SEISMIC ANALYSIS AND RETROFIT OF EXISTING MASONRY BUILDINGS IN NEPAL

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Abstract: **The Nepal(Gorkha) earthquake (Mw=7.8) of April 25, 2015 caused widespread damages that took many lives, ruptured lands and damaged many buildings, heritages and monuments. The disaster turned out to be perilous for many non-engineered masonry structures in rural part of the country. Nepal being a developing country, most of the rural houses are non-engineered. The seismic vulnerability of these types of buildings is presumed to be very high. As Nepal, lying in a very high seismic zone, an immediate attention is imperative to improve the seismic resilience of such buildings.**

To qualify the status and seismic resistivity of these buildings a typical building is to be taken and computer modeled. The modeled building is to be simulated at various magnitude of earthquake and the effects are to be noted and studied. This simulation shows the seismic resistivity and failure of the buildings at various points. These weak points are to be identified and retrofitted by using various available methods.

IndexTerms **- Seismic analysis, Retrofit, Damage Categorization**

I. INTRODUCTION

Nepal 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.

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

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

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

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

II. OBJECTIVE AND SCOPE

The objective of this study is to analyze seismic vulnerability of existing buildings and design methodology for use in the seismic evaluation and retrofit of the existing buildings in Nepal. Also this study helps reduce vulnerability of buildings thereby decreasing likelihood of loss of life and injury to the habitants of the buildings. This is accomplished by limiting the likelihood of damage and controlling the extent of damage in the building.

Also this study helps to find the feasibility of retrofitting instead of reconstruction.

III. LITRATURE REVIEW

Seismic retrofitting is the modification of existing structures to make them more resistant to seismic activity, ground motion, or soil failure due to earthquakes. This goal maybe achieved by adopting one of the following strategies like by reducing the seismic demands on members and the structures as a whole, By increasing the member capacities Stiffness, strength and ductility are the basic seismic response parameters taken into consideration while retrofitting. However, the choice of the technique to be applied depends on locally available materials and technologies, cost considerations, duration of the works and architectural, functional and aesthetic considerations/restrictions. Seismic retrofitting schemes can be either global or local, based on how many members of the structures they are used for. Global (Structural level) Retrofit methods include conventional methods (increase seismic resistance of existing structures) or non-conventional methods (reduction of seismic demand)

- Jacketing construction is the most preferred method of retrofitting that can be applied by the following techniques:
- Confinement with fibre reinforced polymers such as aramid fibres, carbon fibres and glass fiber reinforced composite.
- Confinement with external steel caging techniques.
- Confinement with ferrocement.

Cetin Sahin, 2014 - Seismic Retrofitting Of Existing Structures

In this research project a brief presentation about earthquake resistant design and the methodology about seismic evaluation and rehabilitation of existing structures is made. It also provides certain aspects of computer software modeling against seismic loads and shows the necessity of seismic upgrading in a steel moment-frame building.

The seismic evaluation process consists of investigating if the structure meets the defined target structural performance levels. The main goal during earthquakes is to assure that building collapse doesn't occur and the risk of death or injury to people is minimized and beyond that to satisfy post-earthquake performance level for defined range of seismic hazards. Also seismic evaluation is determined which are the most vulnerable and weak components and deficiencies of a building during an expected earthquake. The seismic rehabilitation process aims to improve seismic performance and correct the deficiencies by increasing strength, stiffness or deformation capacity and improving connections. Thus, a proposed retrofit implementation can be said to be successful if it results an increase in strength and ductility capacity of the structure which is greater than the demands imposed by earthquakes.

Performance-based design aimed to utilize performance objectives to determine acceptable levels of damage for a given earthquake hazard for new buildings or upgrade of existing buildings. These performance objectives can be such as limiting story drift, minimizing component damage etc. This study shows how to model a building in computer software and analyze its seismic resistance with linear methods and propose concentrically braced frame rehabilitation in order to increase the drift capacity. It also describes how the linear analysis may be followed by the pushover analysis in order to estimate the seismic resistance of retrofitted structure.

Case Studies: Nepal and Peru

In order to demonstrate some of the principles and strategies outlined in this Primer, two papers are included below which describe case study programs for the seismic strengthening of housing in Nepal and Peru. Both case studies focus on rural adobe housing but the lessons are prevalent across locations and construction types.

The first paper describes the development (testing and analysis) of a particular seismic retrofit technique followed by a pilot project for implementing that retrofit technique in rural communities. The implementation phase involved a training program for rural masons in Nepal, a public shake-table demonstration, and the retrofit of a house. This implementation model proved effective at reaching rural communities but highlighted that subsidies are required to incentivize the safeguarding of homes among low-income communities, and that the long-term utilization of taught retrofitting and construction techniques is not guaranteed.

The second case study examines this conclusion further by exploring some of the technical, financial, and social challenges faced in the dissemination of seismic retrofit techniques to remote rural communities. A field investigation was carried out in Peru whereby sites of previous dissemination programs were visited and interviews were conducted with members of the affected communities and representatives of the organizations originally involved. This investigation highlighted that although programs must target communities directly, lessons taught to those communities are often lost over time.

Both case studies are useful in demonstrating the principles and strategies outlined in the Overview section of this Primer. They each present programs in which retrofit training has been used to also train in simple anti-seismic construction techniques to both build local capacity and change local construction practice. Technical excellence from around the world has been used to develop retrofit techniques which are simple enough to be applied by local masons or homeowners themselves. The retrofit techniques used are location specific, where the required materials and expertise are widely available in the local communities, and those communities are directly engaged through public demonstrations, training, and assisted self-build. Directly engaging masons is shown to be an effective way of transferring knowledge of earthquake-safe construction directly to those responsible for the construction, and the "cascade" model (training technicians to teach a larger number who then supervise construction) is an effective way of reaching the community while minimizing cost.

The two main conclusions that may be drawn from the following two case studies are:

- The buildings most at risk are built without engineering input, so retrofitting and construction techniques must be simple to apply and programs must target communities directly
- Lessons taught to communities are lost over time and so long-term intervention is essential

Giuseppe Oliveto and Massimo Marletta (2005)

Considered the retrofitting of buildings vulnerable to earthquakes and briefly described the main traditional and innovative methods of seismic retrofitting. Among all the methods of seismic retrofitting, particular attention was devoted to the method which was based on stiffness reduction. This method was carried out in practice by application of the concept of springs in series, which lead in fact to base isolation. One of the two springs in series represented the structure and the other represented the base isolation system. The enhanced resistance of the buildings to the design earthquake clearly showed the effectiveness of the method, while a generally improved seismic performance also emerged from the application.

2.1 Seismic Analysis

Seismic analysis is the assessment of the existing building for the probabilistic future earthquakes. The seismic analysis can be done for the buildings that are affected by earthquake and non-affected buildings. As non-affected buildings can be analyzed for followings:

- Design failure
- Change in codel provisions
- Change in building occupancy

Detail seismic analysis are done under following sections

Building Categorization

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

Damage Categorization

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

Table 1 European Macroseismic Scale (EMS) for damage categorizations

S. No	Damage Grade	Damage
1	**Grade 1**: Negligible to slight damage (No structural damage, slight non- structural damage)	• Hair line cracks in very few walls. • Fall of small pieces of plaster only. • Fall of loose stones from upper parts of buildings in very few cases
2	**Grade 2**: Moderate damage (slight structural damage, moderate non-structural damage)	• Cracks in many walls. • Fall of fairly large pieces of plaster. • Partial collapse of chimneys.
3	**Grade 3**: Substantial to heavy damage (moderate structural damage, heavy non-structural damage)	• Large and extensive cracks in most walls. • Roof tiles detach. Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).
4	**Grade 4**: Very heavy damage (heavy structural damage, very heavy non-structural damage)	• Serious failure of walls; partial structural failure of roofs and floors.
5	**Grade 5**: Destruction (very heavy structural damage)	• Total or near total collapse.

