SEISMIC VULNERABILITY EVALUATION GUIDELINE FOR PRIVATE AND PUBLIC BUILDINGS

Part I: Pre Disaster Vulnerability Assessment

Nepal

NSET

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1. Introduction

1.1 General

This guideline is for assisting professionals and the authorities in Nepal to implement qualitative and quantitative assessment of structural earthquake vulnerability of public and private buildings in Nepal.

The book is based on the experiences gained by the National Society for Earthquake Technology Nepal in conducting visual qualitative as well as quantitative assessment of structural vulnerability of about a thousand buildings including about 20 major hospitals and about 600 schools.

This guideline is not based so much on fundamental research but on adaptation of the different available methodologies to the local conditions of Nepal and it has been tried that the procedure described here in this guideline is simple, it provides step by step suggestions of how to carryout assessment.

1.2 Basis and Scope

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

There are two phases of seismic vulnerability assessment. The qualitative assessment is for planning purpose and for identifying the priorities of intervention in the single building or the buildings complex. The quantitative assessment is for identification of retrofitting option and to examine the extent of intervention that would require in the building with consideration of technical, economic and practical feasibility. This guideline includes only some methods of detail assessment and doesn't mean to replace detail structural vulnerability assessment done by other detail methods.

2. Approaches for Data Collection for Vulnerability Assessment

2.1 Physical Surveys

Acquisition of building data pertaining to the building is the first step in evaluation. The data shall be obtained preferably prior to the initial site visit and shall be confirmed during the visit. Construction documents like as-built drawing and Structural drawing shall be required for preliminary evaluation. Site condition and soil data shall be collected if possible. However, the structural and construction drawings may not be available prior to visit to the buildings. The drawings may not be available even with the owner. When drawings are unavailable or incomplete, all necessary information shall be collected from site visit. The general information required from drawings and/or visit is about building dimension, construction age, structural system description (framing, lateral load resisting system, diaphragm system, basement and foundation system).

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

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

2.2 Interaction with Public Building Staff and Building Owners

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

- The assessment shall not solely rely on secondary information and shall involve primary data collection and confirmation of available information with the active participation of the authority and owners. The authority shall also be involved in the process of identification of mitigation options.

- The assessment work shall be taken as an awareness and education tool to promote overall earthquake safety of buildings as well as collective safety of personnel.

3. Qualitative Structural Assessment

3.1 Introduction

This chapter describes the preliminary evaluation process in general terms. Seismic Evaluation of an existing building shall be conducted in accordance with the process outlined in Sections 3.2 and 3.3. This evaluation process is performed to determine whether the building, in its existing condition, has the desired seismic performance capability. A method basically involves review of available drawings, visual evaluation of the building from the viewpoint of damage that it could suffer in the event of an earthquake. It checks the code compliance for seismic design and detailing. The process is basically a qualitative measure to identify the areas of seismic deficiencies in a building before a detailed evaluation. This will help in deciding the retrofitting requirements for the building.

3.2 Assessment of the Building

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

3.2.1 Identification of Seismicity of the Region

The region of seismicity of the building shall be identified. This is done locating the building in seismic hazard map of the region in which the building stands. The zone map of Nepal is provided in Nepal National Building Code NBC 105: 1994.

3.2.2 Establish Seismic Target Performance Level

Performance level desired is established in level of protection prior to conducting seismic evaluation and strengthening. These are classified as:

- Operational
- Immediate occupancy
- Life safety

- Collapse Prevention

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

3.2.3 Obtain As-Built Information

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

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

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

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

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

3.2.4 Building Typology Identification

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

Table 1: Common Building Types in Nepal

No.	Building Types in Kathmandu Valley	Description
1	Adobe, stone in mud, brick-in-mud (Low Strength Masonry).	Adobe Buildings: These are buildings constructed in sun-dried bricks (earthen) with mud mortar for the construction of structural walls. The walls are usually more than 350 mm. Stone in Mud: These are stone-masonry buildings constructed using dressed or undressed stones with mud mortar. These types of buildings have generally flexible floors and roof. Brick in Mud: These are the brick masonry buildings with fired bricks in mud mortar
2	Brick in Cement, Stone in Cement	These are the brick masonry buildings with fired bricks in cement or lime mortar and stone-masonry buildings using dressed or undressed stones with cement mortar.
3	Non-engineered Reinforced Concrete Moment-Resisting-Frame Buildings	These are the buildings with reinforced concrete frames and unreinforced brick masonry infill in cement mortar. The thickness of infill walls is 230mm (9”) or even 115mm (41/2”) and column size is predominantly 9”x 9”. The prevalent practice of most urban areas of Nepal for the construction of residential and commercial complexes is generally of this type. These Buildings are not structurally designed and supervised by engineers during construction. This category also includes the

		buildings that have architectural drawings prepared by engineers.
4	Engineered Reinforced Concrete Moment-Resisting-Frame Buildings	These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs. Lateral forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. These are engineered buildings with structural design and construction supervision by engineers. Some of the newly constructed reinforced concrete buildings are of this type.
5	Others	Wooden buildings, Mixed buildings like Stone and Adobe, Stone and Brick in Mud, Brick in Mud and Brick in cement etc. are other building type in Kathmandu valley and other part of the country.

