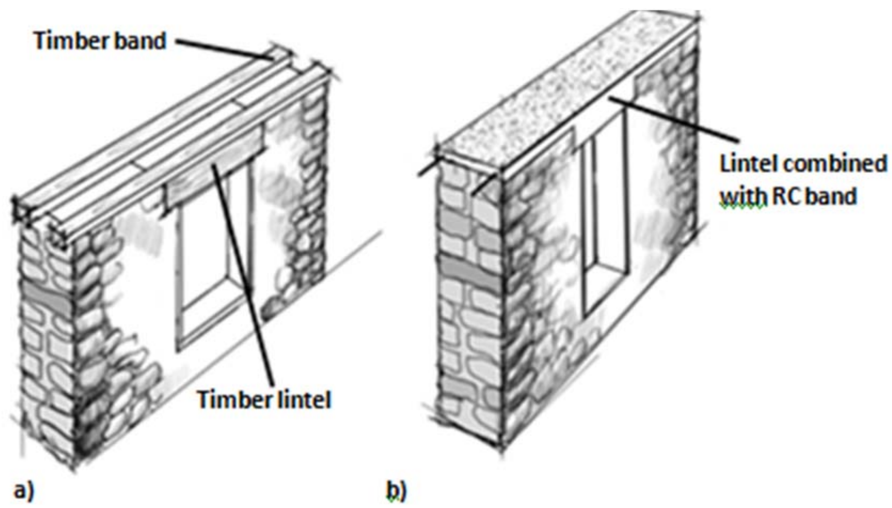

Figure 10-11 Combining floor/roof and lintel band: a) timber band, b) RC band

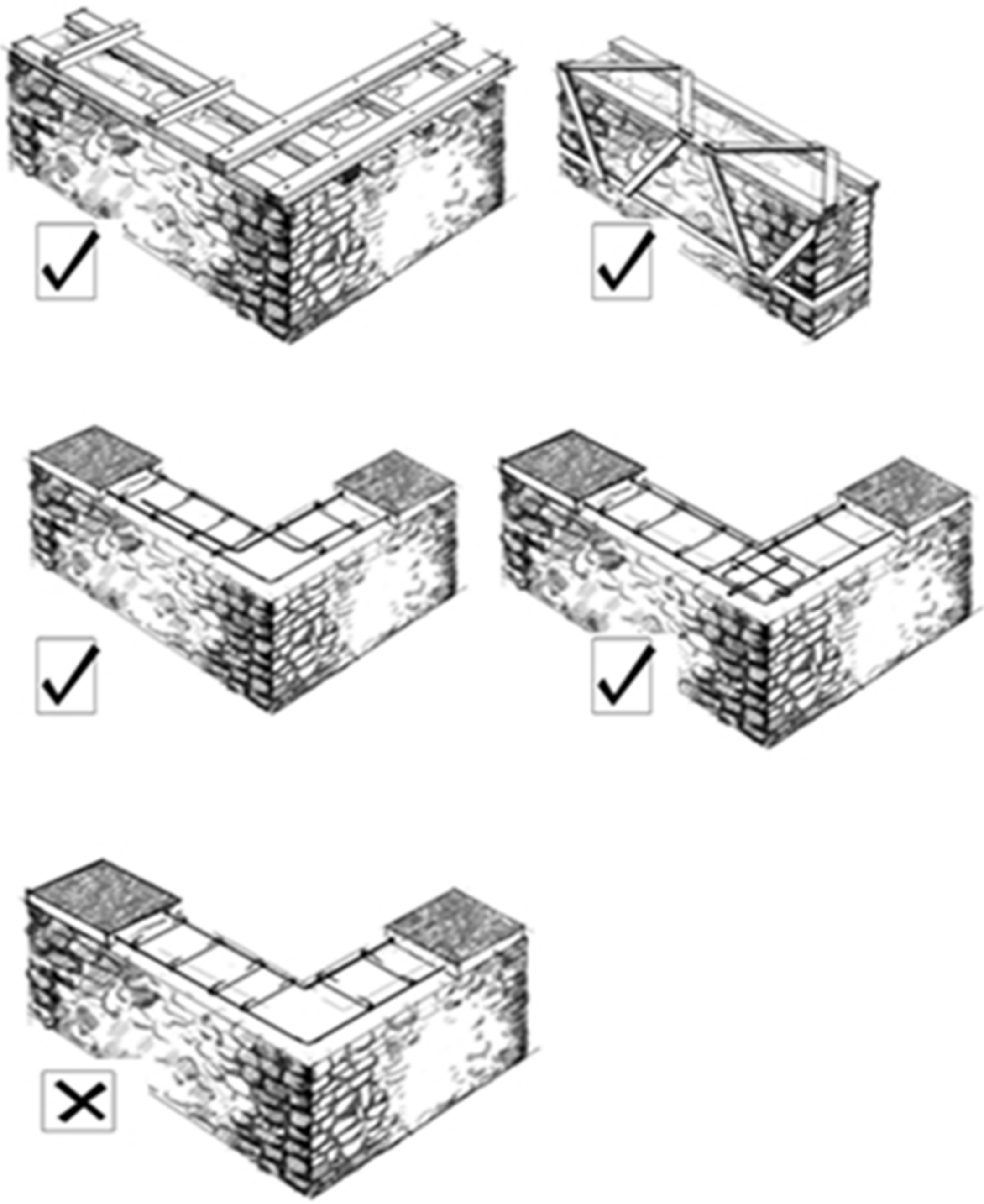

Figure 10-12 Recommended detailing of timber and RC bands (adapted from: T. Schacher and C.V.R Murty)

10.5 VERTICAL REINFORCEMENT AT CORNERS AND THE JUNCTIONS OF WALLS[28]

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.

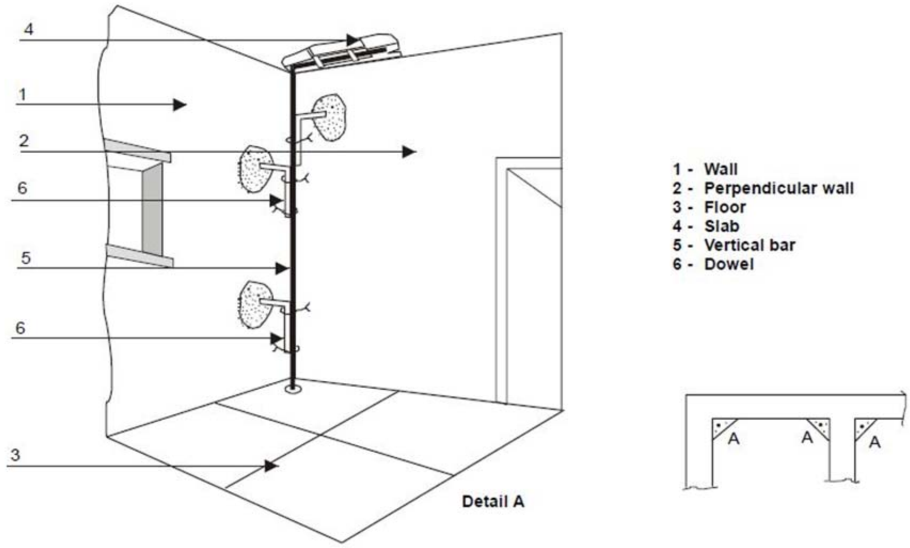

Figure 10-13 Vertical Reinforcement at corner and Junctions of Walls

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	

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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 CORNER[29]

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

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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 RESTRENGHTHENING / 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 sings 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

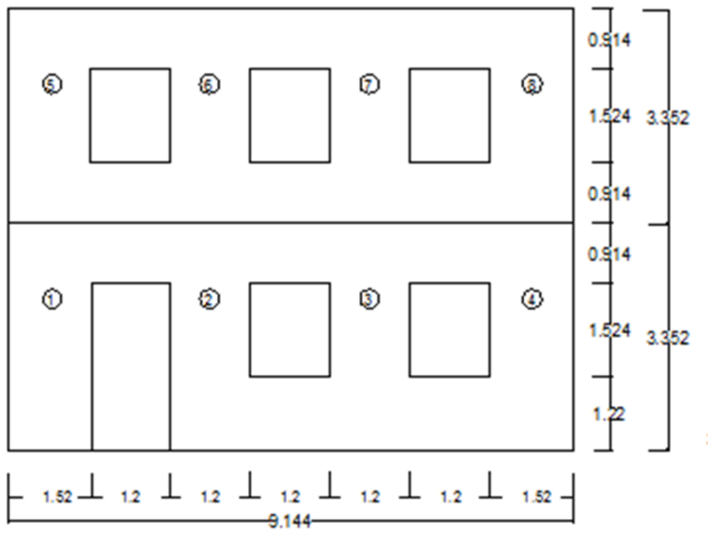
ANNEX

ANNEX A - EXAMPLES

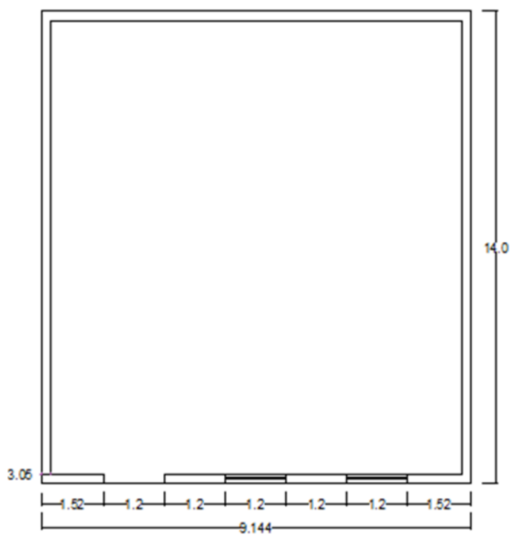
EXAMPLE 1

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.



Front Elevation



Plan

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}

Ahm = 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*h/√d_x	=	0.19953 From IS 1893:2002
Along-y=0.09*h/√dy	=	0.161255 From IS 1893:2002
For this Time Period, Sa/g=Sa/gx=Sa/gy	=	2.5 From IS 1893:2002
From IS 1893:2002, Ah=Z/2*I/R*Sa/g	=	0.300
U(assumed)	=	0.7 From IS15988:2013 Sec 5.4
Ahm	=	0.21

1.5 Calculation of DCR Values

1.5.1 Roof Diaphragm

WD = Roof Load +Load on Side Walls

WD = 439.88 KN

vu = 3.6KN/m From IITK GSDMA Guideline: Table 4

In- plane width dimension of masonry, Dd = 9.144m

∑vuDd=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*Ahm Wd/{K*∑(vuDd)} = 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

Wd = 828.5536 KN

vu = 7.3KN/m From IITK GSDMA Guideline: Table 4

In- plane width dimension of masonry, Dd = 9.144 m

∑vuDd=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*AhmWd/{K*∑(vuDd)}

= 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	∑ vuDd	DCR=2.5* AhmWd/{K*∑(vuDd)}	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

VB= A_{hm} W = 373.6195 KN From IITK GSDMA Guideline: EQ.A.1.

1.6.2. Distribution of Base Shear to Floor Levels

Qi = Vs*$W_i h_i^2$/Summation ($W_j H_j^2$,j=1,n) From IS 1893:2002

Table 1.2 Distribution of Base Shear to

Level	Wj (KN)	hi (m)	Wi*hi^2	Qi (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

Fpx = (∑Qi/∑Wi)&wpx From IITK GSDMA Guideline Claue 4.6

Table 1.3. Checking Strength of the Diaphragm

Level	Qi=VB*Wihi2/ Summation (Wjhj2)	∑Qi KN	Wi, KN	∑Wi KN	wpx, KN	0.35 Ziwpx	0.75 Ziwpx	Fpx=(∑Qi/ ∑Wi)*wpx	Fpx/ wpx
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, Fpx shall not be more than 0.75Ziwpx and less than 0.35Ziwpx
wpx = Wd = 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:

Vd = 1.5*AhmCpWd or From IITK GSDMA Guideline: EQ.A.6
Vd = VuDd From IITK GSDMA Guideline: EQ.A.7
Cp = horizontal force factor = Coefficient given in Table 5 of IITK GSDMA Guideline

1.8.1. Roof Level

Cp = 0.5
Ahm = 0.21

Wd = 439.88
Vu = 3.6
Dd = 9.144
Vd = 1.5 * AhmCpWd or 69.2811 KN
or Vd = vuDd 32.9184 KN

Therefore Adopt, Vd = 32.9184 KN

1.8.2. First Floor

Cp = 0.5
Ahm = 0.21
Wd = 828.5536
Vu = 7.3
Dd = 9.144
Vd = 1.5* AhmCpWd or 130.4972 KN
or Vd = VuDd = 66.7512 KN

Therefore Adopt, Vd = 66.7512 KN

1.9. In-plane Shear for Masonry Wall

1.9.1. Design in-plane Shears

Fwx = Ahm (Wwx + 0.5*Wd) From IITK GSDMA Guideline: EQ. A.8
But should not exceed
Force applied to a wall at level x = Fwx=AhmWwx+vu*Dd
From IITK GSDMA Guideline: EQ. A.9
Wwx=Dead load of an unreinforced masonry wall assigned to level under consideration.

1.9.1.1. Root Level

Side (Long) Walls
Ahm = 0.21
Wwx = 143.1304 KN
Wd = 439.88 KN
vu = 3.6
Dd = 9.144
Fwx = 76.24 KN This should be less than = 62.98

Adopt,
Fwx = 62.98 KN
End (Short Walls)
Ahm = 0.21
Wwx = 85.11784 KN
Wd = 439.88 KN
vu = 3.6
Dd = 9.144
Fwx = 64.05 KN This should be less then = 50.79
Adopt,
Fwx = 50.79 KN

1.9.1.2. First Floor Level

Side (Long) Walls
Ahm = 0.21

Wwx = 286.2608 KN
Wd = 828.5536 KN
Vu = 7.3
Dd = 9.144
Fwx = 147.11KN This should be less then = 126.87

Adopt,
Fwx = 126.87 KN
End (Short Walls)
Ahm = 0.21
Wwx = 170.2357 KN
Wd = 828.5536 KN
vu = 7.3
Dd = 9.144
Fwx = 122.75 KN This should be less then = 102.50

Adopt,
Fwx = 102.50 KN

Table 1.4 Summary of Storey Shear Force

Wall	Level	Storey Force Fwx KN	Wall Storey Shear Froce KN ∑Fwx
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

Va = vaDt GSDMA Guideline Clause 4.10
va = $0.1v_{te}+0.15P_{CE}/A_n$ GSDMA Guideline Clause 4.10

Where,
PCE = Expected gravity compressive force applied to a wall or pier.
va = Expected masonry shear strength
vte = average bed joint shear strength = 0.4 MPa for typical old masonry buildings
An = Area of net mortared grouted section

Rocking Shear Strength

Vr = $0.5P_OD/H$ GSDMA Guideline Clause 4.10
Where,
Vr = Rocking shear capacity of an unreinforced masonry wall or wall pier
PD = Superimposed dead load at the top of the pier under consideration
H = Total Height of the building
If Va>Vr = Rocking controlled mode
If Va<Vr = Shear controlled mode
Width of wall = 0.305 m
Length of pier 1,4,5,8 = 1.572 m An = $0.47946m^2$
Length of pier 2,3,6,7 = 1.2 m An = $0.366\ m^2$

Roof Load Calculation on pier

Total Roof Load = 153.6192 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 APPROACHFOR 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
Poison Ratio = 0.25

Property	Lower (KN/m^2)	Expected (KN/m^2)
Compressive Strength	41308.2	5379.66
Elastic Modulos	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/m3

FIRST STORE

Store Ht = 3.6576
No. of opening = 3
No. of pier = 4

Pier 1

Pier Length = 1.524 m
Effective height, hef = 2.7432
Io = 0.089906 m^4
Av = 0.464515 m^2

	LB	Ex
k (KN/m^2)	67143.617	87286.7

SECOND STORY

Store Ht = 3.6576
No. of opening = 3
No. of pier = 4

Pier 4

Pier Length = 1.524 m
Effective height, hef = 1.524
Io = 0.089906 m^4
Av = 0.464515 m^2