Modified Mercalli Intensity Scale (MMI Scale)

I. Very Weak Intensity

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

II. Slightly Weak Intensity

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

III. Weak Intensity

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

IV. Slightly Strong Intensity

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

V. Strong Intensity

- Can be felt and noticed by almost all people whether they are inside or outside structures. Many will be awakened from sleep and be surprised. Some may even rush out of their homes or buildings in fear. The vibrations and shaking that can be felt inside or outside structures will be very strong.
- Things that are hanged on walls would sway, shake or vibrate much more strongly and intensely. Plates and glasses would also vibrate and shake much strongly and some may even break. Small or lightly weighted objects and furniture would rock and fall off. Stationary vehicles would shake more vigorously.
- The shaking or vibrations on water or liquid surfaces in containers would be very strong which will cause the liquid to spill over. Plant or tree stem, branches and leaves would shake or vibrate slightly.

VI. Very Strong Intensity

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

VII. Damaging Intensity

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

VIII. Highly Damaging Intensity

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

IX. Destructive Intensity

- People would be forcibly thrown/fall down. Chaos, fear and confusion will be extreme.

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

X. Extremely Destructive Intensity

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

Probable Damage Grade of the Building at Different Intensities

Damage Grade for Different Classes of Buildings	MMI	VI	VII	VIII	IX	X
	Weak	DG2	DG3	DG4	DG5	DG5
	Average	DG1	DG2	DG3	DG4	DG5
	Good	-	DG1	DH2	DG3	DG4

Failure Modes

The relative severities of the various types of damage helps to relate life safety and the protection of historic building fabric. Identifying the failure modes 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.

Out-Of-Plane Failure

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

Gable End Wall Collapse

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

Figure 1 Gable End wall Collapse

Out of Plane Flexural Cracks and Collapse

Out-of-plane flexural cracking is one of the first crack types to appear in an adobe building during a seismic event. 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.

Mid Height Out-Of-Plane Flexural Damage

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

In-Plane Failure

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

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

Figure 1 In-Plane Failure

Diaphragm Failure

The failure of the diaphragm is a rare phenomenon in the event of seismic motion. Damage to the diaphragm never impairs its gravity load carrying capacity. Lack of tension anchoring produces a non-bending cantilever action at the base of the wall resulting from the push of diaphragm against the wall. The in plane rotation of the diaphragm ends and the absence of a good shear transfer between diaphragms and reaction of walls account for damage at the corners of the wall.

Corner Damage

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

Vertical Cracks at Corners

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

Diagonal Cracks at Corners

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

Combinations with Other Cracks or Preexisting Damage

A combination of diagonal and vertical cracks can result in an adobe wall that is severely fractured, and several sections of the wall may be susceptible to large offsets or collapse. The diagonal cracking at that location allows the cracked wall sections freedom to move outward. Corners may be more susceptible to collapse if vertical cracks develop and the base of the wall has already been weakened by previous moisture damage.

Cracks at Openings

Cracks occur at window and door openings more often than at any other location in a building. In addition to earthquakes, foundation settlement and slumping due to moisture intrusion at the base can also cause cracking. Cracks at openings develop because stress concentrations are high at these locations and because of the physical incompatibility of the adobe and the wood lintels. Cracks start at the top or bottom corners of openings and extend diagonally or vertically to the tops of the walls. Cracks at openings are not necessarily indicative of severe damage. Wall sections on either side of openings usually prevent these cracks from developing into large offsets. However, in some cases, these cracks result in small cracked wall sections over the openings that can become dislodged and could represent a life-safety hazard.

Intersection of Perpendicular Walls

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

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

3.1 Seismic Vulnerability Evaluation

Seismic Vulnerability Evaluation of earthquake damage buildings provide the primary lateral-force-resisting systems in the building. In this evaluation, buildings are observed in damage caused by the earthquake in terms of the loss in building performance capability.

Vulnerability Evaluation

The evaluation procedure is done when an earthquake causes damage to a building. Vulnerability evaluation can assess the damage effects, at least partially, through visual inspection augmented by investigative tests, structural analysis, and knowledge of the building construction by determining how the structural damage has changed structural properties.

Levels of Damage Assessment

- Windshield: Overall scope of damage
- Rapid : Assessment sufficient for most buildings
- Detailed: Closer assessment of difficult or complex buildings
- Engineering : Consultant engaged by owner

Seismic Assessment

Seismic assessment is the procedure of survey which concludes the status of the building as it is suitable to live in, can be retrofitted. Seismic assessment contains following evaluations:

Rapid Assessment (Visual Survey)

This is the first level of site inspection which gives the recent status of the building:

- Age of building
- Structural system
- Foundation Exploration
- Load Path
- Geometry
- Wall details
- Beam and column size
- Water proofing method
- Renovation of building
- Other structural system

Methodology for Rapid Visual Assessment

- Review available structural and architectural drawings
- Review of the design data
- interview with designer
- Inspection of the buildings
- Identification of vulnerability factors (As per FEMA 310)
- Determination of strength of the structural components
- Analysis of the structural system

- Building photographs

Preliminary Evaluation

A preliminary evaluation of building is carried out which involves broad assessment of its physical condition, robustness, structural integrity and strength of structure, including simple calculations. Approximate and quick checks can be used to compute the strength and stiffness of building components. The seismic base shear and storey shears for the building can be computed in accordance with IS1893 (Part1).

The preliminary evaluation is a quick procedure to establish actual structural layout and assess its characteristics that can affect its seismic vulnerability. It is a very approximate procedure based on conservative parameters to identify the potential earthquake risk of a building and can be used to screen buildings for detailed evaluation.

Site Visit

A site visit is to be conducted to verify available existing building data or collect additional data, and to determine the condition of the building and its components.

Building Configuration

Load Path

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

Redundancy

The number of lines of vertical lateral load resisting elements in each principal direction shall be greater than or equal to 2. In the case of moment/braced frames, the number of bays in each line shall be greater than or equal to 2.

Geometry

No change in the horizontal dimension of lateral force resisting system of more than 50% in a storey relative to adjacent stories, excluding penthouses and mezzanine floors, should be made.

Weak Storey

The strength of the vertical lateral force resisting system in any storey shall not be less than 70% of the strength in an adjacent storey.

Soft Storey

It is one in which the lateral stiffness is less than70 percent of that in the storey above or less than80 percent of the average lateral stiffness of the three storeys above.

Vertical Discontinuities

All vertical elements in the lateral force resisting system shall be continuous from the root to the foundation.

Mass

There shall be no change in effective mass more than 100% from one storey to the next. Light roofs, penthouses, and mezzanine floors need not be considered.

Torsion

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

Adjacent Buildings

The clear horizontal distance between the building under consideration and any adjacent building shall be greater than 4% of the height of the shorter building, except for buildings that are of the same height with floors located at the same levels.

Short Columns

The reduced height of a column due to surrounding parapet, infill wall, etc. shall not be less than five times the dimension of the column in the direction of parapet, in fill wall, etc. or 50% of the nominal height of the typical columns in that storey.

Strength Checks

The seismic base shear and storey shears for the building is computed in accordance with IS 1893 (Part1)

Table 2 Earthquake load calculation using seismic coefficient method

The design horizontal seismic coefficient	$Ah = \frac{ZISa}{2Rg}$ **Where,** **Z = Zone factor** **I = Importance factor**

	R = Response reduction factor **Sa/g= Average response acceleration coefficient**
The total design lateral force or design seismic base shear (VB) along any principal direction is determined by using the expression	VB = AhW Where, W = Seismic weight of the building
The approximate fundamental natural period of vibration (Ta) in seconds, of all other buildings, including moment-resisting rame buildings with brick infill panels, may be estimated by the empirical expression	$Ta = \frac{0.09h}{\sqrt{d}}$ Where, h = Height of Building in meter d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force

Table 3 Base shear distribution and shear stress calculation

The design base shear (VB) shall be distributed long the height of the building as per the expression	$Qi = Vb\frac{Wihi2}{\Sigma Wihi2}$ **Where,** **Qi= Design lateral force at floor i** **Wi = Seismic weight of floor i** **hi = height of floor i , from the base**

Detailed Evaluation

A detailed evaluation is required unless results of preliminary evaluation are acceptable. The detailed evaluation procedure is based on determining the probable strength of lateral load resisting elements and comparing them with the expected seismic demands.