Detail description of building type is given in **Annex I**

3.2.5 Determining Fragility of the Identified Building Typology

The probable damage to the building structures, that are available in Nepal and the region, at different intensities are derived based on *"The Development of Alternative Building Materials and Technologies for Nepal, Appendix-C: Vulnerability Assessment, UNDP/UNCHS 1994"* and *"European Macro-seismic Scale (EMS 98)"* http://www.gfz-potsdam.de/pb5/pb53/projekt/ems/ core/emsa_cor.htm is given in Table 2. Detail description of damage grade is shown in **Annex IV**.

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

Shaking Intensity (MMI)	**VI**	**VII**	**VIII**	**IX**
PGA (%g)	**5-10**	**10-20**	**20-35**	**>35**

Damage Grade for different classes of buildings	Weak	DG4	DG5	DG5	DG5
	Average	DG3	DG4	DG5	DG5
	Good	DG2	DG3	DG4	DG4

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

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

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

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

Table 2 (d) Non-Engineered Reinforced Concrete Frame Buildings (≥ 4 story)

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

Table 2 (e) Non-Engineered Reinforced Concrete Frame Buildings (≤ 3 story) + Engineered Reinforced Concrete Buildings +Reinforced Masonry Buildings

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

3.2.6 Identification of Vulnerability Factors

Different Vulnerability factors associated with the particular type of building are checked with a set of appropriate checklists from FEMA 310, "Handbook for the Seismic Evaluation of Buildings" and “IS Guidelines for Seismic Evaluation and Strengthening of Existing Buildings”. Separate checklist is used for each of the common building types. The design professional shall select and complete the appropriate checklist in accordance with **Annex III**. The general purpose of the checklist is to identify potential links in structures that have been observed in past significant earthquakes.

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

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

Analysis performed as part of this evaluation is limited to quick checks. The evaluation involves a set of initial calculations and identifies areas of potential weaknesses in the building. The checks to be investigated are classified into two groups: configuration related and strength related. The preliminary evaluation also checks the compliance with the provisions of the seismic design and detailing codes. Quick checks shall be performed in accordance with evaluation statement to verify compliance or non-compliance situation of the statement. Seismic shear force for use in the quick checks shall be computed as per National building seismic code of the region.

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

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

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

3.2.7 Reinterpretation of the Building Fragility Based on Observed Vulnerability Factors

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

The states of damage of Reinforced Concrete and Masonry buildings are classified into five grades as given in **Annex IV**

3.3 Conclusions and Recommendation

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

The evaluation helps in deciding whether the safety provided by the building is adequate or not. And decision is taken whether the building needs to be repaired, retrofitted or demolished based on the importance, target life, extend of deficiency of the building, the economic viability, the availability of the materials and technical resources, the expected life after retrofit. The stakeholders such as house owners, design engineers, occupants, municipality etc. are responsible in making the decision. The action can be either of the following.

a) The safety of the building is adequate. The building needs some repair and regular maintenance, ensuring adequate performance during a future earthquake.

b) The safety of the building is inadequate and hence, retrofit is necessary. The proposed retrofit scheme should be technically feasible and economically viable (Usually retrofitting is considered suitable if the cost of retrofitting is within 30% of the cost of new construction). The required retrofit is to be implemented.

c) The safety of the building is inadequate and the building is in imminent danger of collapse in the event of an earthquake. The retrofit scheme is not economically viable or feasible. It is recommended demolition and reconstruction than retrofitting for better seismic performance unless the building has historical importance and is of traditional nature.