	LB	Ex
k (KN/m^2)	198208	257670.3

FIRST STOREY			SECOND STORY		
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	Compressive Strength	4138.2	5379.66
Masonry Unit Wt = 18.8505	Elastic Modulos	2276010	29588.13
Length = 9.144 m	GM	910404	1183525
Width = 9.144 m	Flexural tensile strength	68.97107	89.66239
Dead Load KN/m2 = 1.2	Shear strength of		
Live Load KN/m2 = 1.2	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		*Reference: Fema 356 Clause 7.4.2.2.1*
heff (m) =	2.7432.		heff (m) =	2.7432.		
An(m²) =	0.464515		An(m²) =	0.464515		
vt (KN/m²) =	179.32		vt (KN/m²) =	179.32		
PCE (KN) =	80.83		PCE (KN) =	30.14		
vme (KN/m²) =	154.2497		vme (KN/m²) =	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	

Pier 2			Pier 6			
Length (m) =	1.2192		Length (m) =	1.2192		*Reference: Fema 356 Clause 7.4.2.2.1*
heff (m) =	1.524		heff (m) =	1.524		
An(m²) =	0.371612		An(m²) =	0.371612		
vt (KN/m²) =	179.32		vt (KN/m²) =	179.32		
PCE (KN) =	64.66		PCE (KN) =	24.11		
vme (KN/m²) =	154.2443		vme (KN/m²) =	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	

Pier 3			Pier 7			
Length (m) =	1.2192		Length (m) =	1.2192		*Reference: Fema 356 Clause 7.4.2.2.1*
heff (m) =	1.524		heff (m) =	1.524		
An(m²) =	0.371612		An(m²) =	0.371612		
vt (KN/m²) =	179.32		vt (KN/m²) =	179.32		
PCE (KN) =	64.66		PCE (KN) =	24.11		
vme (KN/m²) =	154.2443		vme (KN/m²) =	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	

Pier 4			Pier 8			
Length (m) =	1.524		Length (m) =	1.524		*Reference: Fema 356 Clause 7.4.2.2.1*
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			**Expected Storey**			
Vbjs (KN) =	**257.9408**		**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		*Reference: Fema 356 Clause 7.4.2.2.2*
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:			
Vdt (KN) =	168.7961		Vdt (KN) =	54.19874		
Vtc (KN) =	48.6082	Controls	Vtc (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		*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:			
Vdt (KN) =	93.77109		Vdt (KN) =	54.20067		
Vtc (KN) =	55.9897	Controls	Vtc (KN) =	1502725	Controls	
PCL (KN)	**1045.708**	**Eq-7.7**	**PCL (KN)**	**1045.708**	**Eq-7.7**	

Pier 3			Pier 7			
Length (m) = heff (m) = An(m²) = f'a (KN/m²) = PCL (KN) = f'dt (KN/m²) = f'm (KN/m²) = 6 = QCE: Vdt (KN) = Vtc (KN) =	1.2192 1.524 0.371612 109.9533 40.86 154.2443 4138.2 1 93.77109 55.9897	Controls	Length (m) = heff (m) = An(m²) = f'a (KN/m²) = PCL (KN) = f'dt (KN/m²) = f'm (KN/m²) = 6 = QCE: Vdt (KN) = Vtc (KN) =	1.2192 1.524 0.371612 36.8933 13.71 99.68474 4138.2 1 54.20067 1502725	Controls	*Reference: Fema 356 Clause 7.4.2.2.2*
PCL (KN)	**1045.708**	**Eq-7.7**	**PCL (KN)**	**1045.708**	**Eq-7.7**	

Pier 4			Pier 8			
Length (m) = heff (m) = An(m²) = f'a (KN/m²) = PCL (KN) = f'dt (KN/m²) = f'm (KN/m²) = 6 = QCE: Vdt (KN) = Vtc (KN) =	1.524 1.524 0.464515 109.9641 51.08 154.2497 4138.2 1 93.77559 87.49477	Controls	Length (m) = heff (m) = An(m²) = f'a (KN/m²) = PCL (KN) = f'dt (KN/m²) = f'm (KN/m²) = 6 = QCE: Vdt (KN) = Vtc (KN) =	1.524 1.524 0.464515 36.87716 17.13 99.68743 4138.2 1 54.19874 23.46686	Controls	*Reference: Fema 356 Clause 7.4.2.2.2*
PCL (KN)	**1307.135**	**Eq-7.7**	**PCL (KN)**	**1307.135**	**Eq-7.7**	
Lowe Bound Storey Vdt (KN)= Strength (QCL) Vtc (KN) =	**450.1138** **248.0824**	**Controls**	**Lowe Bound Storey Vdt (KN)= Strength (QCL) Vtc (KN) =**	**216.7988** **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: Feam 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	
Deformation Controlled		***Deformation Controlled***	
Limit State:	Rocking	Limit State:	Rocking
Performance Level:	IO	Performance Level:	LS
Knowledge Factor (k)	0.75	Knowledge Factor (k)	0.75
m factor	1	m factor	3
mk QCE		mk QCE	
Floor Two (KN)	**66.7278**	**Floor Two (KN)**	**200.1834**
Floor One (KN)	**154.7043**	**Floor One (KN)**	**464.1129**
Force Controlled		***Force Controlled***	
Limit State Toe Crushing		Limit State Toe Crushing	
Knowledge factor (K)	0.75	Knowledge factor (K)	0.75
KQCL		KQCL	
Floor Two (KN)	**57.74118**	**Floor Two (KN)**	**57.74118**
Floor One (KN)	**186.0618**	**Floor One (KN)**	**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.

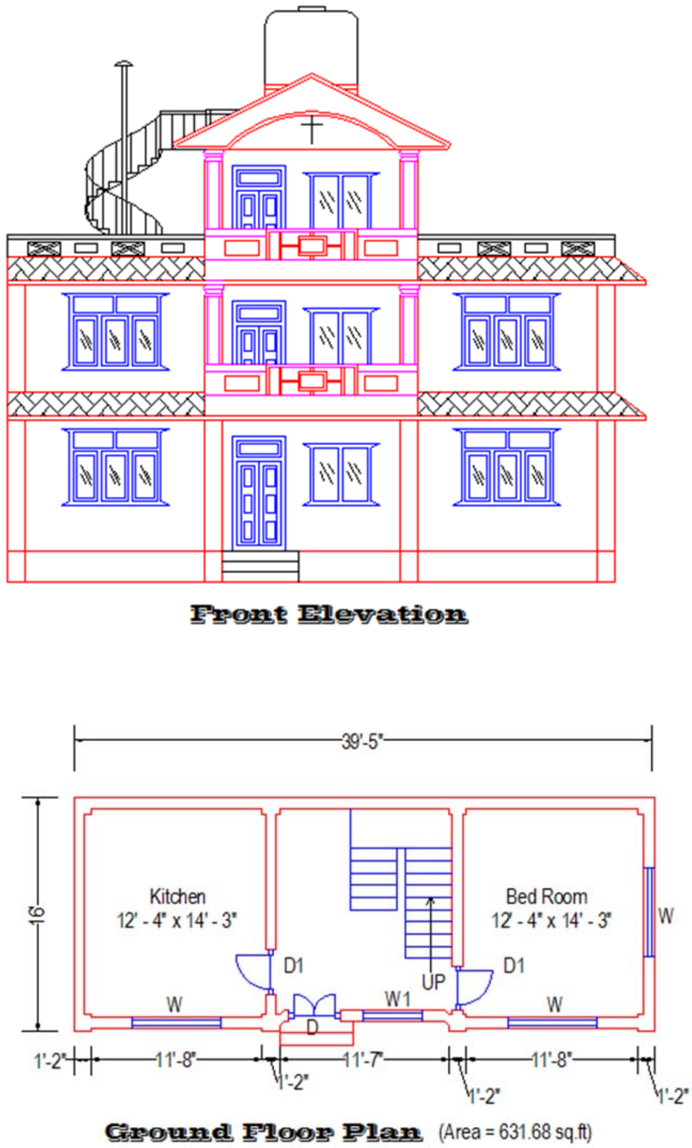


Figure A- 2: Elevation and Plan of Local Building.

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=1.82N/mm^2, E=509N/mm^2, and G=203.6N/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.

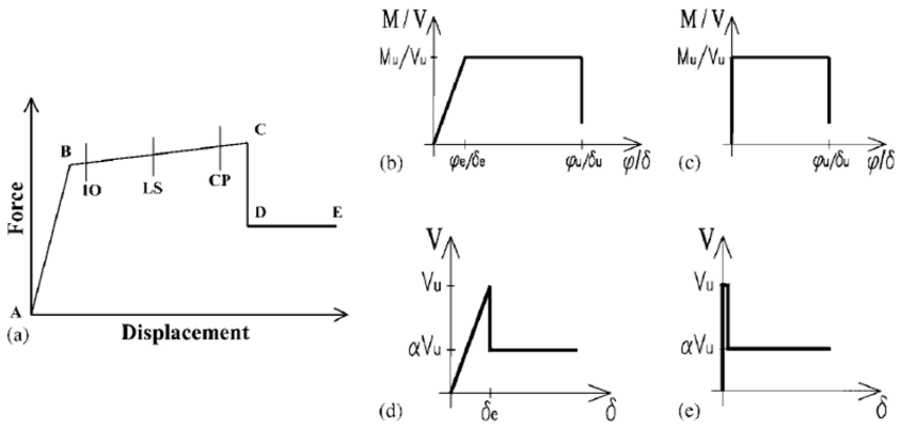


Figure A- 3: (a) Standard shape of the force vs displacement curve in SAP2000® v.10 for the plastic hinge element ; (b) and (c): behavior assumed, respectively, for the entire pier and the correspondent plastic hinge; (d) and (e): behavior assumed, respectively, for the entire spandrel beam and the correspondent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

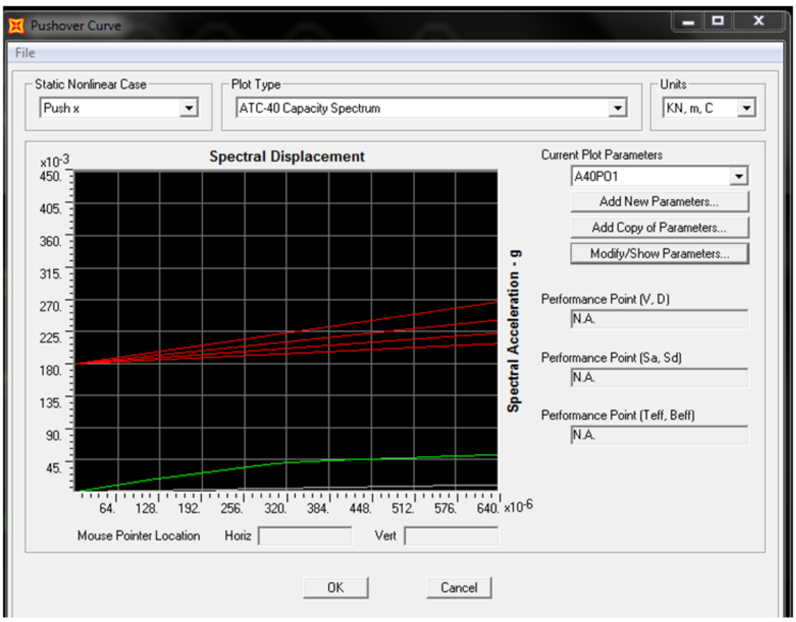


Figure A-4: Push Over Curve of Local Building without retrofitting, from SAP 2000 v 14.0.0 Analysis

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 (Ca = 0.18, Cv = 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.

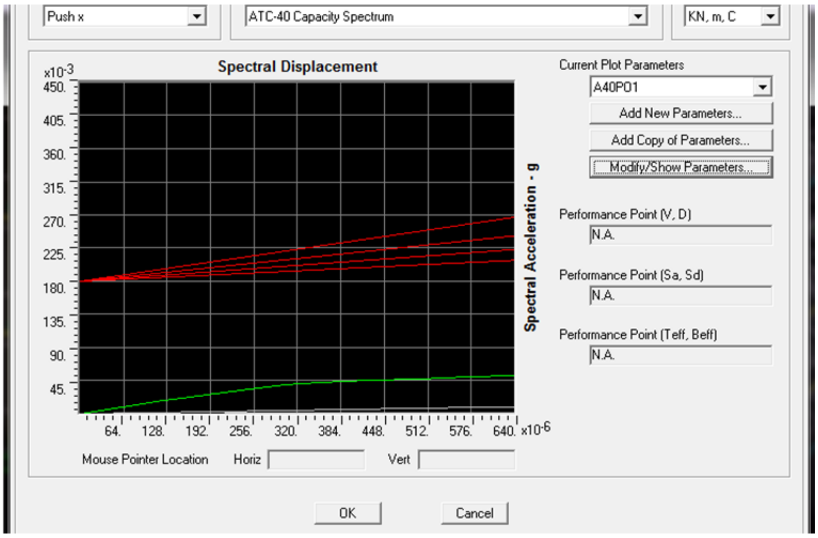


Figure A-5: Push over Curve of Local Building with lintel, from SAP 2000 v 14.0.0 Analysis

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.

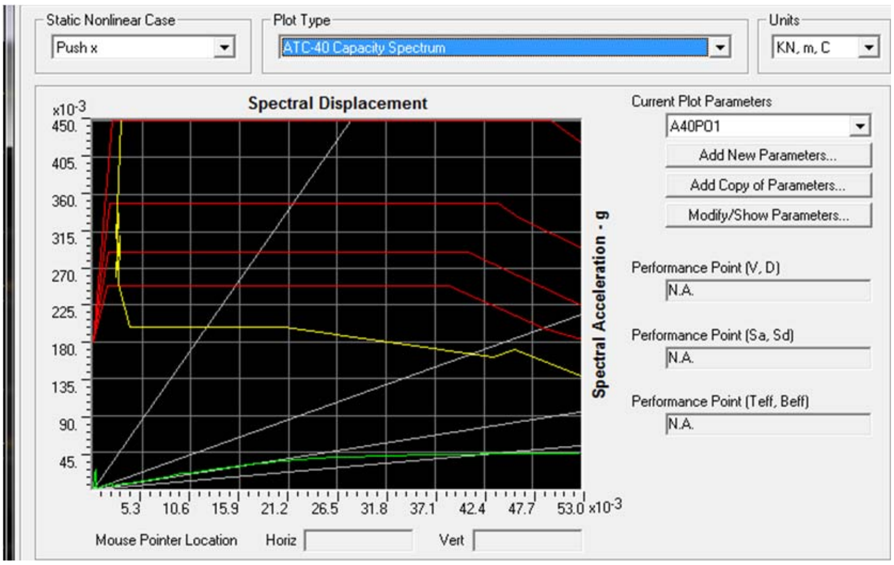


Figure A-6: Push over Curve of Local Building with lintel and rigid diaphragm, from SAP 2000 v 14.0.0 Analysis

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 KN, 0.027m). Pushover curve is shown in Figure A-7.