Building Components Conditions

Deterioration of Concrete

There should be no visible deterioration of the concrete or reinforcing steel in any of the vertical or lateral force resisting elements.

Cracks in Boundary Columns

There shall be no existing diagonal cracks wider than 3 mm in concrete columns that encase masonry infills.

Masonry Units

There shall be no visible deterioration of masonry units.

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.

Cracks in Infill Walls

There shall be no existing diagonal cracks in infill walls that extend throughout a panel, are greater than 3 mm, or have out-of-plane offsets in the bed joint greater than 3 mm.

Condition of the Building Materials

An evaluation of the present day strength of materials can be performed using on-site nondestructive testing and laboratory analysis of samples taken from the building. Field tests are usually indicative tests and therefore should be supplemented with proper laboratory facilities for accurate quantitative results.

Tests for Detailed Vulnerability Assessment

Following Non Destructive Test (NDT) gives the detailed building condition of the building:

Sounding Test

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

Rebound Hammer Test

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

Rebar Detection Test

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

In-Situ Testing In-Place Shear

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

Analytical Process

Introduction

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:

Simplified Linear Analysis

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. This method can be applied, if all of the following conditions are met:

- The building doesn't have any irregularity
- The building is within the specified limit of height-to-thickness ratio
- Opening in the building walls meet the conditions set in building code

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. This method is applicable, if following conditions are met:

- The building's irregularities are within the limit
- The building is within the specified limit of height-to-thickness ratio.

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. For all other buildings, a detailed non-linear analysis should be performed. It is advisable to check performance of structure above using, at least, non-linear static analysis if possible.

Analysis Methods

As per FEMA 356 specifies four procedures which can be used to analyze an existing building

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

Linear Static Procedure

FEMA 356 gives an equation for the approximate the fundamental period for URM buildings with flexible diaphragms

$$T = 0.0254(0.078\Delta d)^{0.5}$$

Where, Δd 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 = C1C 2C3CmSaW

Where:

V = Pseudo lateral load

C1 = Modification factor relating expected inelastic displacements to the calculated elastic response.

C2 = Modification factor for stiffness degradation and strength deterioration (1.0 for LSP)

C3 = Modification factor to account for increased displacements due to P-Delta effects

Cm = Effective mass factor to account for higher mode mass participation (1.0 for URM)

Sa = 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 above equation, the pseudo-lateral load is calculated for each span of the building and for each floor. It is then distributed to the vertical seismic-resisting elements (walls) according to tributary area. Forces in the diaphragm can then be calculated using these results.

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

KQcL ≥ QUF

Where:

K = Knowledge factor

QCL = Lower-bound strength of component

QUF = 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.

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:

mKQCE ≥ QUD

Where:

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

QCE = Expected strength of component

QUD = Deformation-controlled design action

The "m" factor is obtained from FEMA 356and depends on the failure mode (only for deformation-controlled mechanisms) and the performance level of the building.

Linear Dynamic Procedure

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.

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.

Nonlinear Static Procedure

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.

Alternative Approach for Analysis

Alternative approach for analysis gives the alternative way of analysis of existing buildings. The alternative approach for analysis includes the followings methods:

Performance Based Behavior of Masonry

Expected post-earthquake damage state of target building performance levels that can be used to assess an existing building. The main performance levels are as follows:

- Operational (O)- Backup utility services maintain functions; very little damage (S1+NA)
- Immediate occupancy (IO) - The building remains safe to occupy, any repairs are minor. (S1+NB)
- Life-safety (LS) - Structure remains stable and has significant reserve capacity, hazardous non-structural damage is controlled. (S3+NC)

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

These performance levels are self-explanatory and based on the desired condition of structural and architectural components in the building after an earthquake. These performance levels are combined with the earthquake hazard level of the site to obtain the rehabilitation objective.

In Plane Properties of Unreinforced Masonry Wall

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 below.

$$Vme = \frac{0.75(0.75Vte + \frac{Pce}{An})}{1.5}$$

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}

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

$$Vto = \frac{Vtest}{Ab} - Pd + l$$

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

Pd+l = Stress due to gravity loads at the test location

Out of Plane Properties of URM Wall

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

Out-of-Plane Anchorage to Diaphragms

Walls are 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 expression below, 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.

$$F_p = S_{xs}wX$$

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

SXS= Spectral response acceleration parameter at short periods for the selected hazard level and damping adjusted for site class

X= Weight of the wall tributary to the anchor

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 below.

$F_p = S_{xs} w \chi$

Fp = 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

SXS = 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

Stiffness

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

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

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.

In Plane Properties of Reinforced Masonry Wall

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.

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.

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, fye, shall be taken as the expected modulus of elasticity, Ese, 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, fye.
2. Tensile strength of masonry shall be neglected when calculating the flexural strength of a reinforced masonry cross-section.
3. Flexural compressive stress in masonry shall be assumed to be distributed across an equivalent rectangular stress block.
4. Masonry stress of 0.85 times the expected compressive strength, fme, 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 crosssection. 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

Shear strength of walls and piers

The lower bound shear strength of an RM wall or pier, VCL, shall not exceed the value computed in accordance with above equations. For intermediate values of M/Vdv, interpolation shall be used.

For M/Vdvless than 0.25:

$$V_{cl} \leq \square 6\sqrt{f_m'}\, A_n$$

For M/Vdvless than 1.00:

$$V_{cl} \leq \square 4\sqrt{f_m'}\, A_n$$

where:

An = Area of net mortared/grouted section
f_m = Lower bound compressive strength of masonry
M = Moment on the masonry section
V = Shear on the masonry section
dv = Wall length in direction of shear force

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.

Vertical compressive strength of walls and piers

Lower bound vertical compressive strength of existing RM wall or pier components shall be determined using Equation:

$$Q_{cl} = P_{cl} = 0.8\,[0.85\, f'_m\,(A_n - A_s) + A_s\, f_y]$$

Where:

f'm = Lower bound masonry compressive strength
fy = Lower bound reinforcement yield strength

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 below. 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 below.

Table 4 Lower Bound Masonry Properties

Property Good Fair Poor	Masonry Condition		
	Good	**Fair**	**Poor**
Compressive Strength (fm)	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
Masonry condition shall be classified as good, fair, or poor as defined in this standard.			

Table 5 Expected Strength Masonry Properties

Property	Factor
Compressive Strength (fme)	**1.3**
Elastic Modulus in Compression2	**-**
Flexural Tensile Strength	**1.3**

The expected elastic modulus in compression shall be taken as 550fme, where fme is the expected masonry compressive strength.

Repair, Restoration and Retrofitting

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

The actions will include the following:

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

Restoration

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.

Retrofitting (Seismic Strengthening)

Retrofitting involve actions for upgrading the seismic resistance of an existing building so that it becomes safer under the occurrence of probable future earthquakes. 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. The actions will include the following:

i) Modification of roofs
ii) Substitution or strengthening of floors
iii) Modification in the building plan
iv) Strengthening of walls including provision of horizontal and vertical bands or belts, introduction of through or header stones in thick stone walls, and injection grouting
v) Adding to the sections of beams and columns by casing or jacketing
vi) Adding shear walls or diagonal bracings
vii) Strengthening of foundations

The comparison of Repair and Retrofit can be made as this:

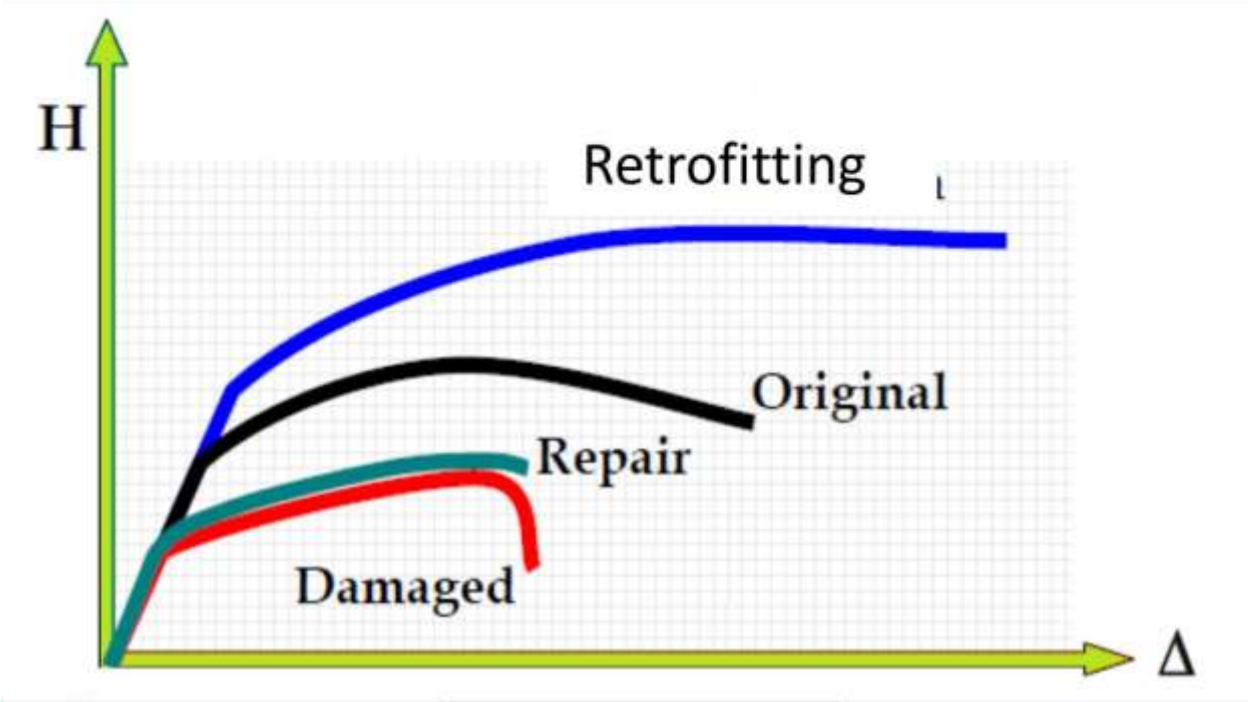


Figure 2 Comparison of Repair and Retrofit

Retrofitting Of Different Elements

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:

1. Increasing the lateral strength in one or both directions, by reinforcement or by increasing wall areas or the number of walls and columns.
2. 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.
3. 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.
4. 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.
5. 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.
6. Openings in load bearing walls should be restricted as shown in Figure below.

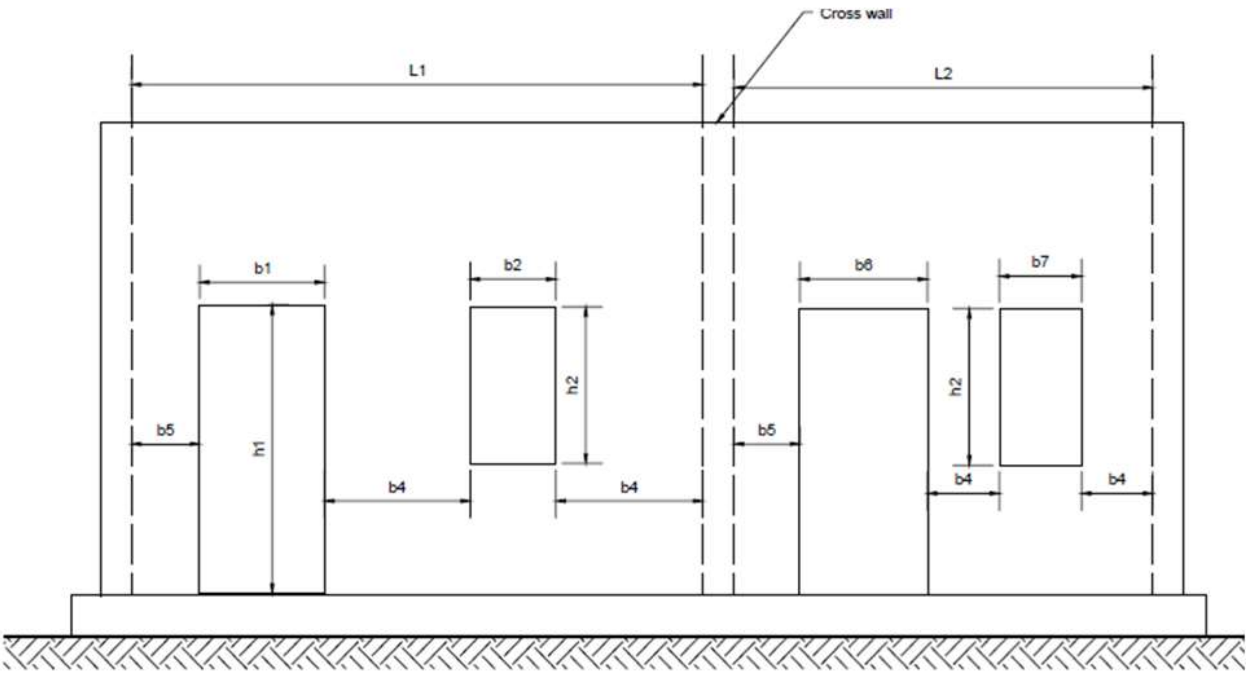


Figure 3Openings in Load Bearing Walls

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.

Strengthening Of Floor/Roof

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.

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.
FEMA 356 suggests Diaphragms and their connections to vertical elements providing lateral support shall comply with the following requirements.

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.

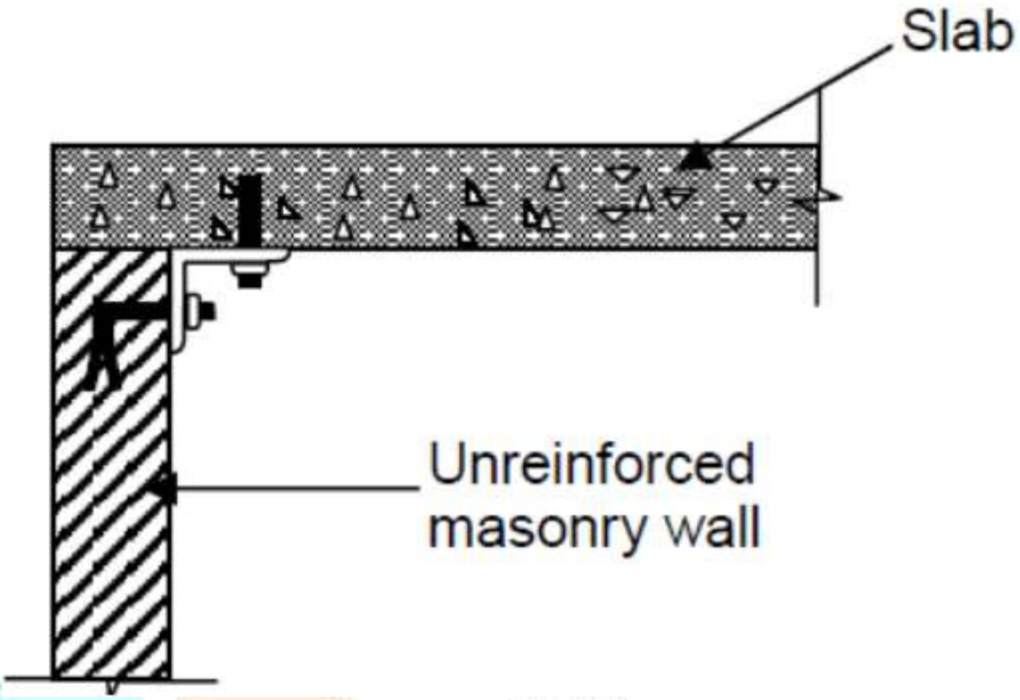


Figure 4 RCC Slab and wall anchorage

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.

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. 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.

Stiffening the Sloping Roof Surface

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 Notes 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. The important point in retrofitting is the provision of seismic belts just below eave level and the gable level.

Notes:

- The mesh should be continuous with 200mm overlap at the corner or elsewhere.
- Using galvanized binding wire, tie up the roof rafters with the nails of the eave level belt before applying the plaster over the mesh.
- 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.