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

4. Quantitative Assessment

4.1 Introduction

This Chapter describes the second phase study of seismic vulnerability assessment which is a quantitative approach and follows qualitative analysis. Before embarking on seismic retrofitting, seismic deficiencies shall have to be identified through a seismic evaluation process using a methodology described in Chapter 3. The first phase assessment is general seismic vulnerability assessment method based on qualitative approach to identify the seismic deficiencies in the building. If the first phase study finds seismic deficiencies in the building and possible seismic performance is not up to the acceptable level/criteria, it recommends either second phase assessment or conclude the evaluation and state that potential deficiencies are identified. The second phase assessment involves a more detail seismic evaluation with complete analysis of the building for seismic strengthening measures as modifications to correct/ reduce seismic deficiencies identified during the evaluation procedure in first phase. Detail information about the building is required for this step of evaluation. Seismic retrofit becomes necessary if the building does not meet minimum requirements of the current Building Code, and may suffer severe damage or even collapse during a seismic event.

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

Quantitative assessment of an existing building shall be conducted in accordance with the process outlined in sections 4.1 through 4.10.

4.2 Review Initial Considerations

The design professional shall review initial considerations which include structural characteristic of the building, seismic hazard including geological site hazards, results of prior seismic evaluations, areas of structural deficiencies, building use and occupancy requirements, historical status, economic considerations, societal issues, and local jurisdictional requirements. This step of evaluation should focus on the potential deficiencies identified in Section 3.

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

4.3 Decide Performance Objective

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

4.4 Design Basis Earthquake

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

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

4.5 Detail Investigation

This includes the following:

a) Obtaining the properties of the structural materials used in the building.

b) Determining the type and disposition of reinforcement in structural members.

c) Locating deteriorated material and other defects, and identifying their causes.

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

4.5.1 Non-Destructive Tests

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

4.5.1.1 Sounding Test

<u>Description</u>

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

<u>Equipment</u>

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

<u>Conducting Test</u>

In areas where the visual observations indicate that the wall may have delaminations, the wall can be sounded by tapping with a hammer. Delaminations and spalls will generally produce a hollow sound when compared with solid material. The wall should be tapped several times in the suspect area and away from the suspect area, and the sounds compared. It is important to test an area that is undamaged, and of the same material and thickness to use as a baseline comparison. For a valid comparison, the force exerted by the tapping should be similar for both the suspect and baseline areas. In reinforced masonry construction, sounding can be used to assess whether the cells in the wall have been grouted. Near the ends of a block, the unit is solid for the full thickness of the wall. For most of the length of the block, it is relatively thin at the faces. If the sound near the end of the block is substantially different than at the middle of the cell, the cell is probably not grouted.

<u>Personal Qualification</u>

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

<u>Reporting Requirements</u>

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

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

Limitations

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

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

4.5.1.2 Rebound Hammer Test

Description

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.

Equipment

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

Execution

The person operating the equipment places the impact plunger of the hammer against the concrete and then presses the hammer until the hammer releases. The operator then records the value on the scale of the hammer. Typically three or more tests are conducted at a location. If the values from the tests are consistent, record the average value. If the values vary significantly, additional readings should be taken until a consistent pattern of results is obtained.

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

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 1. (a) Use of Rebound Hammer	**Fig 1. (b) Rebound Hammer**

Personal Qualification

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

Reporting Requirements

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

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

Limitations

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

The results of the rebound hammer tests are sensitive to the quality of the concrete on the outer several inches of the wall. More reproducible results can be obtained from formed surfaces rather than from finished surfaces. Surface moisture and roughness can also affect the readings. The impact from the rebound hammer can produce a slight dimple in the surface of the wall. Do not take more than one reading at the same spot, since the first impact can affect the surface, and thus affect the results of a subsequent test.

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

4.5.1.3 Rebar Detection Test

Description

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.

Equipment

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

Conducting Test

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

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

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

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

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 2. (a) Use of Rebar Detector for Verification of Reinforcement Details	**Fig 2. (b) Ferroscan Detector**

<u>Personnel Qualifications</u>

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

<u>Reporting Requirements</u>

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

- Mark the locations of the test on either a floor plan or wall elevation.
- Report the results of the test, including bar size and spacing and whether the size was verified.

- List the type of rebar detector used.
- Report the date of the test.
- List the responsible engineer overseeing the test and the name of the company conducting the test.

Limitations

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

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

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

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

4.5.1.4 In-Situ Testing In-Place Shear

Description

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

Equipment

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

Execution

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

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

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

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

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 3. Test Set up for In-Situ Shear Test	

Personnel Qualifications

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

Reporting Results

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

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

Limitations

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

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

The test results can only be qualitatively adjusted to account for the presence of mortar in the collar joints. If mortar is present in the collar joint, the engineer or technician conducting the test is not able to discern how much of that mortar actually resisted the force from the ram.