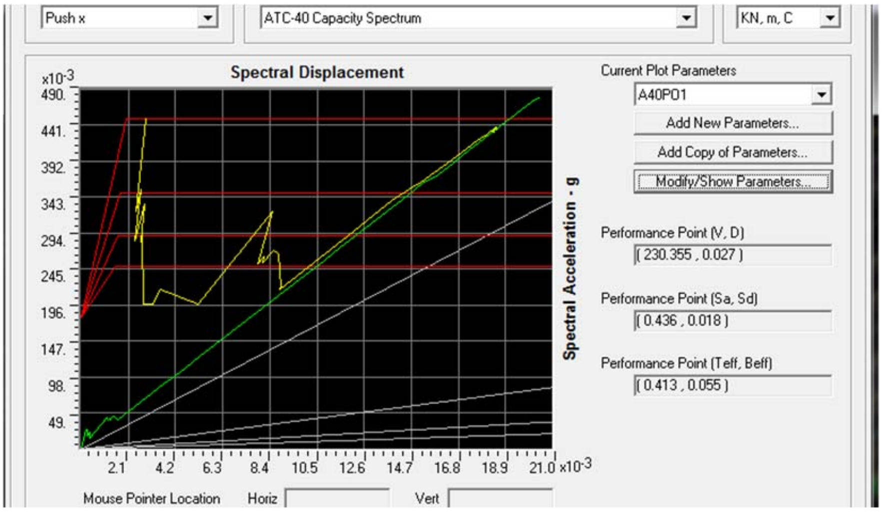


Figure A-7 Push over Curve of Local Building with lintel, rigid diaphragm and columns, from SAP 2000 v 14.0.0 Analysis

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, $^{Sa}/_{g}$ =2.5, R =3)

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

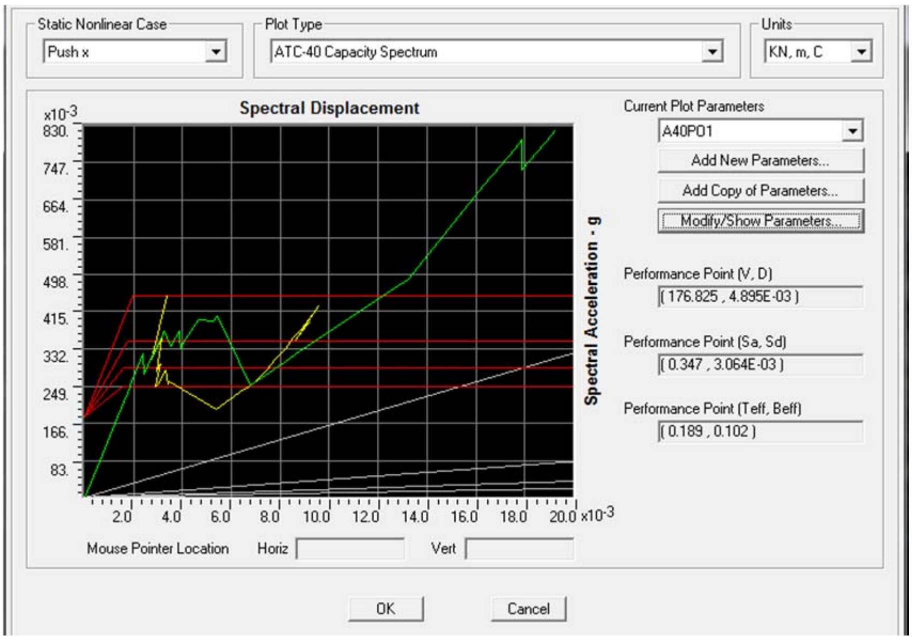
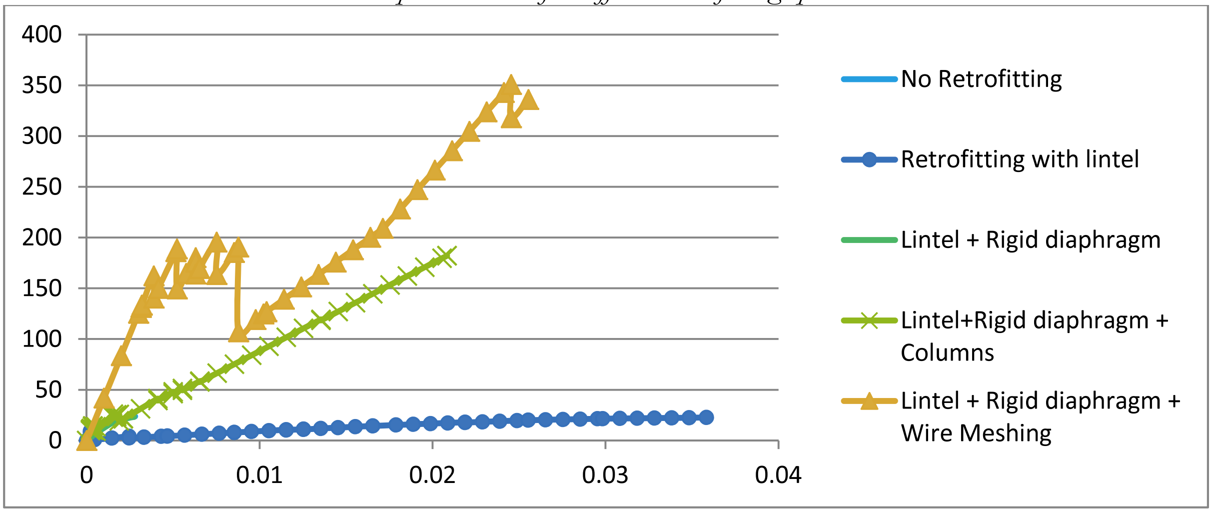


Figure A-8: Push over Curve of Local Building with lintel, rigid diaphragm and meshing, from SAP 2000 v 14.0.0 Analysis

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

EXAMPLE 4

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

Gabion wire to strengthen masonry walls

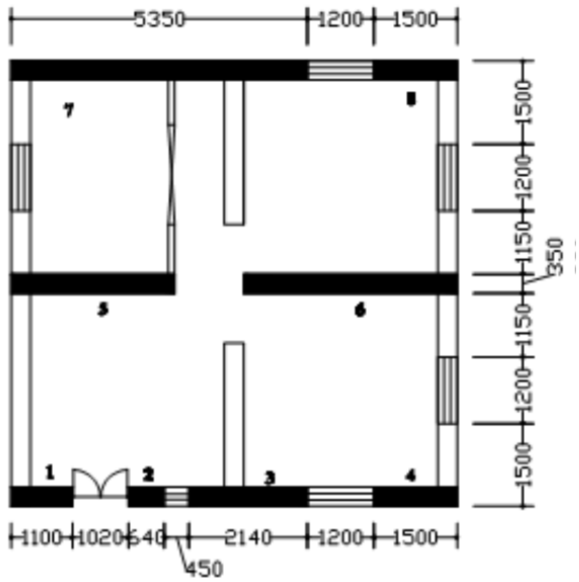


Figure A- 4: A sample masonry building

For a two storey building

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

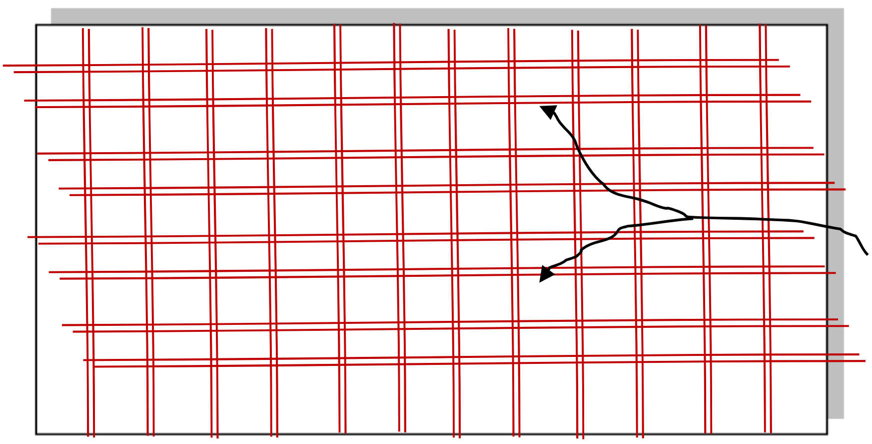
If the outer wall is supposed to carry 1/3rd of the 1st storey shear then shear in the wall is 100 KN. If the shear strength of the wall is assumed to be zero, the shear force has to be taken by GI wire placed on both sides of the wall.

Supposing 2.0 mm GI wire with x-sectional area of 3.1 mm^2 with 0.45KN/mm^2 strength[30], 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

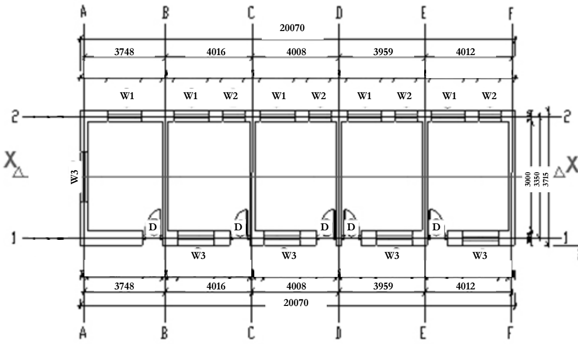


2 mm dia GI wire fabricated with 6" x 6" by box placed on both sides of the wall

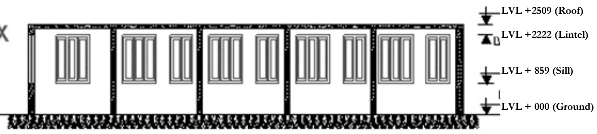
[30] *British standard wire gauge*

A.5. RETROFIT DESIGN OF MASONRY BUILDING

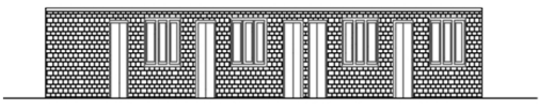
Measured drawing of the building



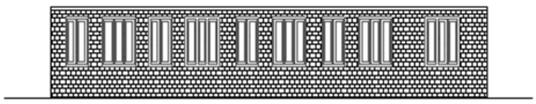
GROUND FLOOR PLAN



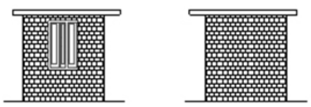
CROSS SECTION AT X-X



EAST ELEVATION



WEST ELEVATION



SOUTH AND NORTH ELEVATION

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	Dx =	19.74	m
Dimension of the building along Y	Dy =	3.37	m
Time Period along X	Tx =	**0.051**	Sec
Time Period along Y	Ty =	**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 ($550*f_m$)	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 M1corresponding to cement sand ratio of 1:5

Basic compressive strength of wall (f_b) = 0.96

Permissible compressive stress (f_c) = f_b X k_s X k_a X k_p

wall thickness= $\frac{230}{2510}$

slenderness ratio = 8.2

Stress reduction factor (k_s) = 0.92

Area reduction factor (k_a) = 0.7+1.5 A, A being the area of section in m^2

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 m^2.

width of the brick= 110 mm

height of the brick= 55 mm

Sectional Area (A) = 0.11*0.055 = 0.00605 m^2

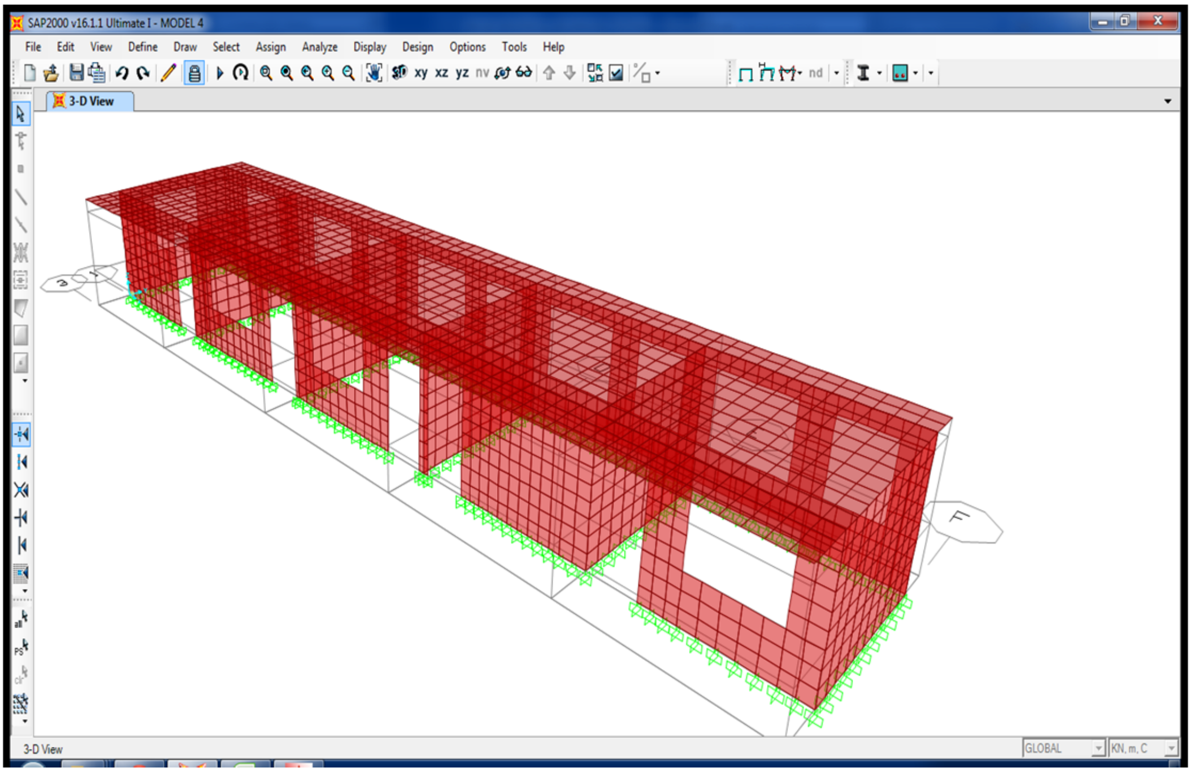
ka = 0.7+1.5*0.00605= 0.71

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.

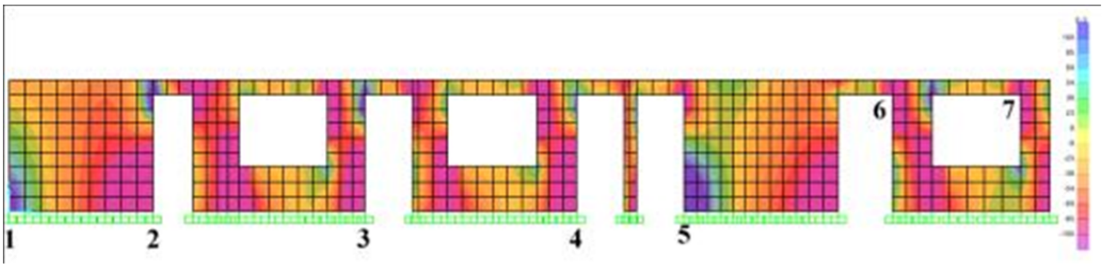
Output Case	Case Type	Global FX	Global FY	Global FZ			
Text	Text	KN	KN	KN			
LL	Lin Static	0	0	186.702			
EQx	Lin Static	-919.707	0	0			
EQy	Lin Static	0	-919.707	0			
TLL	Lin Static	0	0	85.481			
DL-ALL	Combination	0	0	1801.335	*(Including floor finishes etc)*		
seismic weight =		1916.0563	KN				
Base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

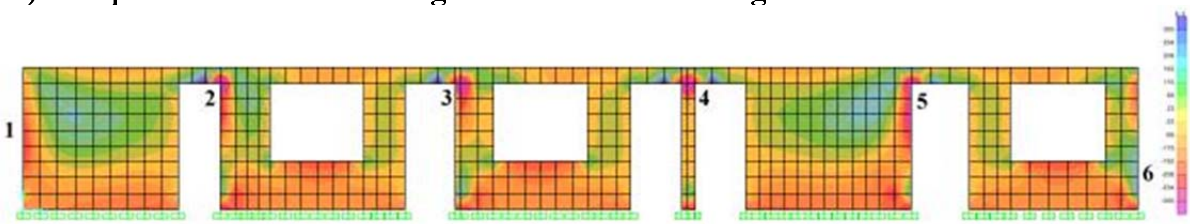
For Wall 1

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+EQy	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 EQx is equal to 168.45 KN (From SAP).