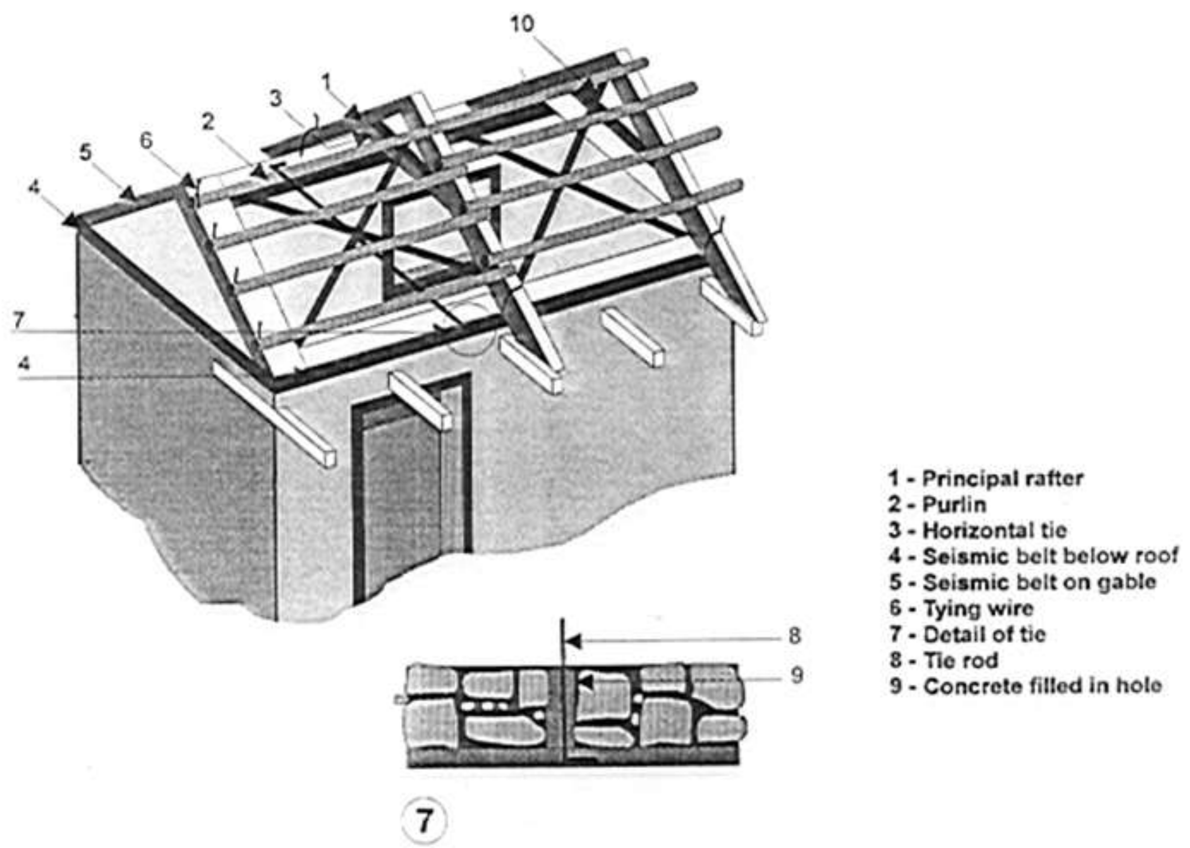


Figure 5 Sloping Roof surface stiffening

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 anddesigners shall check it before starting the construction.

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.

Control on Door and Window Openings in Masonry Walls

Infill Openings

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.

Seismic Belts around Door / Window Opening

The jambs and piers between window and door openings require vertical reinforcement as in Table below:
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.

No. of Storeys	Storeys	Reinforcement		
		Single Bar. mm	Mesh	
			No of longitudinal wires in the mesh.	Width of the micro concrete belt
One	Ground	10	20	500

Two	First	10	20	500
	Ground	12	28	700
Three	Second	10	20	500
	First	12	28	700
	Ground	12	28	700

Stiffening the Flat Wooden Floor / Roof

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 used.

Shear Walls

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.

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.
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

Advanced Techniques for Retrofitting Of Structures

Energy Dissipation Devices for Earthquake Resistance (Damper)

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.

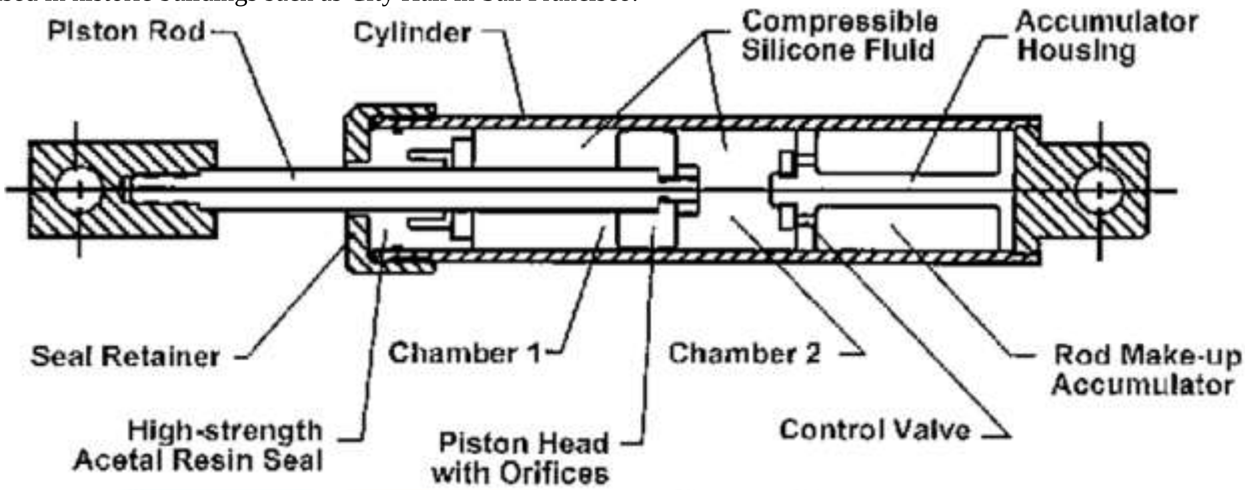


Figure 6 Energy Dissipating Damper

Commonly used types of seismic dampers are:

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

Base Isolation

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

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

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

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

Non-Metallic Fibre Composites / Fibre Reinforced Composites (FRC)

Fibre Reinforced Polymer (FRP) composites comprise fibres of high tensile strength within a polymer matrix such as vinylester or epoxy. FRP composites have emerged from being exotic materials used only in niche applications following the Second World

War, to common engineering materials used in a diverse range of applications such as aircraft, helicopters, spacecraft, satellites, ships, submarines, automobiles, chemical processing equipment, sporting goods and civil infrastructure. The role of FRP for strengthening of existing or new reinforced concrete structures is growing at an extremely rapid pace owing mainly to the ease and speed of construction, and the possibility of application without disturbing the existing functionality of the structure. FRP composites have proved to be extremely useful for strengthening of RCC structures against both normal and seismic loads. The FRP used for strengthening of RC structures can be mainly categorized as:

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.

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).

Different Techniques for Strengthening

Application of Steel Wire Mesh/Ferrocement Plating in Masonry Building

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, ferrocement planting be provided on both faces.

Details of retrofitting elements Ferro-Cement Planting

1. Mark the height or width of the desired planting based on the weld mesh number of longitudinal wires and the mesh size.
2. Cut the existing plaster at the edge by a mechanical cutter for neatness, and remove the plaster.
3. Rake the exposed joints to a depth of 20 mm. Clean the joints with water jet.
4. 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.
5. 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.
6. 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.

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. 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.

Methodology for Grouting Of Cracks

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

1. Remove the plaster in the vicinity of crack exposing the cracked bare masonry.
2. Make the shape of crack in the V-shape by chiseling out.
3. Fix the grouting nipples in the V-groove on the faces of the wall at spacing of 150-200 mm c/c.
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.
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.
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.
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.
8. After injection grouting through all the nipples is completed, replaster the finish the same.