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

4.6 Seismic Analysis and Design

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

4.7 Intervention Options for Better Seismic Performance

4.7.1 General

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

(i). Requirements to complies to the Building Code for design, materials and construction

(ii). Compatibility of the solution with the functional requirements of the structure

(iii). Possible cost implication

(iv). Indirect cost of retrofitting such as relocation cost

(v). Availability of construction technique (materials, equipments and workmanship) in construction industry

(vi). Enhancement of the safety of the building after intervention of the selected option

(vii). Aesthetic view of the building

Once these considerations are made, different options of modifying the building to reduce the risk of damage should be studied. The corrective measures include stiffening or strengthening the structure, adding local elements to eliminate irregularities or tie the structure together, reducing the demand on the structure through the use of seismic isolation or energy dissipation devices, and reducing the height or mass of the structure.

4.7.2 Retrofitting Methods

4.7.2.1 General Improvement

Plan Shape

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

Elevation Improvement

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

Load Path

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

Inserting New Walls

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

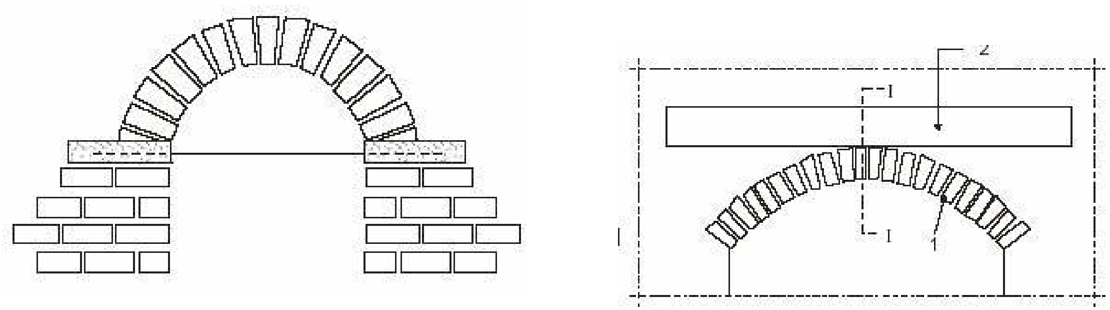
Modification of Roofs or Floors

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

Strengthening the Arches

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

To prevent spreading of arches, it is proposed to install tie rods across them at spring levels or slightly above it by drilling holes on both sides and grouting steel rods in them (Figure 4.a below). However, where it is not possible a lintel consisting of steel channels or I-section, could be inserted just above the arch to take the load and relieve the arch as shown in Figure 4.b.



(b)

Fig 4. Strengthening of Arches (IAEE, 1984)

Reduction in Building Mass

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

4.7.3 Seismic Retrofitting Strategies of Masonry Buildings

4.7.3.1 Major Weaknesses Revealed During Earthquakes in Similar Building Typology

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

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

4.7.3.2 Common Retrofitting Methods for the Masonry Buildings

The concept of retrofitting masonry buildings start from enhancing integrity to the structure by providing proper connections between its resisting elements in such a way that inertia forces generated by the vibration of the building can be transmitted to the members that have ability to resist. Typical important aspects are the connection a) between components of floors and roof; b) between roof or floors and walls; c) between intersecting walls; and d) walls and foundation.

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

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

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

Jacketing

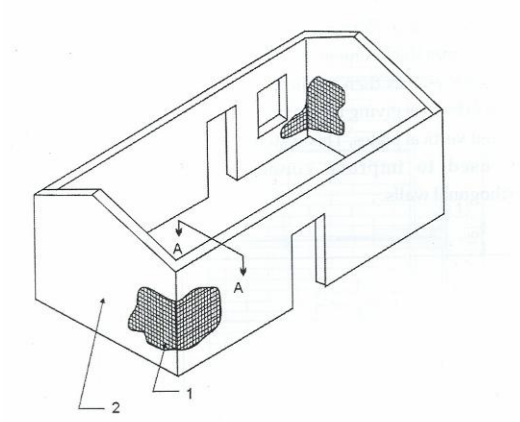
This method is adopted on buildings constructed with a material that is of heavy in weight, weak in strength, and brittle. It helps to basket the wall, hence improve its shear strength and ductility. This method also improves integrity and deformability. Main improvements in different structural elements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

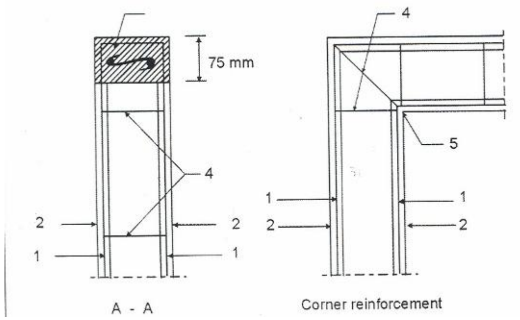
Walls: To improve strength, deformability and to reduce risk of disintegration, delamination of walls resulting in total collapse of the building, thin reinforcement concrete jacketing of all the walls is done. In this alternative two steel meshes should be placed on either two sides or one side of the wall and both the meshes should be connected by some steel bars connectors passing through the wall. The thickness of the added concrete should be about 40 to 50 mm thick. The concrete used ought to be a micro-concrete i.e. concrete with small aggregates. Selection of one side jacketing or two side jacketing depends on the analysis result.