1. DESIGN OF RETROFITTING MEASURES

Wire-mesh Retrofitting Options-Capacity Calculation

Option 1 (Using gauge15 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*140*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*10 =	= 260 mm

Applying condition for the equilibrium

Neutral axis for the triangular distribution of the stress from one face=

= 24.95*1000/(0.5*1000*0.63) = 79.210 mm

Lever arm= 260-79.21/3 = 233.60 mm

Moment of Resistance per m strip		= 24.95*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^3

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%

Total allowable tensile force= 1.33*232.4*141.77/1000 = 43.82 KN

Allowable compressive strength in masonry = 0.630 N/mm²

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*10 = = 260 mm

Applying condition for the equilibrium

Neutral axis for the triangular distribution of the stress from one face=

= 43.82*1000/(0.5*1000*0.63) = 139.120 mm

Lever arm= 260-139.12/3 = 213.630 mm

Moment of Resistance per m strip=	24.95*213.63/1000		=	9.361 KN-m
Moment of Resistance per	0.4	m strip	=	3.745 KN-m
Moment of Resistance per	0.6	m strip	=	5.617 KN-m
Moment of Resistance per	0.8	m strip	=	7.489 KN-m

Summary

Allowable Tensile Strength of the wall of all wires per m strip= 43.82 KN

For 230 mm wall, Wall Area per m strip= 230000 mm²

Allowable tensile Stress in 230 mm wall= 0.19 N/mm²

Summary

Option 1	*1.829 mm@*	*20 mm c/c*	*Fe 250*
Option 2	*4.75 mm@*	*150 mm c/c*	*Fe415*

Mesh option	Jacketing Strip width(m)	Moment of Resistance(KN-m) Wall Thickness(mm)
		230
OPTION 1	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/mm² 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/mm^2

Maximum Compression due to pier action= 0.422 N/mm^2

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/mm^2

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/mm^2

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/mm^2

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+Eqy

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

Width of the inside band for resisting bending moment = 0.6232 KN-m/m
= 110 mm
Thus band size adopted = 150 mm

Thus, use 1.829 mm@20mm c/c, fy=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, fy=415 Mpa wire mesh in horizontal bands on both inner and outer faces of the wall.
Moment of Resistance of the 1m band strip= 9.361 KN-m/m
Width of the inside band for resisting bending moment = 0.6232 KN-m =70 mm
Thus band size adopted = 100 mm

Thus, use 4.75mm@150mm c/c, fy=415 wire mesh of 100 mm horizontal band on both inner and outer faces of the wall.
No. of 4.75mm dia bar in the band = 2
Equivalent Nos. of 10mm-dia bars in horizontal band = 2 (Nominal)
Use 2 -10 mm dia bars in150mm horizontal band in both inner and outer faces of the wall.
width of band= 150

iii) Check for Shear Stress

Wall ID 1 for load combination EQ_x

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

Considering **Option 1**

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

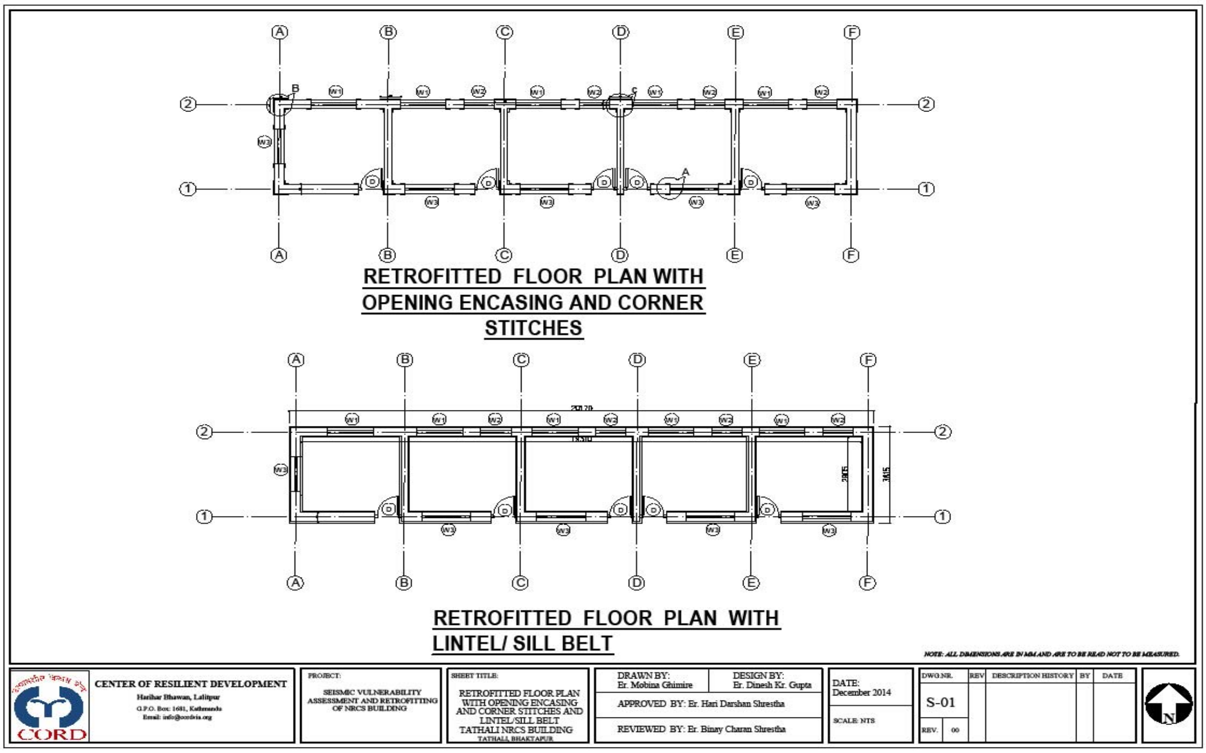
Considering **Option 2**

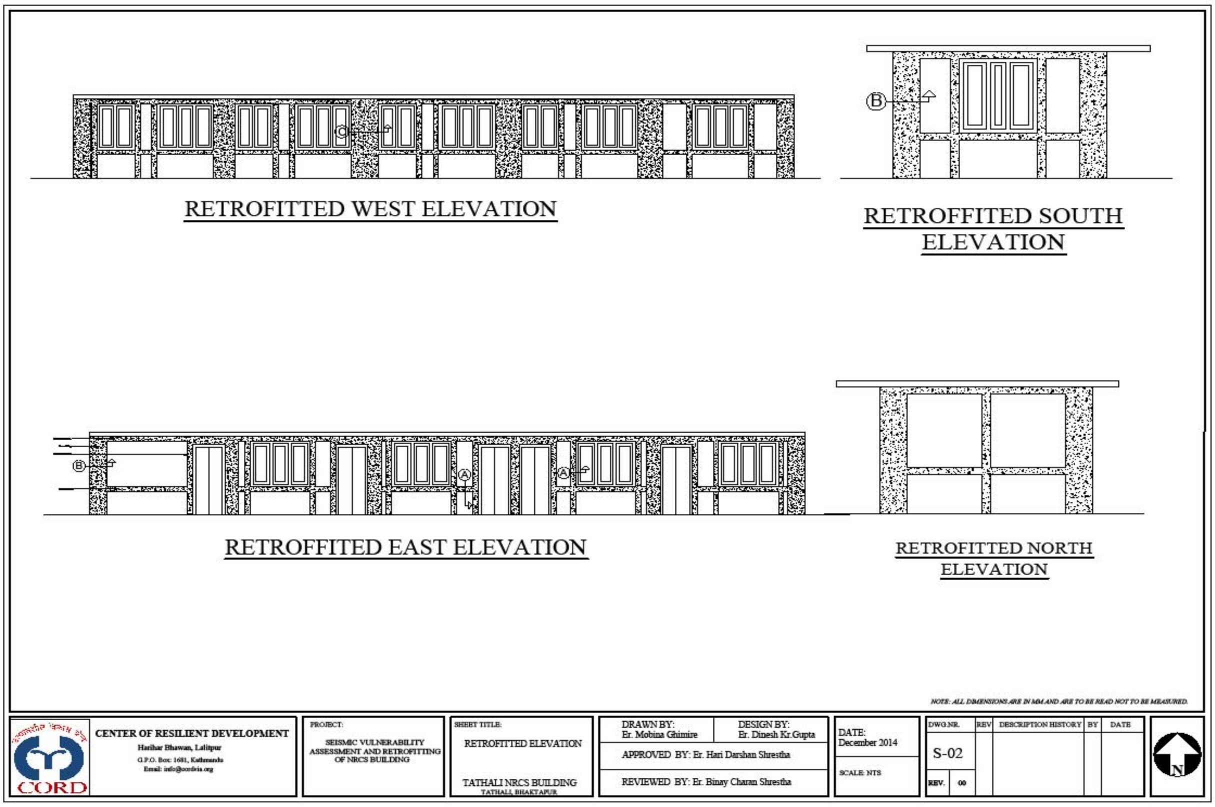
4.75mm@150mm c/c, fy=415 wire mesh vertical band in both inner and outer faces of the wall.
shear strength per meter of the vertical bands= 43.82 KN/m
shear strength of 350 mm splint band in inner face = 15.337 KN
Number of vertical bands along the inner face of the wall= 10
Total shear strength of outer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

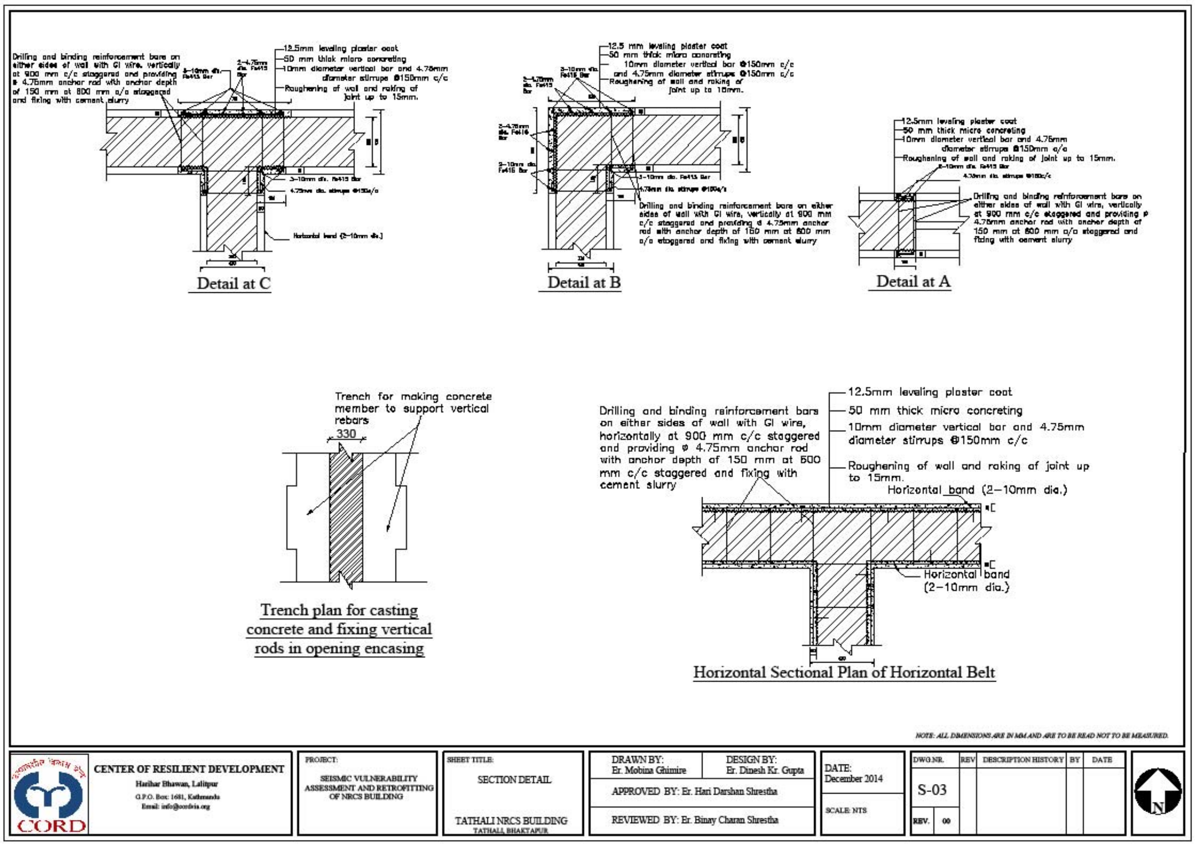
2. DESIGN OUTPUT SUMMARY

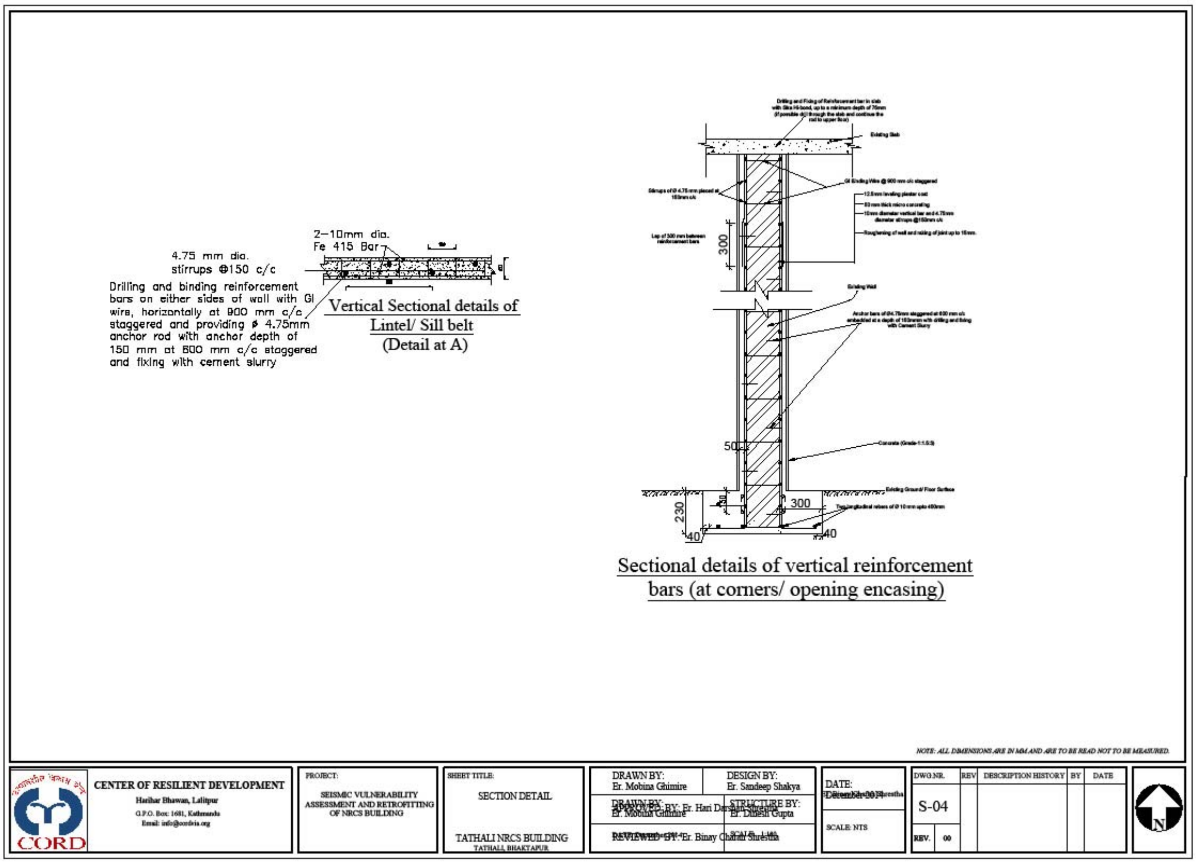
Wall ID	Option 1		Option 2	
	Vertical Band	Horizontal Band	Vertical Band	Horizontal Band
1	size=590 mm	size=150 mm	size=150 mm	size=150 mm
	1.829mm@20mm c/c, fy=250	1.829mm@20mm c/c, fy=250	Bar: 2 Nos. -10 mm dia	Bar: 2 Nos. -10 mm dia
2	size=300 mm	size=100 mm	size=150	size=150
	1.829mm@20mm c/c, fy=250	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