Major Crack (Crack width more than 5.0mm)

1. Remove the plaster in the vicinity of crack exposing the cracked bare masonry.
2. Make the shape of crack in the V-Shape by chiseling out.

3. Clean the crack with compressed air.
4. Fix the grouting nipples in the V-groove in both faces of the wall at spacing of 150-200 mm c/c.
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.
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.
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.
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.
9. After injection grouting through all the nipples is completed, replaster the surface and finish the same.

Jacketing

Jacketing consists of covering the wall surface with a thin layer of reinforced mortar, microconcrete, or shotcrete overlays interconnected by means of through-wall anchors. 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.

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 m2.
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. 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.

Seismic Band and Belt

Seismic Bands (Ring Beams)

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. 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. 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. 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.

Vertical Reinforcement at Corners and the Junctions of Walls

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. 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.

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. The transverse wire in the mesh could be at spacing up to 150 mm.

Install Vertical Bar in a Corner

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.
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.

Foundation Restrenghthening / Rehabilitation

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. 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 finegrained 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.

Shallow Foundation Rehabilitation

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

1. New isolated or spread footings may be added to existing structures to support new structural elements such as shear walls or frames.
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.
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.
4. Uplift capacity may be improved by increasing the resisting soil mass above the footing.

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.

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.

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.
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. 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.
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.

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.

Analysis And Design Of A Residential Building Using - Strength Based Method

The sample building of following building description is done using strength based method.