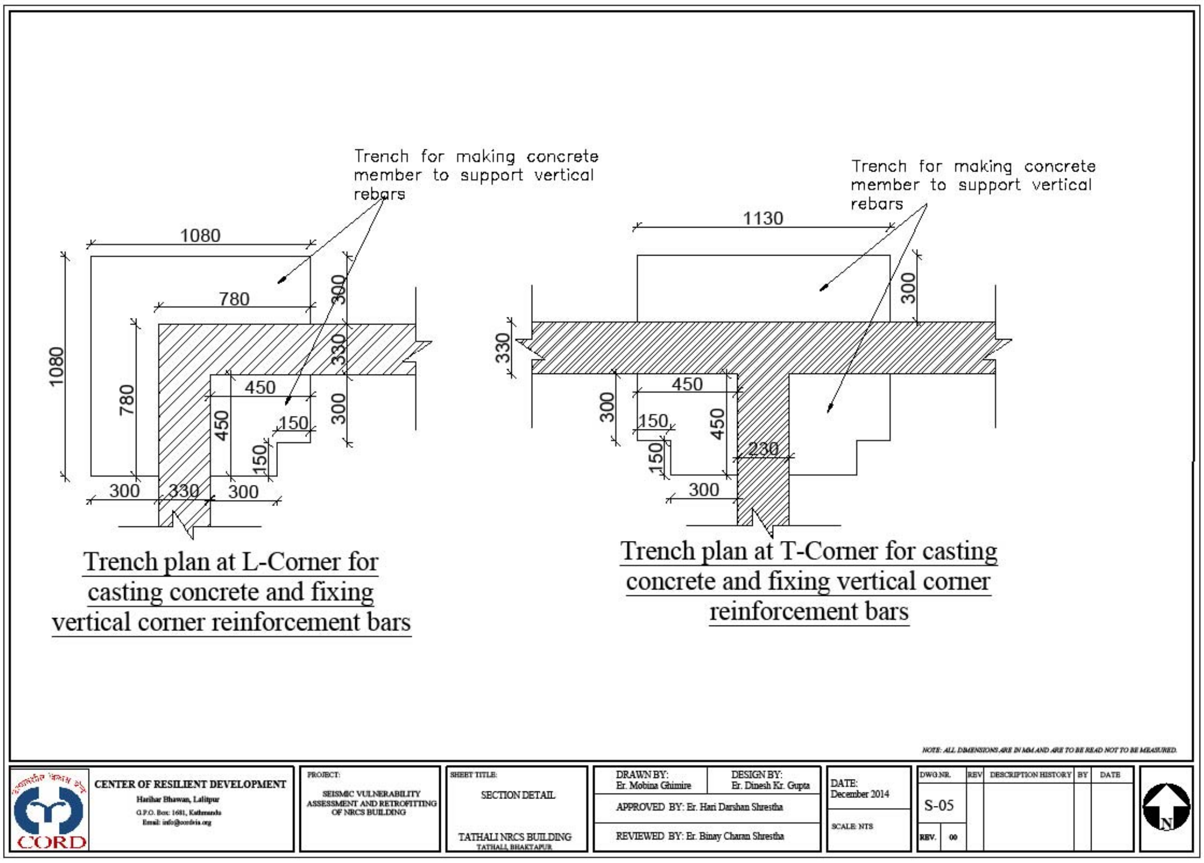
3. RETROFIT DRAWING




RETROFITTED WEST ELEVATION
RETROFFITED SOUTH ELEVATION
RETROFFITED EAST ELEVATION
RETROFITTED NORTH ELEVATION
NOTE: ALL DIMENSIONS ARE IN MM AND ARE TO BE READ NOT TO BE MEASURED.
CENTER OF RESILIENT DEVELOPMENT
Harihar Bhawan, Lalitpur
G.P.O. Box: 1681, Kathmandu
Email: info@cordvia.org
CORD
PROJECT:
SEISMIC VULNERABILITY ASSESSMENT AND RETROFITTING OF NRCS BUILDING
SHEET TITLE:
RETROFITTED ELEVATION
TATHALI NRCS BUILDING
TATHALI, BHAKTAPUR
DRAWN BY: Er. Mobina Ghimire
DESIGN BY: Er. Dinesh Kr.Gupta
APPROVED BY: Er. Hari Darshan Shrestha
REVIEWED BY: Er. Binay Charan Shrestha
DATE: December 2014
SCALE: NTS
DWG.NR.
S-02
REV. 00
REV
DESCRIPTION HISTORY
BY
DATE
N


Detail at C
Detail at B
Detail at A
12.5mm leveling plaster coat
50 mm thick micro concreting
10mm diameter vertical bar and 4.75mm diameter stirrups @150mm c/c
Roughening of wall and raking of joint up to 15mm.
Drilling and binding reinforcement bars on either sides of wall with GI wire, vertically at 900 mm c/c staggered and providing φ 4.75mm anchor rod with anchor depth of 150 mm at 600 mm c/c staggered and fixing with cement slurry
Horizontal band (2-10mm dia.)
Trench for making concrete member to support vertical rebars
330
Trench plan for casting concrete and fixing vertical rods in opening encasing
Drilling and binding reinforcement bars on either sides of wall with GI wire, horizontally at 900 mm c/c staggered and providing φ 4.75mm anchor rod with anchor depth of 150 mm at 600 mm c/c staggered and fixing with cement slurry
Horizontal band (2-10mm dia.)
Horizontal Sectional Plan of Horizontal Belt
NOTE: ALL DIMENSIONS ARE IN MM AND ARE TO BE READ NOT TO BE MEASURED.
CENTER OF RESILIENT DEVELOPMENT
Harihar Bhawan, Lalitpur
G.P.O. Box: 1681, Kathmandu
Email: info@cordvia.org
CORD
PROJECT:
SEISMIC VULNERABILITY ASSESSMENT AND RETROFITTING OF NRCS BUILDING
SHEET TITLE:
SECTION DETAIL
TATHALI NRCS BUILDING
TATHALI, BHAKTAPUR
DRAWN BY: Er. Mobina Ghimire
DESIGN BY: Er. Dinesh Kr. Gupta
APPROVED BY: Er. Hari Darshan Shrestha
REVIEWED BY: Er. Binay Charan Shrestha
DATE: December 2014
SCALE: NTS
DWG.NR.
S-03
REV. 00
REV
DESCRIPTION HISTORY
BY
DATE
N


2-10mm dia.
Fe 415 Bar
4.75 mm dia.
stirrups @150 c/c
Drilling and binding reinforcement bars on either sides of wall with GI wire, horizontally at 900 mm c/c staggered and providing ø 4.75mm anchor rod with anchor depth of 150 mm at 600 mm c/c staggered and fixing with cement slurry
Vertical Sectional details of Lintel/ Sill belt
(Detail at A)
300
50
230
300
40
40
Sectional details of vertical reinforcement bars (at corners/ opening encasing)
NOTE: ALL DIMENSIONS ARE IN MM AND ARE TO BE READ NOT TO BE MEASURED.
CORD
CENTER OF RESILIENT DEVELOPMENT
Harihar Bhawan, Lalitpur
G.P.O. Box: 1681, Kathmandu
Email: info@cordivia.org
PROJECT:
SEISMIC VULNERABILITY ASSESSMENT AND RETROFITTING OF NRCS BUILDING
SHEET TITLE:
SECTION DETAIL
TATHALI NRCS BUILDING
TATHALI, BHAKTAPUR
DRAWN BY:
Er. Mobina Ghimire
DESIGN BY:
Er. Sandeep Shakya
APPROVED BY: Er. Hari Darshan Shrestha
STRUCTURE BY:
Er. Dinesh Gupta
REVIEWED BY: Er. Binay Charan Shrestha
DATE:
December 2014
SCALE: NTS
DWG.NR.
S-04
REV.
00
DESCRIPTION HISTORY
BY
DATE


Trench for making concrete member to support vertical rebars
1080
780
300
330
300
1080
780
450
450
150
150
300
330
300
Trench plan at L-Corner for casting concrete and fixing vertical corner reinforcement bars
Trench for making concrete member to support vertical rebars
1130
300
330
450
300
150
450
150
230
300
Trench plan at T-Corner for casting concrete and fixing vertical corner reinforcement bars
NOTE: ALL DIMENSIONS ARE IN MM AND ARE TO BE READ NOT TO BE MEASURED.
CORD
CENTER OF RESILIENT DEVELOPMENT
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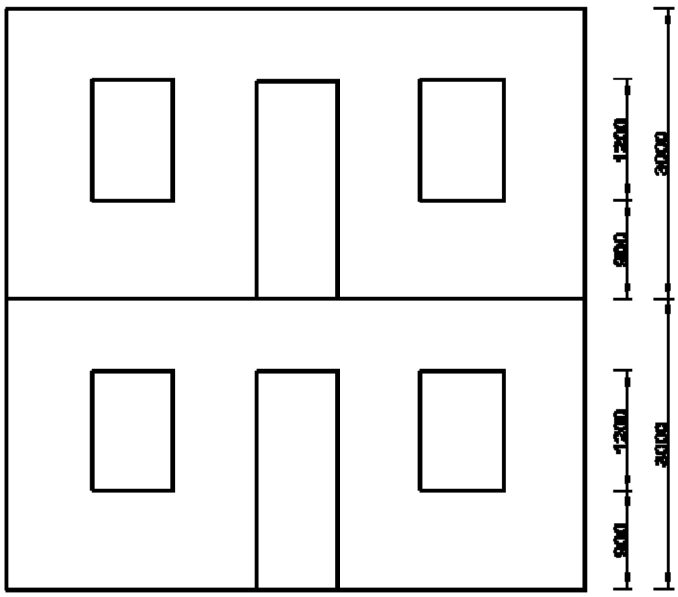
REVIEWED BY: Er. Binay Charan Shrestha
DATE:
December 2014
SCALE: NTS
DWG.NR.
S-05
REV.
00
DESCRIPTION HISTORY
BY
DATE

ANNEX B – CASE STUDIES

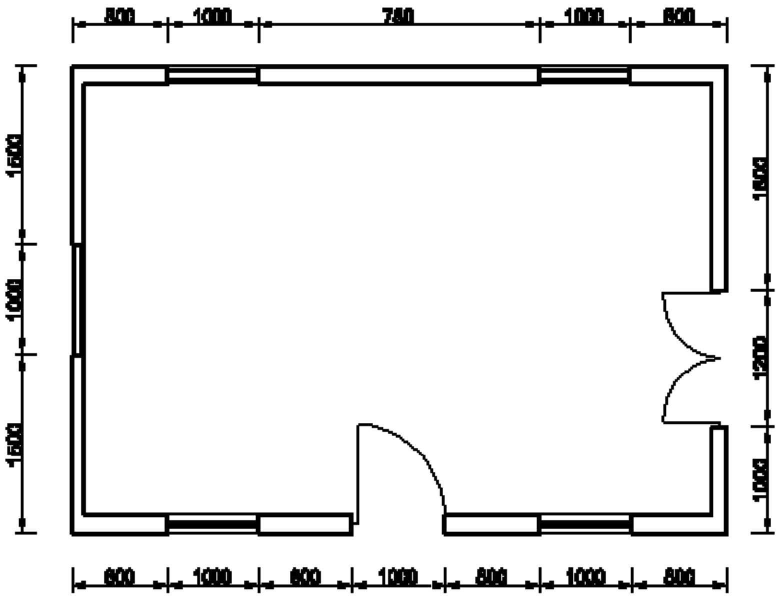
CASE STUDY - 1

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. or 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

<u>Summary of lumped load calculation:</u>

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

For imposed uniformly distributed floor loads up to 3KN/m^2, % 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, Table6	School Building
Importance factor	I	Cl. 6.4.2, Table6	1.5
Lateral load resisting system		Cl. 6.4.2, Table7	Retrofitted masonry
Response reduction factor	R	Cl. 6.4.2, Table7	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, $T_y = \frac{0.09h}{\sqrt{dx}}$	0.217 Sec.
Time period of the building along Y	T_y	Cl. 7.6.2, $Ty= \frac{0.09h}{\sqrt{d_y}}$	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

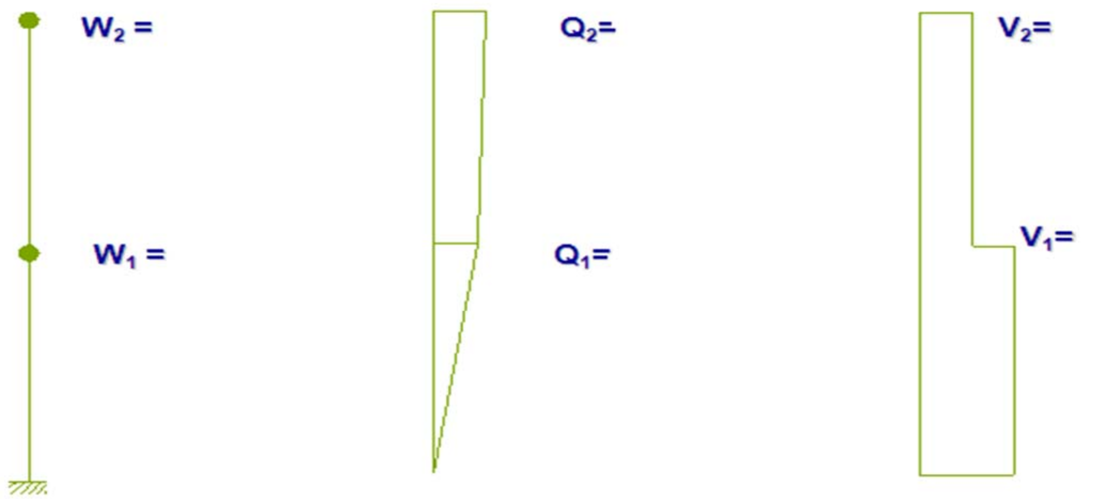
Distribution of Lateral Forces at different storey:
on Indian Seismic Code IS 1893(Part 1), Cl. 7.7.1

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

T = 0.216869219 Sec.
= 0.216869219
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 m

Load Carried by bandage

Wt. of tributary volume=(y×t×th×1)

q = C× (wt. of tributary wall of unit length)= 1.713 KN/m

M= $= \frac{ql^2}{10}$ 6.106 KN/m

Design of bandage

Assumed size of band:

d= 250 mm

t= 50 mm

Lever arm= z= 225 mm

f_s of steel= 0.56 1.25 × *fy* = 290.5 N/mm^2

$A_{st}= \frac{M}{(fs \times z)} =$ 9E-05 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= (Ast/(d*t))*100= 0.805%

Check for shear

Shear force in band= v= q*L/2+(M1+M2)/l= 7.159 KN

Considering all shear carried by band,

Induced shear stress = $\frac{V}{2td}$ = 0.286356016 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 mm^2

Interface bonding force=T=C=A_{st}×0.56×f_y= 23372.8 N

Wall length, l= 5970 mm

Band depth, d= 250 mm

Induced shear stress in band and wall interface

=T/(wall length*Band depth) = 0.016 N/mm^2

Assume minimum bond stress between concrete band and brick masonry= 0.1 N/mm^2

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 mm^2

Allowable shearing stress, $f_s=0.4f_y=$ 166 N/mm^2

Shear resistance per anchor, $F=A\times f_s=$ 2942.794643 N

No. of anchor rods reqd., $N={}^T/_F=$ 7.942382271 ≈ 8

Spacing between anchors, S=Length of band/(N-1)= 852.9 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 m width of wall.

Lateral Load, w= C*(Wt. of wall of "b" height)= 1.71306542 KN/m

Height of wall below lintel band = 1.2 m

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

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

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

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

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

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

Vertical load, P = Wt. of wall+Slab+Finishing = = 75.291 KN

Vertical Stress, $f_a={}^P/_A$ = 0.052798738 N/mm^2

Check of combined stress:

Combined Stress= $f_a+f_b=$ 0.076 N/mm^2

$f_a-f_b=$ -0.029 N/mm^2

Permissible tensile bending stress= -0.07 Mpa

Remark: ***No, tension reinforcement required***

Design of stitches:

Lateral Load carried by stitch, w= C× (Wt. of triangular portion of wall)= 3.854 KN

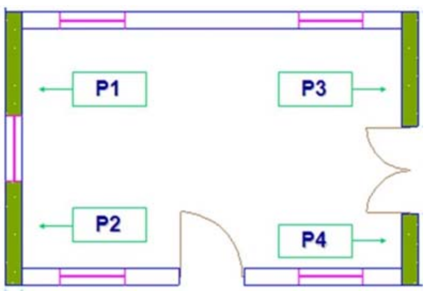
Count lintel and sill also as stitch band; therefore total number of stitch considered= 3

$A_{st}=\frac{w}{(No.of\ stitches\times 0.56\times f_y)}$ = 5.528 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 P_1 and P_2 along Grid 1 and P_3 and P_4 along grid 2
Analyzing piers P_3 and P_4 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 KN
Masonry, E = 2400000 KN/m^2

Pier	Height h (m)	Depth d (m)	Width b (m)	I- $bd^3/12$ (m^4)
P_4	2.1	1.00	0.23	0.01917
P_3	2.1	1.8	0.23	0.11178
				$\sum$K=

Stiffness (KN/m)	Proportion of lateral load carried by pier	Lateral load carried by pier
K= $\frac{12EI}{\left[\left\{1+2.4\left(\frac{d}{h}\right)^2\right\}h^3\right]}$	p= $\frac{K}{\sum K}$	**$F_i = V_i \times P$**
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_o):

Lateral load Q_1= = 62.15930396 KN

Q_2= = 75.39981004 KN

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

Centroid of pier P_3& P_4= $\frac{(A_3X_3 + A_4X_4)}{(A_3 + A_4)}$ = 1.914 m

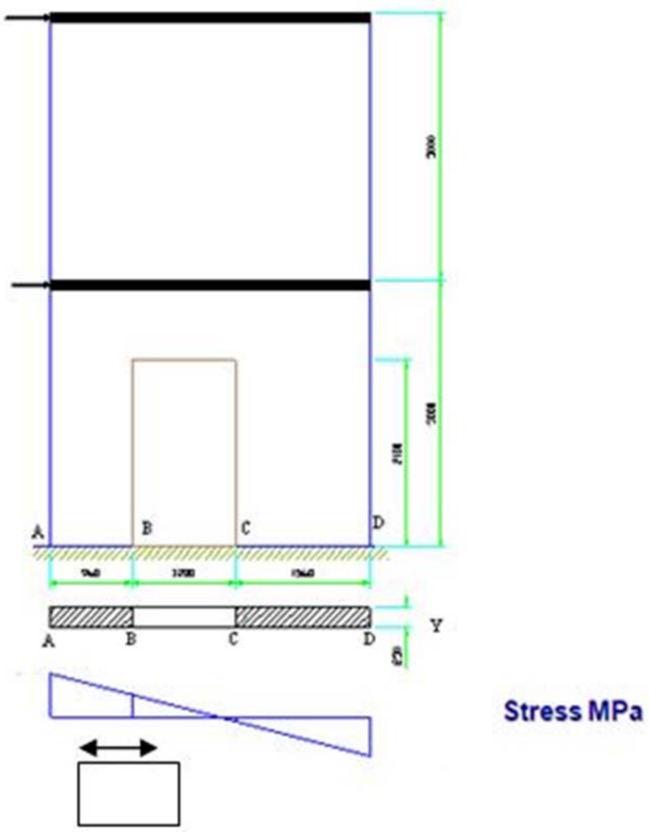
h_3= = 1.186 m

h_4= = -0.914

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

M.O.I about centroid, $I_C=I_3+A_3h_3^2+I_4+A_4h_4^2$= 9.05258E+11 mm^4

Overturning stress at different piers



Points	y (mm)	$f_0=Mo^y/I_c$(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 (fa):

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

Ground Floor slab,
(Triangular area× ((thickness of slab×unit wt. of rcc+Floor finish)+LL)= 28.5KN

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

Total vertical load = 138.868 KN

Area= 0.644 m²

Vertical Stress, $f_a = \frac{\text{Vertical load}}{\text{Area}}$ **= 215.633 KN/m²**

= 0.21563 N/mm²

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

Distance of NA from point A, x = 0.729024286 m
= 729.0242864 mm

Total tensile force, $T=f_nA$ = 75637.9962 N

$A_{st}reqd = \frac{T}{(0.56 \times f_y)}$ = 325.4646997 mm²

Dia. of rod provided = 10 mm

NO. of rods required = 4.142277996 ≈ 5

Check for Shear

Shear force, $V=F_4$ = 16.15 KN

Shear stress= $^V/_A$ = 0.070211668 N/mm²

Where, f_d=compressive shear stress=f_a= 0.215632919

From IS 1905, 5.4.3, Permissible shear stress= $0.1 + \frac{f_d}{6}$ =0.14 N/mm²

Remarks: Safe

CASE STUDY: 2

B.2 A NALYSIS 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 m^2. 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

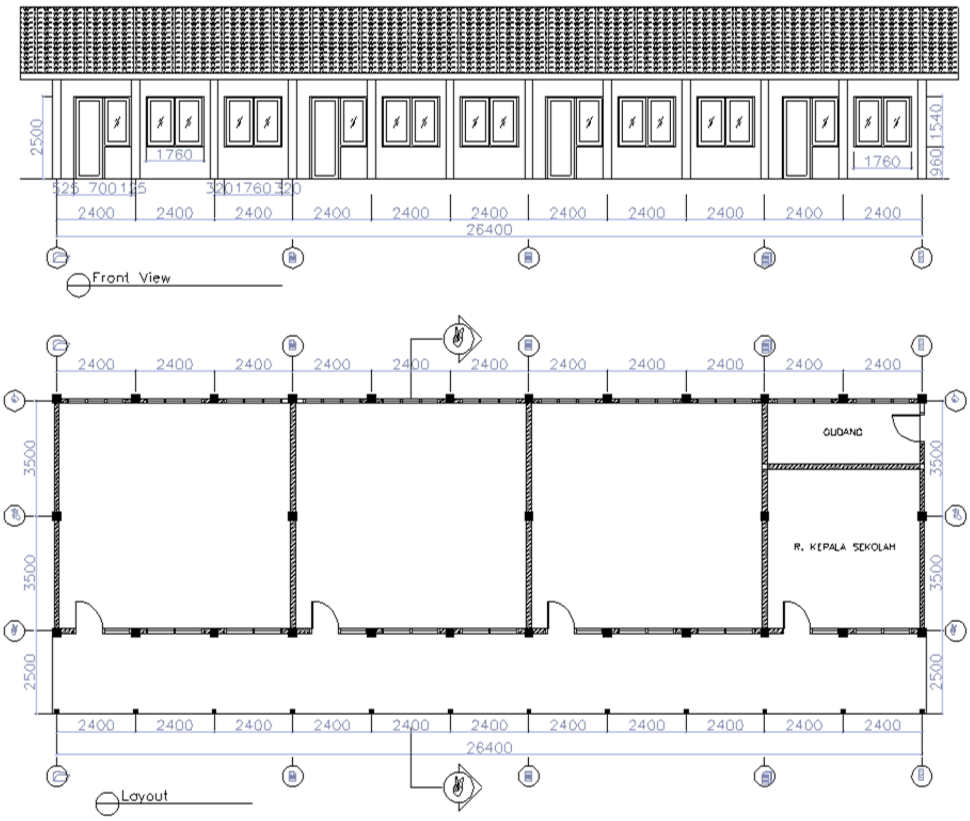


Figure B- 2: Drawings of the existing school building

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. Shrestha 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 pga 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.

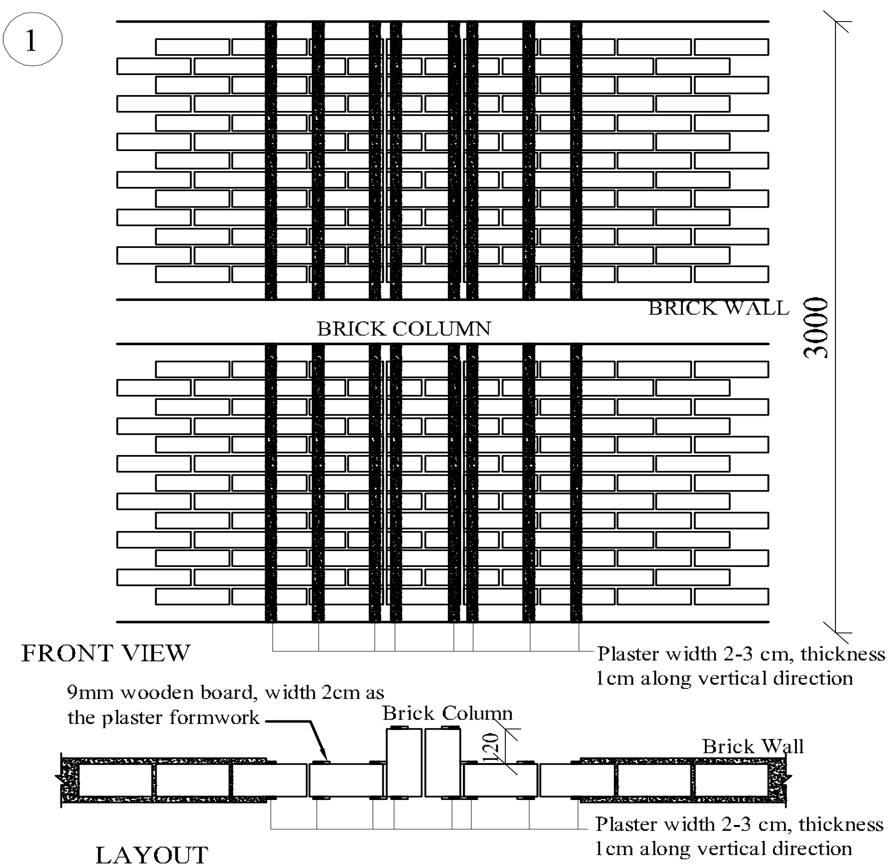
B.2.4 RETROFITTING DESIGN

B.2.4.1 RETROFITTING APPROACH

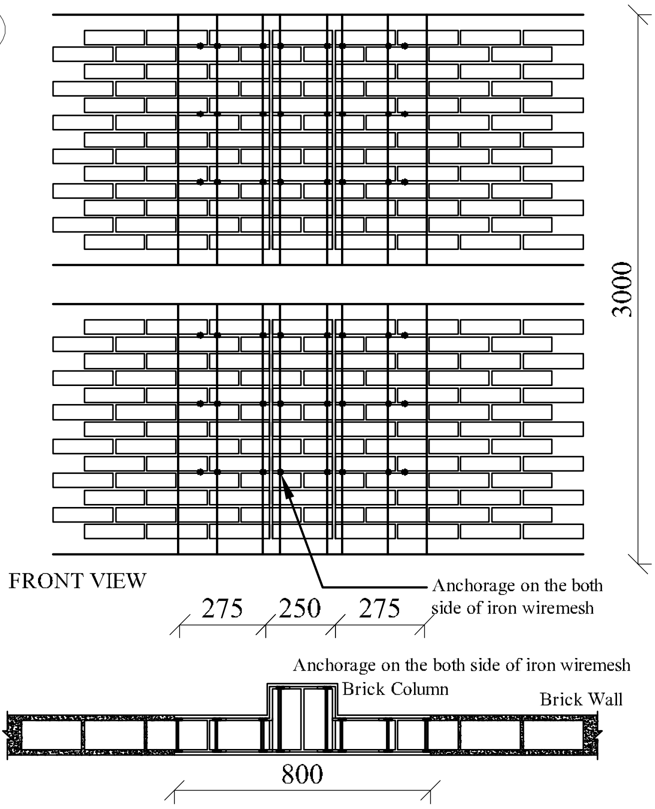
The buildings were retrofitted using iron wire mesh for strengthening wall elements, inserting columns on building corners. The wire mesh was installed in the locations of ordinary beams and columns, as well as diagonally on the perimeter walls. The wire mesh was installed on the both side of the wall 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

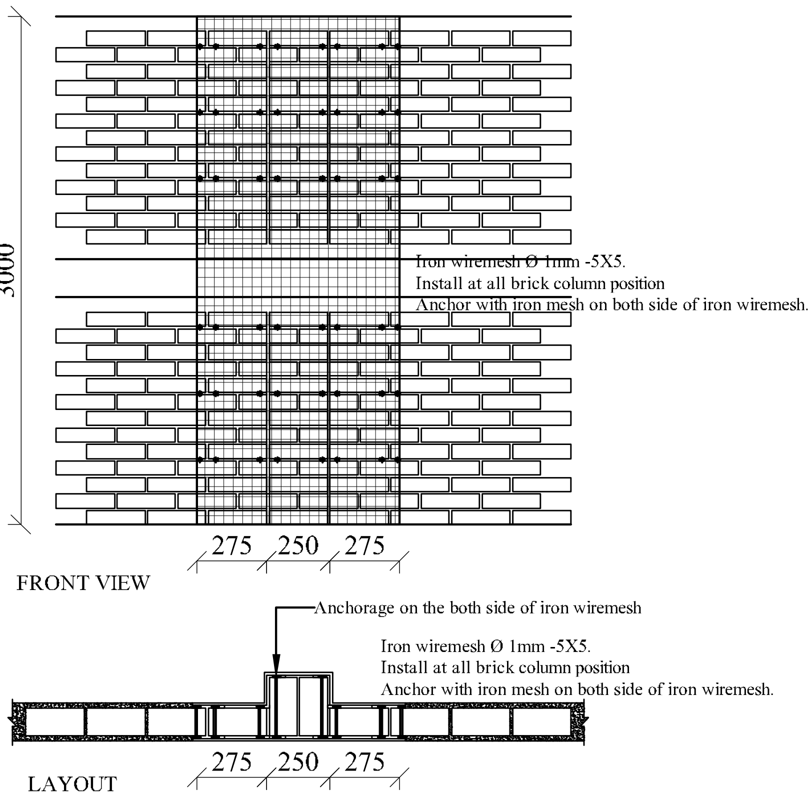
Detail of Column with Iron Wiremesh Reinforcement



2


3000
FRONT VIEW
275
250
275
Anchorage on the both side of iron wiremesh
Anchorage on the both side of iron wiremesh
Brick Column
Brick Wall
800

3


3000
Iron wiremesh Ø 1mm -5X5.
Install at all brick column position
Anchor with iron mesh on both side of iron wiremesh.
275
250
275
FRONT VIEW
Anchorage on the both side of iron wiremesh
Iron wiremesh Ø 1mm -5X5.
Install at all brick column position
Anchor with iron mesh on both side of iron wiremesh.
275
250
275
LAYOUT

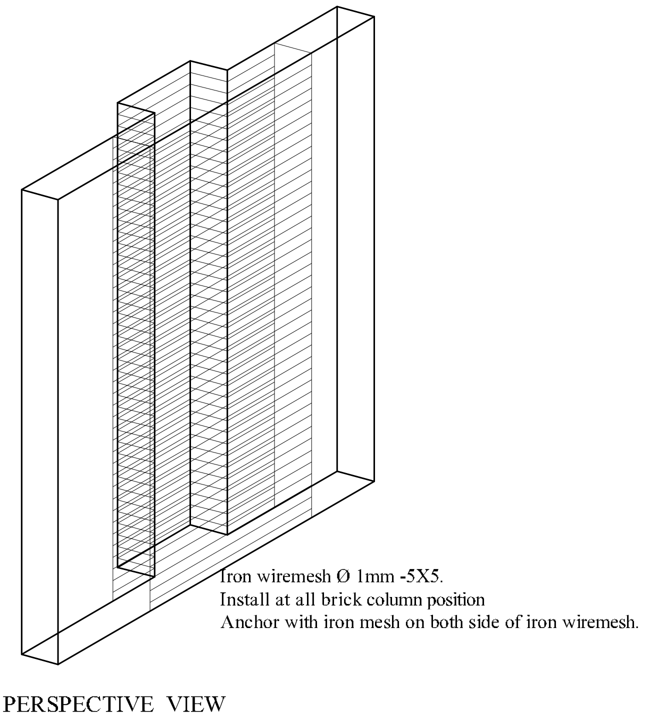
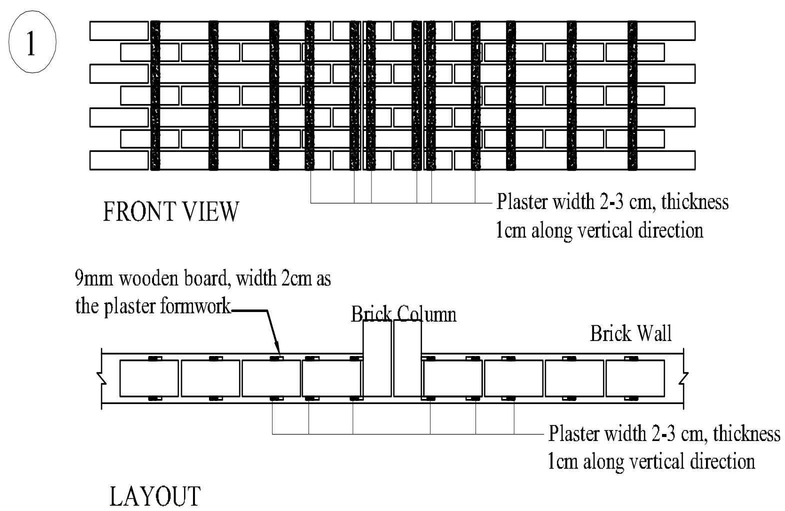
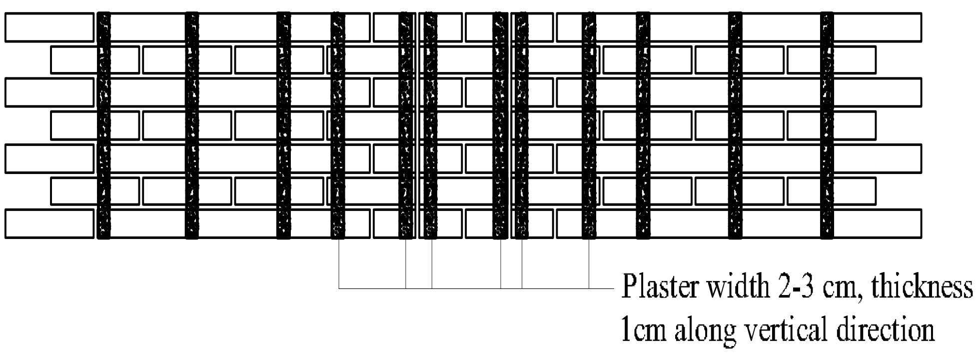