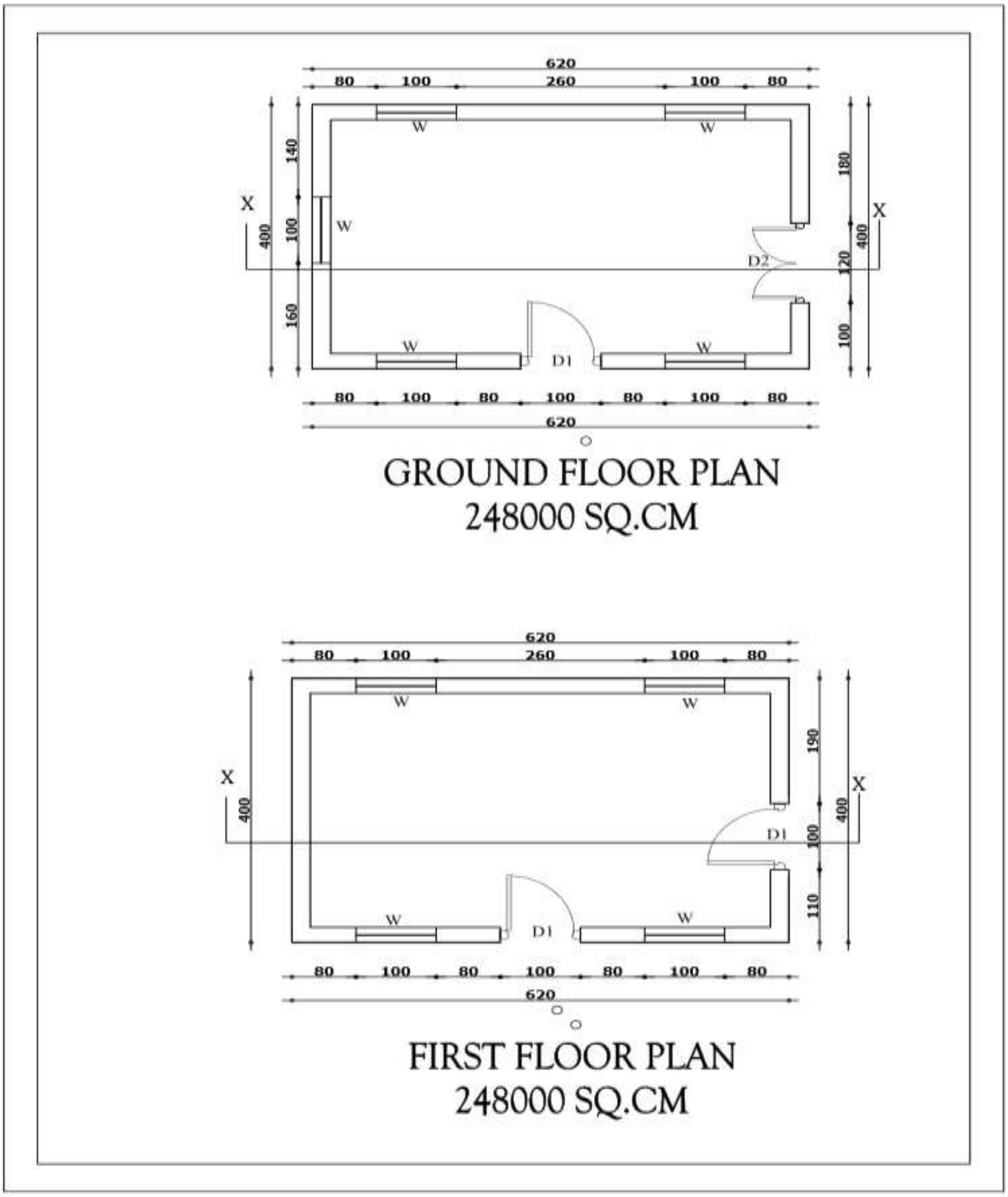


Figure 7 Building Plan

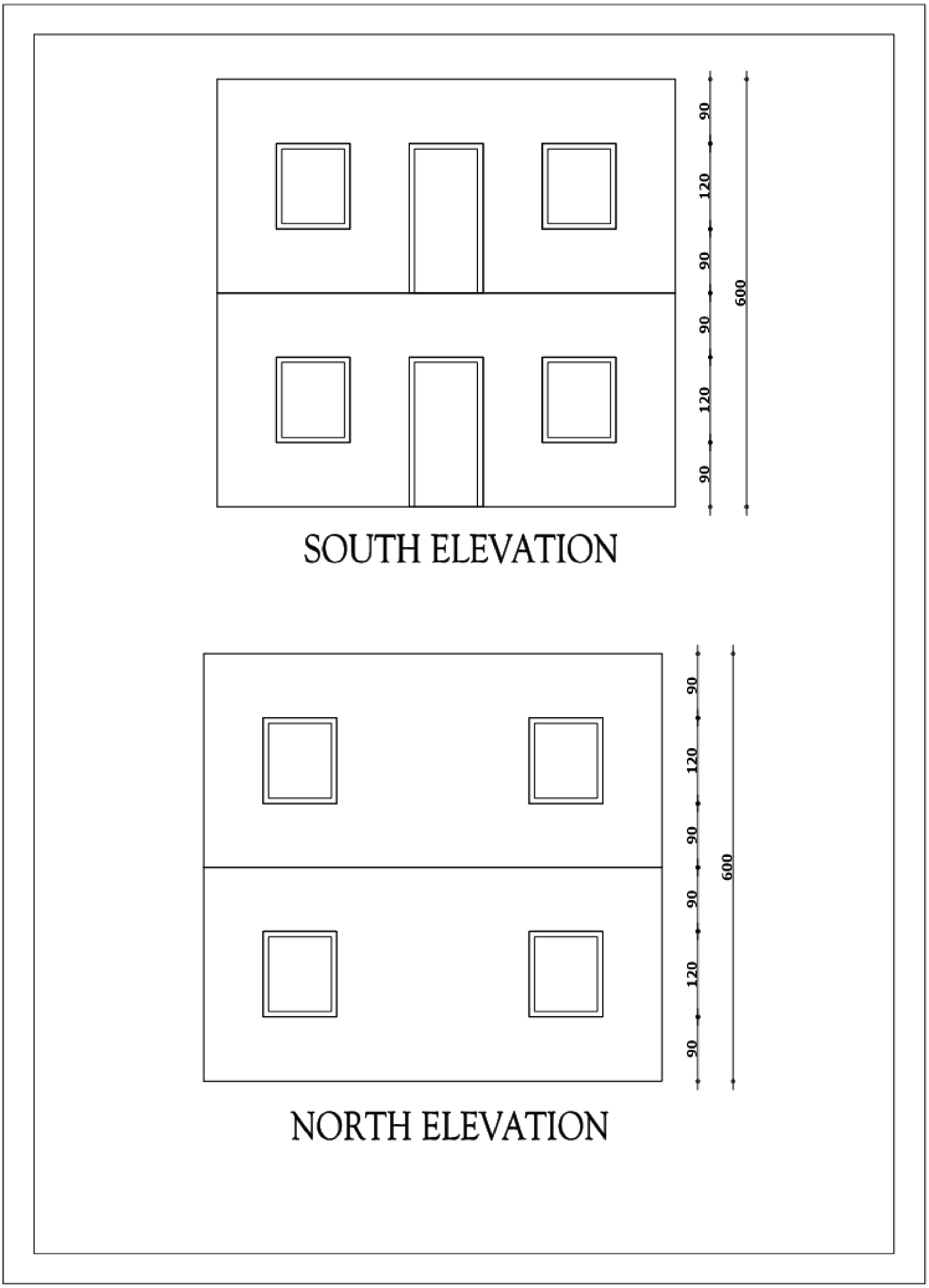


Figure 8 Building Elevation

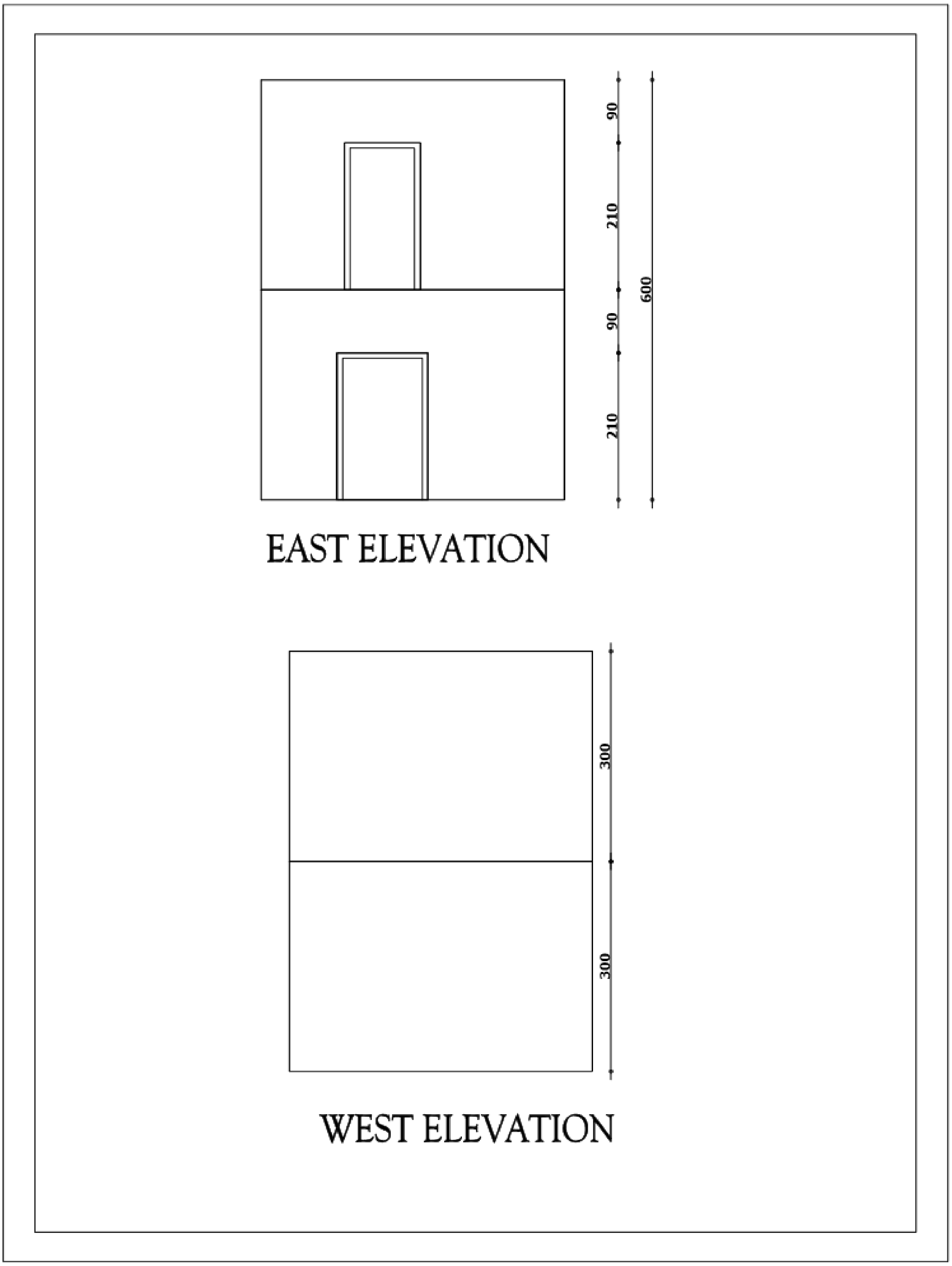
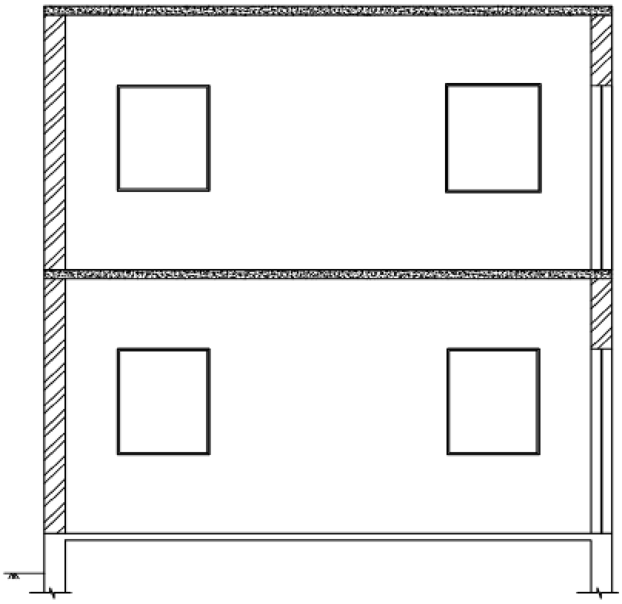


Figure 9 Building Elevation



SECTION X - X

OPENING SCHEDULE					
S.N.	DESCRIPTION	LENGTH	HEIGHT	NO.	REMARKS
1	DOOR - D	1000 MM	2100 MM	3	
2	DOOR - D2	1200 MM	2100 MM	1	
3	WINDOW - W	1000 MM	1200 MM	5	

OPENING SCHEDULE

Figure 10 Building Section and Opening Schedule

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= Residencial
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
Door: 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^3

Live Load

Live Load for floor= 3 KN/m^2
Live load for roof= 0 KN/m2

Earthquake Loads

Summary of lumped load calculation

For imposed uniformly distributed floor loads up to 3KN/m2, % of imposed load =25% (IS 1893-2002)
Dead load at storey 2 = 192.3436
Live Load at storey 2 = 0
Seismic Weight, Wi=DL+25%LL (KN) = 192.3
Dead load at storey 1 = 298.5346
Live Load at storey 1 = 74.4
Seismic Weight, Wi=DL+25%LL (KN) = 317.1
Total Dead Load = (Dead load at storey 2+ Dead load at storey 1)= 490.8782
Total Seismic weight = (Seismic Weight at storey 2 + Seismic Weight at storey 1)= 509.5

Calculation Of base shear

Seismic Zone V
Seismic Zone factor Z 0.36
Structure type School Building Importance factor 1.5
Lateral load resisting system Retrofitted masonry
Response reduction factor R 2.5
Height of the building h 6 m
Dimension of the building Along X Dx 6.2 m
Dimension of the building Along Y Dy 4 m
Time period of the building along X, Tx Ty = 0.217 Sec.
Time period of the building along Y Ty Ty= 0.2700 Sec.
Soil type Medium Soil
Average Response acceleration coefficients along X 2.5
Average Response acceleration coefficients along Y Cl. 2.5
Design Horizontal Seismic Coefficient Ah Ah= 0.27
Seismic Wt of the Building W 509.5 KN
Base Shear VB VB =AhW = 137.6 KN