Figure B-4: Retrofitting of SDN Padasuka II (Courtesy of PT Teddy BoenKonsultan)

Detail of Beam with Iron Wire mesh Reinforcement





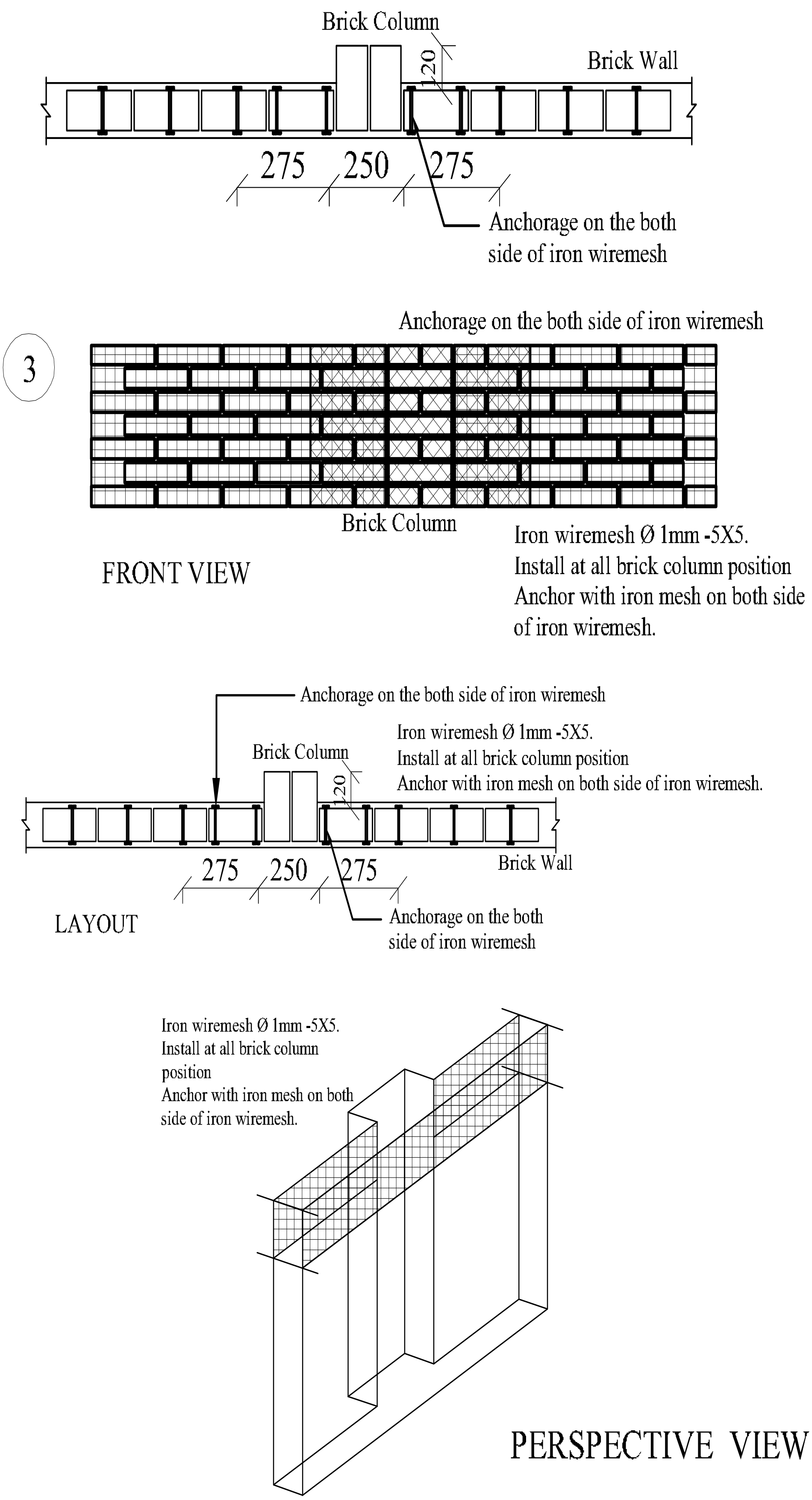
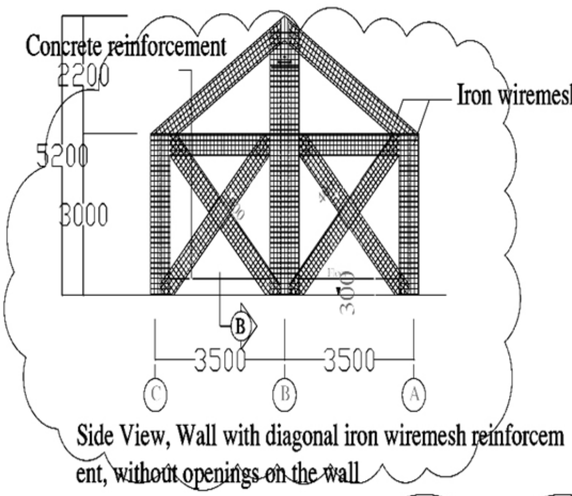
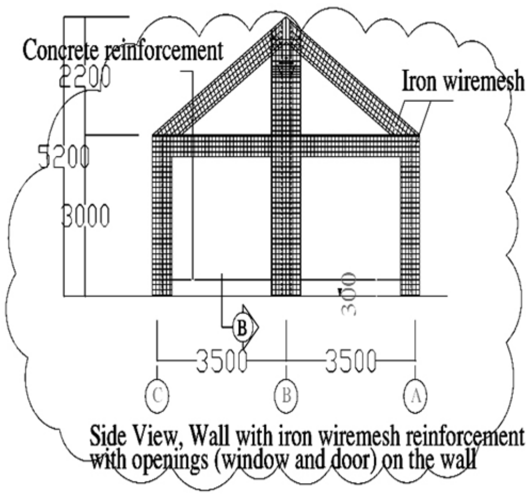
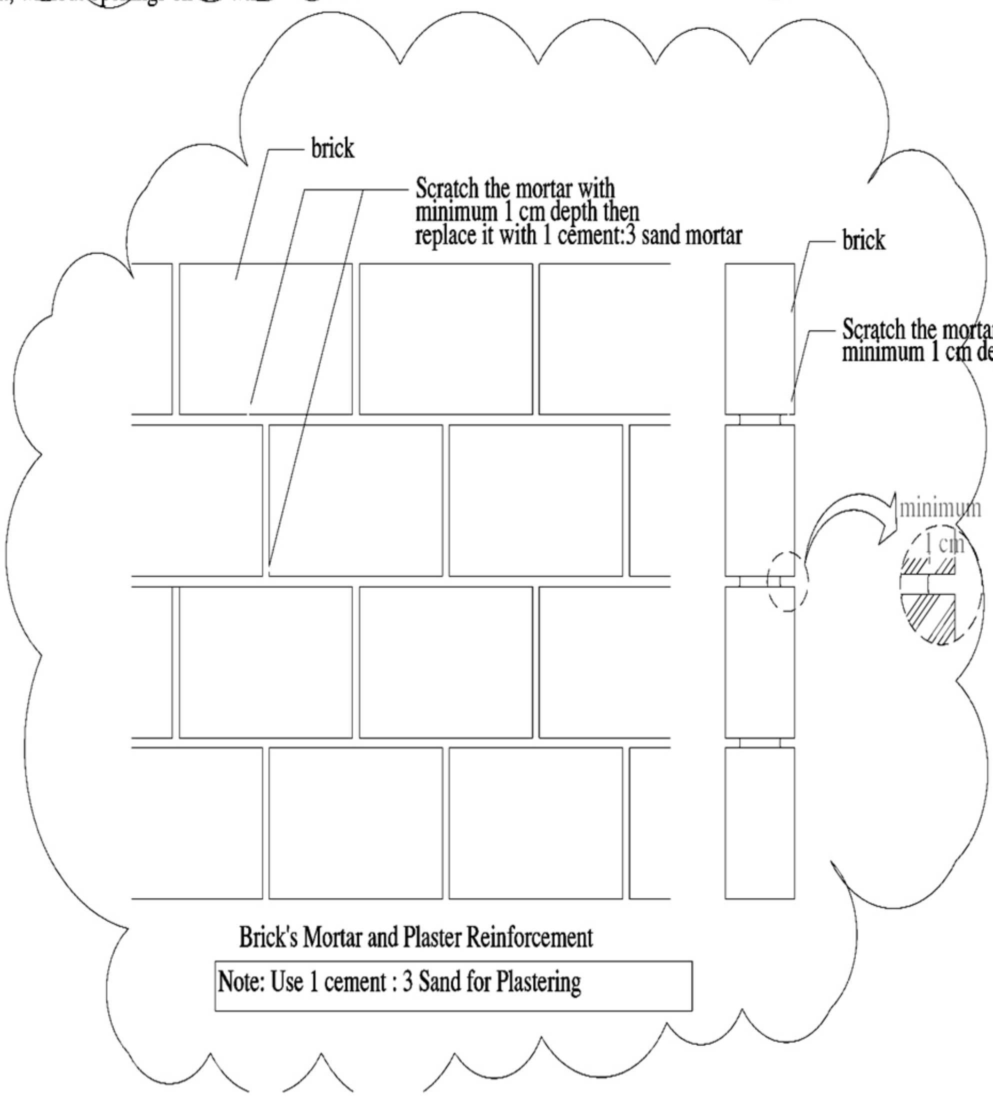


Figure B-5: Retrofitting strategy for beams at SDN Padasuka II

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

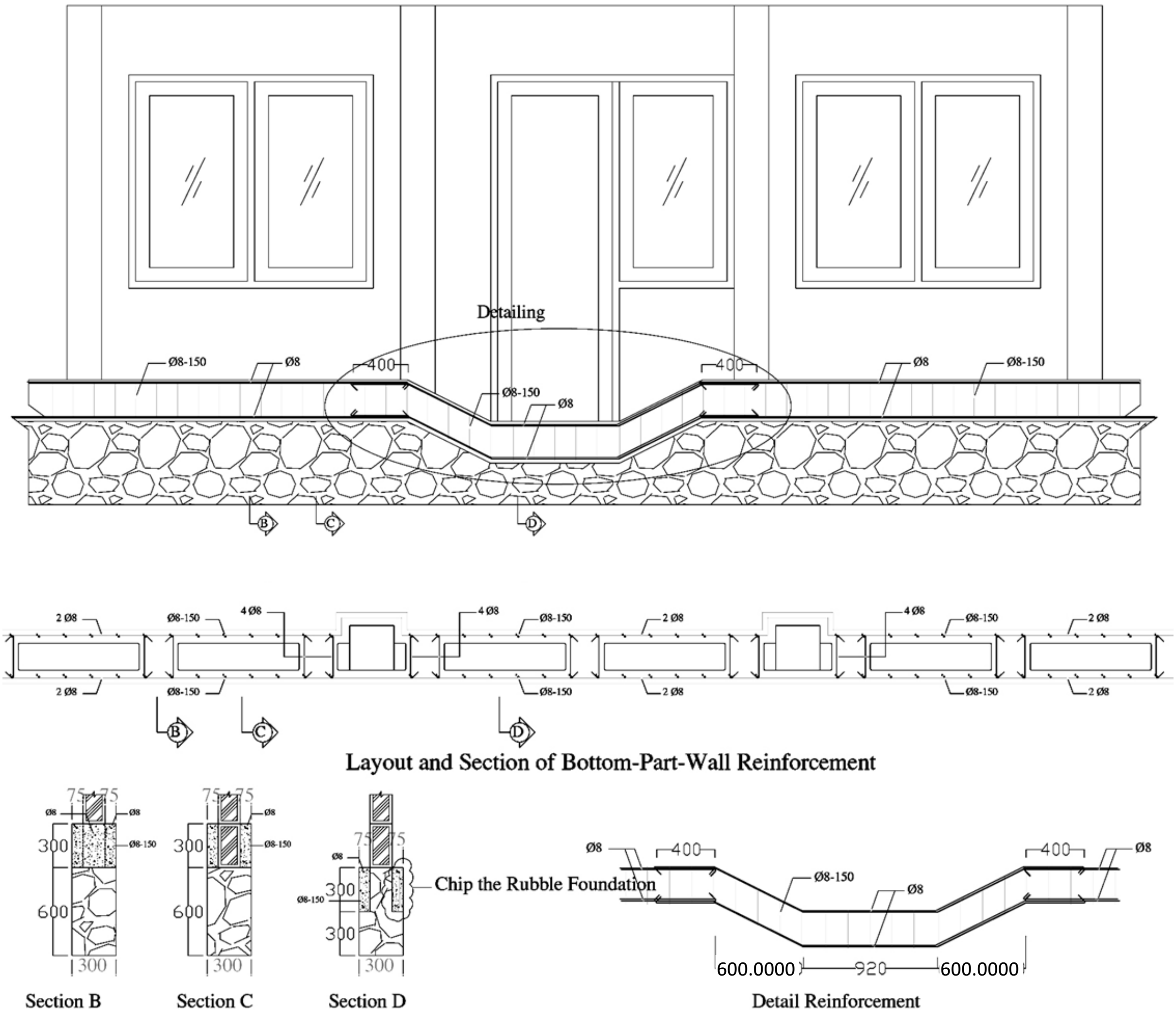


Figure B-7: Retrofitting of SDN Padasuka II
(Courtesy of PT Teddy Boen Konsultan) (cont'd)

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.2.4.3.1POST-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

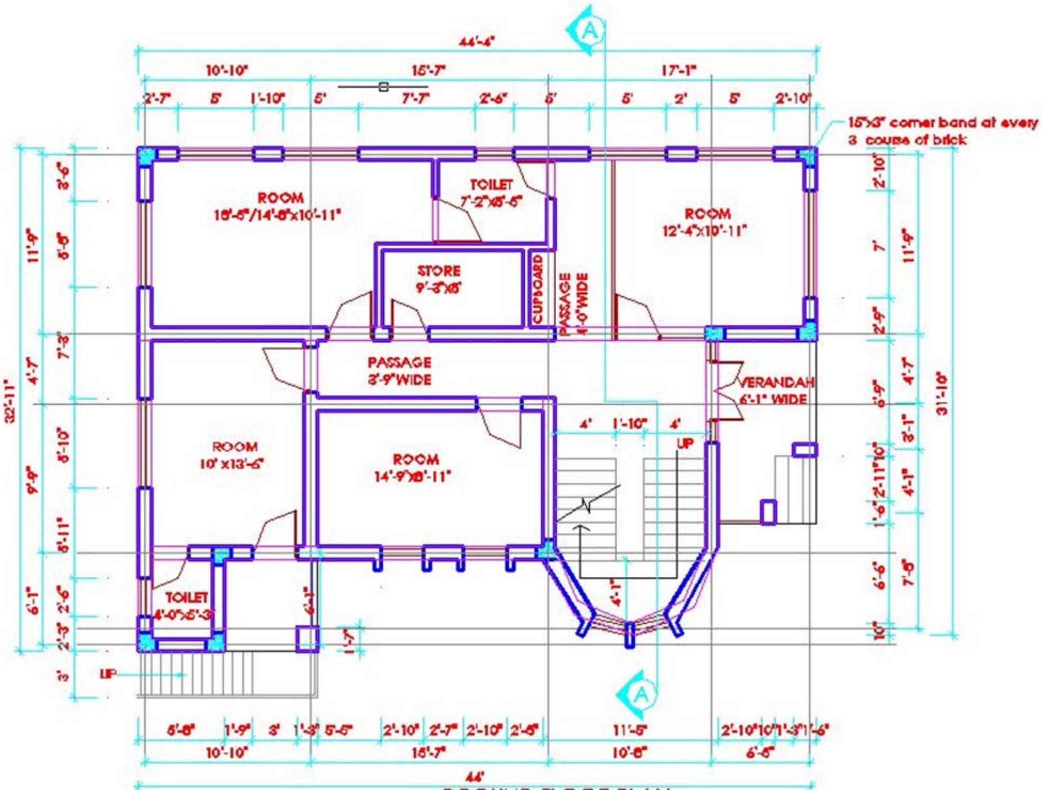
Figure B-19: Crack at wall

Figure B-20: Crack at Parapet wall

Figure B-21: Crack at Plinth Protection.

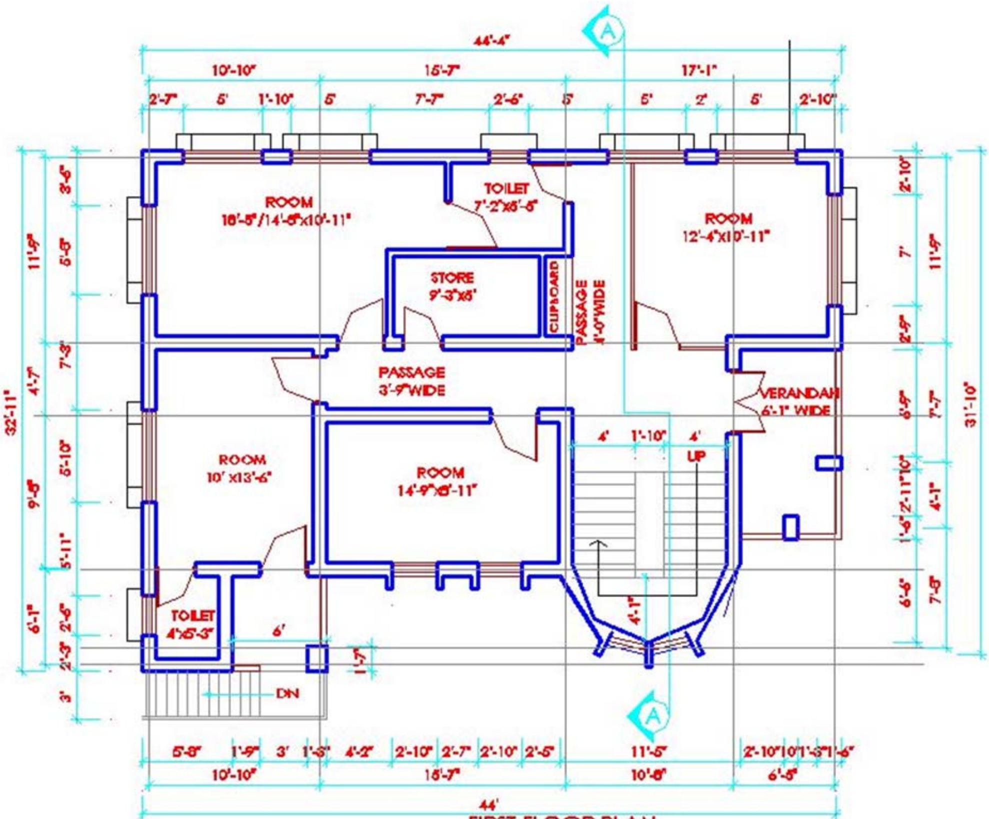
Figure B-22: Vertical crack on wall due to pipe duck

B.3.3 BUILDING DRAWINGS



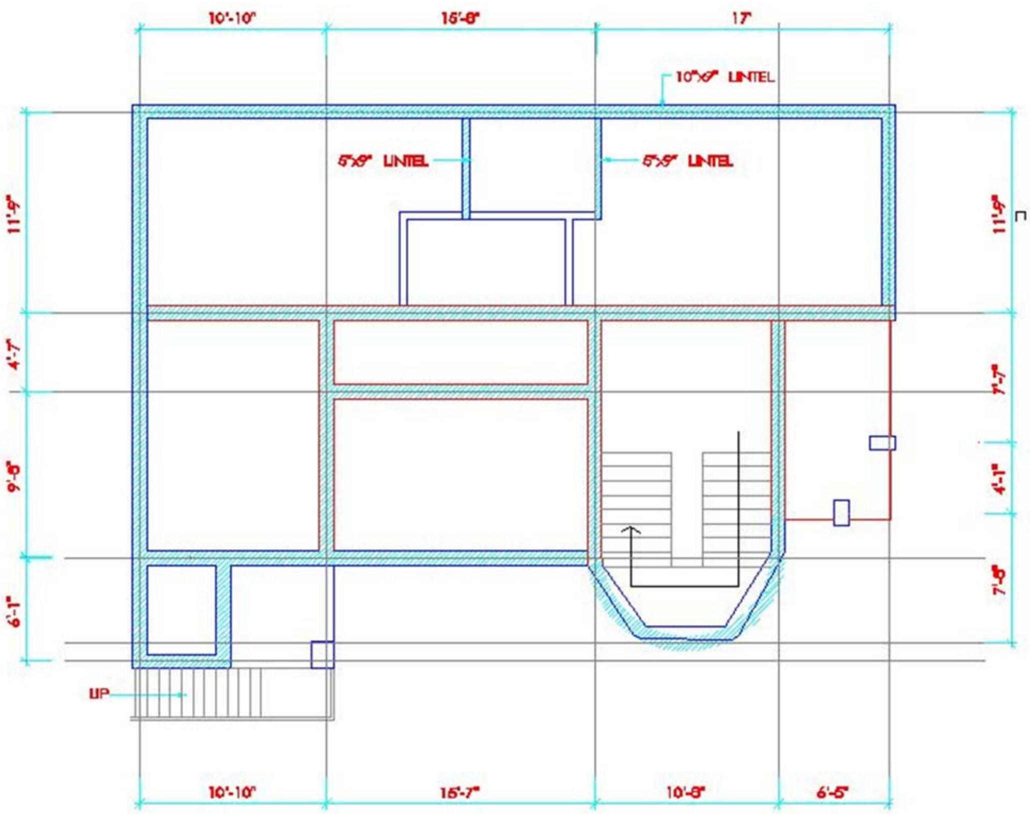
GROUND FLOOR PLAN

AREA=1290.56 SQ.FT.

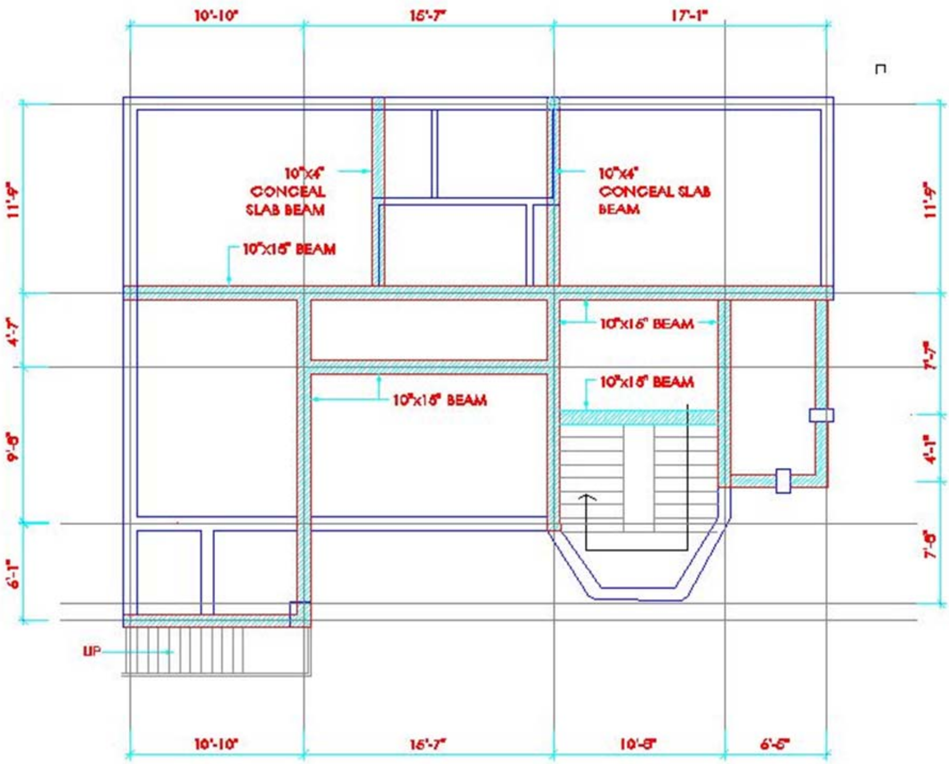


FIRST FLOOR PLAN

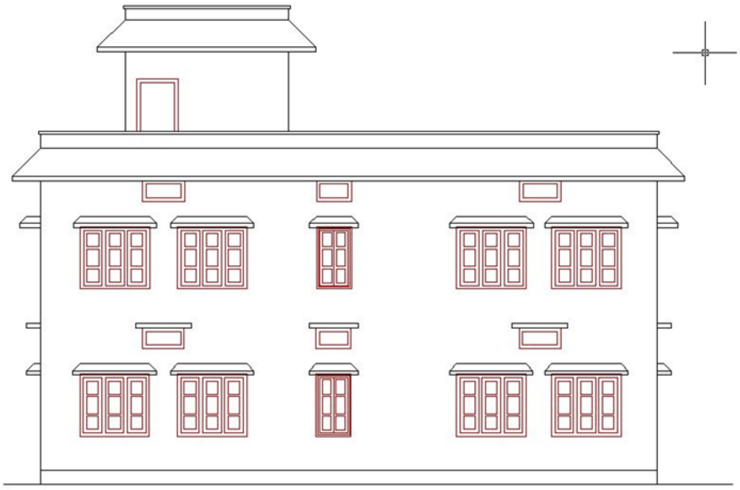
AREA=1290.56 SQ.FT.



LINTEL LAYOUT PLAN

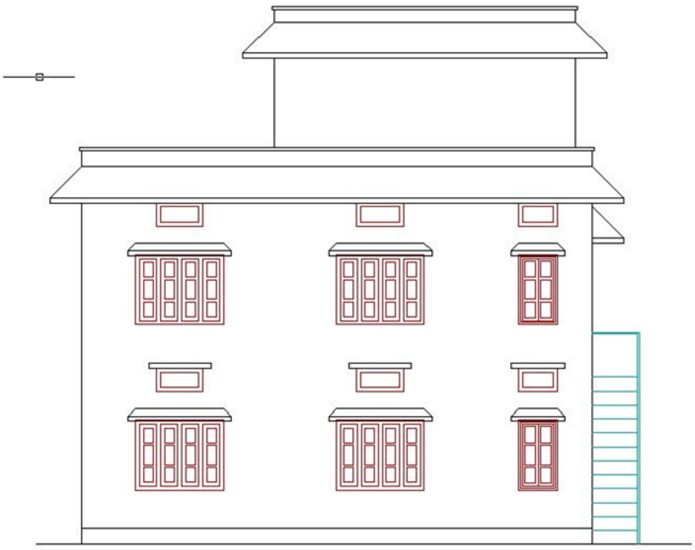


BEAM LAYOUT PLAN



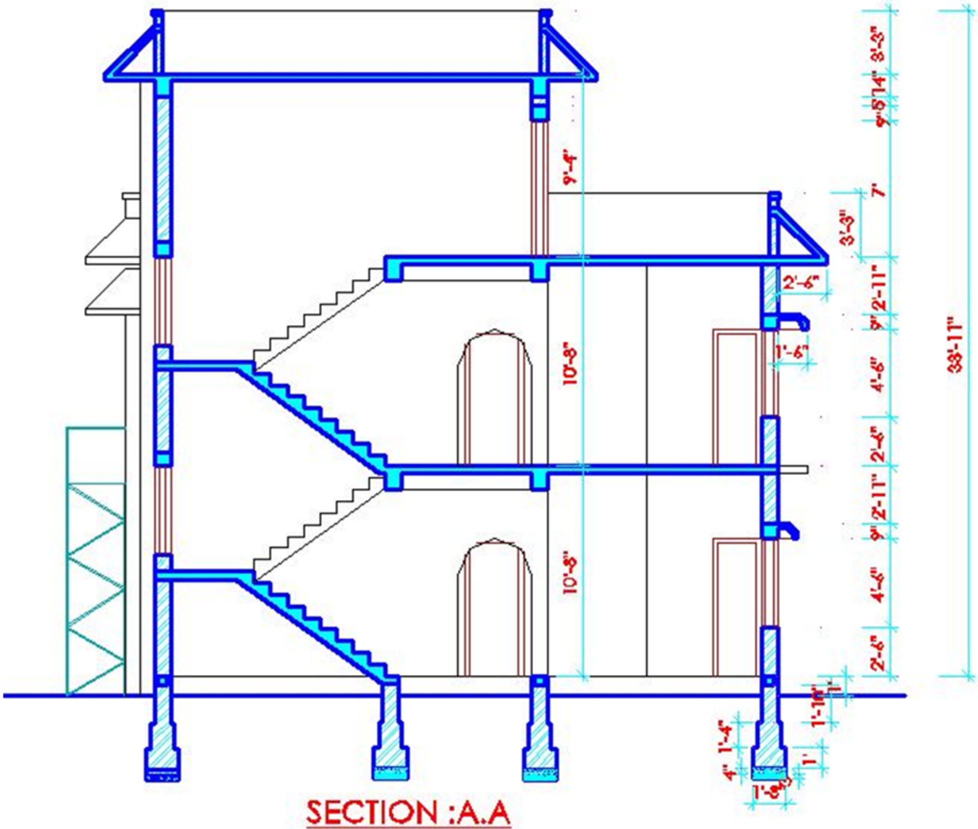
SIDE VIEW-1

FRONT VIEW



BACK VIEW

SIDE VIEW-2



SECTION :A.A

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:

Influence of Different Vulnerability Factors to the Structureof Field Office Building

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).	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. 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. Brick in Mud: These are the brick masonry buildings with fired bricks in mud mortar
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 (41/2") 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).

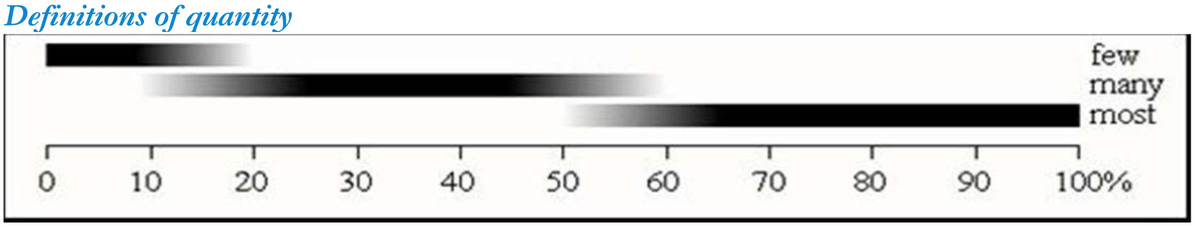
Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage)

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.

ANNNEX 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
dx = 13.67m

dz = 10.238m

$T_{ax} = \frac{0.09h}{\sqrt{dx}}$

$= \frac{0.09 \times 9.75}{(13.67)^{0.5}}$

= 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 , $\frac{S_a}{g}$ = 2.5 for medium soil (IS :1893(Part 1) : 2002

Z = 0.36

I = 1.5 (6.4.2, IS 1893 (Part 1) 2002)

$\frac{S_a}{g}$ = 2.5 (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

Base shear = $V_b = m_1 A_h W$

= 0.67 × 0.45 × 1467.92

= 442.577 KN

F.2 CALCULATION OF STOREY SHEAR

Vbz = 442.577 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
1st 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**
1st 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 FIELDOFFICEBUILDING .Kathmandu, Nepal			
Building	OFFICE	**Date of survey**	22nd 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

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.
The building contains a load path except the top floor.

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.
The building is two and half storey. Top floor of the building does not meet this criterion . This may suffer stress concentration

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.
Only top floor does not meet this criterion.

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.
Only top floor does not meet this criterion.

C NC 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.

C **NC** 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.

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.
Only top floor does not meet this criterion.

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.
Deterioration of concrete is not seen.

C NC N/A NK — MASONRY UNITS: There shall be no visible deterioration of masonry units.
Deterioration of masonry is not seen.

C NC 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.

C**NC** 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*
C**NC** 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 slopped roof, gable band shall be provided to the building. *Flat roof available.*
C NC **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.*
C**NC** 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

C**NC** N/A NK	OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls shall be less than 15% of the wall length.

C**<u>NC</u>** 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.

<u>C</u>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

CNC N/A **<u>NK</u>** | DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragms openings larger than 50% of the building width in either major plan dimension.

CNC**<u>N/A</u>** 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.

CNC**<u>N/A</u>** NK | LATERAL RESTRAINERS: For flexible roof and floor, each joists and rafters shall be restrained by timber keys in both sides of wall.

CONNECTIONS

C**<u>NC</u>** 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.

<u>C</u>NC 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.

C**<u>NC</u>** 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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25) *SEISMIC EVALUATION AND REHABILITATION OF MASONRY BUILDING, GIAN MICHELE CALVI AND GUIDO MAGENES.*

SEISMIC RETROFITTING GUIDELINES OF BUILDINGS IN NEPAL

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