Distribution of Lateral Forces at different storey

Design lateral force at floor i (Qi) =VB (Wihi k/ΣWihi k) =VB
T = 0.216869219 Sec.
= 0.216869219
Hence, K = 1
Seismic wt. Wi (kN) at storey 2 = 192.3436
Seismic wt. Wi (kN) at storey 1 = 317.1346
Storey level hi(m) of storey 2 = 6m
Storey level hi(m) of storey 2 = 3m
Wihi k (KNm) Storey 2 = 1154
Wihi k (KNm) Storey 1 = 951

Total = 2105
Design Lateral Force Qi(KN) at storey 2 = 75.4
Design Lateral Force Qi(KN) at storey 1 = 62.159
Total Design Lateral Force Qi(KN) = 137.56
Storey Shear Vj(KN) at storey 2 = 75.4
Storey Shear Vj(KN) at storey 2 = 137.6

Calculation of Lateral Coefficients

Seismic wt. Wi (kN) at storey 2 = 192.3436
Seismic wt. Wi (kN) at storey 1 = 317.1346
Design Lateral Force Qi(KN) at storey 2= 75.39
Design Lateral Force Qi(KN) at storey 1 = 62.15
Lateral Coefficient at storey 2 = 0.39
Lateral Coefficient at storey 2 = 0.20
Hence: Ci>Ah

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
fs of steel= 0.56 1.25 × fy = 290.5 N/mm2
Ast= M/ (fs×z) = 9E-05 m2
= 93.41

Dia of rods used= 8

No. of rods reqd. = 1.8575877
= 2
A_{st} provided = 100.5
% 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 = = 0.286356016 N/mm2
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, Ast= 100.57 mm^2
Interface bonding force=T=C=Ast×0.56×fy= 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/mm2
Assume minimum bond stress between concrete band and brick masonry= 0.1 N/mm2
So, the induced shear stress is less than the minimum bond stress
However, bond failure is a brittle kind of failure which is not desirable in earthquake resistant construction.
So, Provided dia of anchor= 4.75 mm from band to wall
Shearing area of the anchor=Area of provided bar = A = 17.73 mm2
Allowable shearing stress, fs=0.4fy= 166 N/mm2
Shear resistance per anchor, F=A×fs= 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)= 1.713 KN/m
Height of wall below lintel band = 1.2 m
M= $\frac{Wl2}{12}$ = 0.20556785 KN/m
Z=$\frac{bd2}{6}$ = 8816666.667
Bending stress, fb=$\frac{m}{z}$ = 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)= 8.40 m^2
Triangular load on short wall=(0.5*B^2/4)= = 4.00 m^2
Vertical load, P = Wt. of wall+Slab+Finishing = 75.291 KN
Vertical Stress, fa= P/A = 0.05279 N/mm^2

Check of combined stress

Combined Stress= fa+fb= 0.076 N/mm^2
fa-fb= -0.029 N/mm2
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

$$Ast= \frac{W}{(No\ of\ stiches*0.56*fy)} = 5.528\ mm2$$

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 P1= 1.5 m
Width of P2= 1.5 m
Width of P3= 1.8 m
Width of P4= 1 m
Width of opening in grid 2, between P3 and P4 = 1.2 m
Width of opening in grid 1, between P1 and P2 = 1 m

Pier analysis

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

Lateral load carried by piers:

We have, VB= 137.55 KN (From calculation of base shear)
Lateral load on each wall (grid 1 and grid 2) V i=$\frac{Vb}{No\ of\ walls}$ = 68.78 KN
Masonry, E = 2400000 KN/m^2
Height of pier P_4 = 2.1
Height of pier P_3 = 2.1
Depth of pier P_4 = 1
Depth of pier P_3 = 1.8
Width of pier P_4 = 0.23
Width of pier P_3 = 0.23
K= I- $\frac{bd3}{12}$
K for Pier 4 = 0.0197
K for Pier 3 = 0.1117
Moment in pier 4 = M =F×h/2 = 16.96
Moment in pier 3 = M =F×h/2 = 55.26
$Z = bd^2/6$
Z for pier 4 = 0.038
Z for pier 3 = 0.1242
f_b = M/Z mpa
f_b for pier 4 = 0.44

f_b for pier 3 = 0.44

Overturning stress (fo)

Lateral load Q1= 62.15930396 KN
Q2= 75.39981004 KN
Overturning moment= Mo=Q2/2*(h1+h2)+Q1/2*h1 = 319.44 KNm
Centroid of pier P3& P4= $\frac{A3X3+A4X4}{A3+A4}$ = 1.914 m
h3= 1.186 m
h4= -0.914
where h3 is the distance between the centroid of P3 and centroid of the grid 2.
M.O.I about centroid, IC= $I3+A3h3^{2}+I4+A4h4^{2}$= 9. 05258E+11 mm^4

Overturning stress at different piers

Point A distance from centroid = 1914.29
Point B distance from centroid = 914.28
Point C distance from centroid = 285.71
Point D distance from centroid = 2085.71
f0=Moy/Ic(MPa)
f_0 For point A= 0.68
f_0 For point B= 0.32
f_0 For point C= 0.10
f_0 For point D= 0.74

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 m^2
Vertical Stress, fa = $\frac{vertical\ load}{area}$ = 215.633 KN/m^2 = 0.21563 N/mm^2
Bending stress f_b at point A= 0.44
Bending stress f_b at point B= -0.44
Bending stress f_b at point C= 0.44
Bending stress f_b at point D= -0.44
Overturning Stress f_0 at point A = 0.68
Overturning Stress f_0 at point B = 0.32
Overturning Stress f_0 at point C = -0.10
Overturning Stress f_0 at point D = -0.74
Vertical Stress f_a at point A = -0.216
Vertical Stress f_a at point B = -0.216
Vertical Stress f_a at point C = -0.216
Vertical Stress f_a at point D = -0.216
Net Stress f_n at point A = 0.90
Net Stress f_n at point B = -0.34
Net Stress f_n at point C = 0.13
Net Stress f_n at point D = -1.40

Design of pier P4

Distance of NA from point A, x = 0.729024286 m
= 729.0242864 mm
Total tensile force, T=fnA = 75637.9962 N
Astreqd= $\frac{T}{0.56*Fy}$ = 325.4646997 mm2
Dia. of rod provided = 10 mm
NO. of rods required = 4.142277996 ≈ 5

Check for Shear

Shear force, V=F4 = 16.15 KN
Shear stress=V/A = 0.0702 N/mm2
Where, fd=compressive shear stress=fa= 0.215632919
Permissible shear stress=0.1+ f_d/6 =0.14 N/mm2
Remarks: Safe

Conclusions

Final Conclusions

In this study, first the seismic resistance of residential building was evaluated. The analysis was carried with "Strength Based Analysis Method". The analysis carried out in order to obtain the various loads of the building those were Dead Load, Live Load, and Earthquake Loads and the base shear. These forces were used to find the design lateral force and lateral coefficient also base shear. Out of plane analysis is was done and lintel band was designed which was found to be 8 mm of bars of 2 nos. these bars are safe in shear with the anchorage of 4.75mm bars at 850 mm spacing C/C. The vertical bars below the lintel bars were not required as tension reinforcement. The in plane analysis and design for vertical band were carried out and found to use of 5 nos of 10 mm of bars.

The objective of this study was to analyze seismic vulnerability of existing buildings and design methodology for use in the seismic evaluation and retrofit of the existing buildings in Nepal. Also this result helps reduce vulnerability of buildings thereby decreasing likelihood of loss of life and injury to the habitants of the buildings. From this study it was found that the buildings can be retrofitted by using various techniques to sustain the future probabilistic earthquakes which could save the loss of life and loss of properties.

Recommendations

For this study, analysis of building was carried out with strength based method. Although, SAP 2000 is largely accepted in academia and engineering offices for structural analysis, it is advisable to use different applications to compare the results rather than to rely on information from one source. In further research, it is highly recommended to use different structural analysis software to validate the results.