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

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

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





1 - Welded wire mesh ~ 50 mm × 50 mm
2 - Mortar or micro-concrete rendering
3 - Concrete roof band
4 - Cross ties at 500 to 750 mm apart
5 - Corner bar diameter 8 mm

Fig 5.(a) General Scheme of Jacketing

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 5. (b) Erection of Reinforcement for Wall Jacketing	**Fig 5. (c) Wall Jacketing in Process**
Process of Wall Jacketing	

Splint and Bandage

The Splint and Bandage system is considered as an economic version of jacketing where reinforcing bars are provided at most critical locations (Figure 6), wherever stress concentrations can develop. Splints are vertical elements provided at corners, wall junctions and jambs of openings in the external faces of the building. The objective is to provide integrity in vertical direction.

The bandages are horizontal elements running around all the walls and building to integrate various walls together thereby preventing potential out of plane collapse of walls. In addition, openings are also surrounded by splints and bandages to prevent initiation and widening of cracks from their corners. Splints are provided in the external face only. The bandages could be provided on both the faces of the walls just at the lintel, eaves and sill level. This method is inferior to jacketing but better than bolting as discussed below in terms of safety enhancement. In splint and bandage system, the strengthening and stiffening of the floor and roof is made in the same way as discussed above under Section 4.7.3.2 Jacketing.

	Before
During	

	After
Fig 6. Process of Retrofitting by Splint and Bandage Method (Photo Courtesy: NSET)	

Bolting/ Pre-stressing

A horizontal compression state induced by horizontal tendons is used to improve the shear strength of in-plane walls. This also considerably improves the connections between orthogonal walls. The easiest way of affecting the pre-stressing is to place two steel rods on the two sides of the wall and strengthening them by turnbuckles (Figure 7). These are done at two levels each storey viz. a) lintel level and b) just below the floor and roof structure. This method improves the earthquake resistance of the building and will delay the collapse, but it is still much inferior to the jacketing or split and bandage in terms of increasing safety. This method is cheaper and will be effective for small and simple buildings.

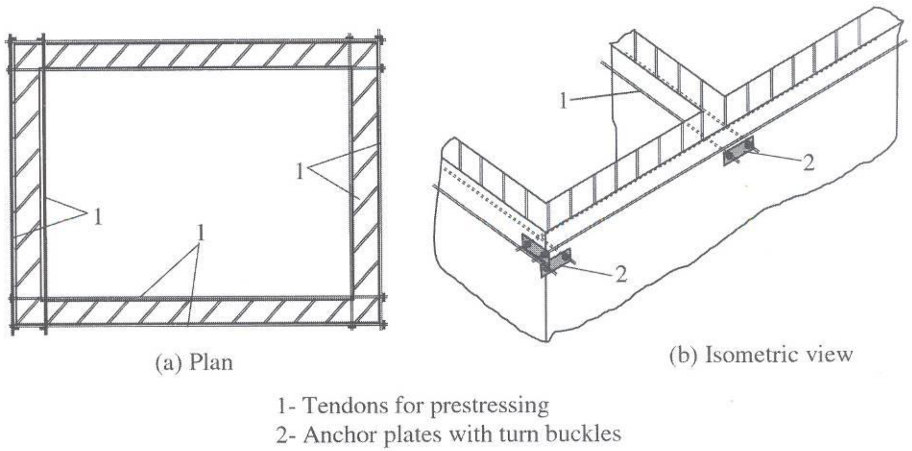


Figure 7: Retrofitting of Masonry Building by Pre-stressing

Confinement with Reinforced Concrete Elements

Confinement with reinforced concrete elements (beam and columns) make the existing masonry act as "confined masonry" in the sense that reinforced concrete elements are inserted surrounding the wall panel or middle of the long wall, allowing the entire wall, or its portion, to act as a truss element, where the struts are inclined strip of unreinforced masonry. In this way, brittle and non-ductile wall becomes more ductile and its load carrying capacity increased several times with added confinement of the reinforced concrete elements. It is more suitable for buildings of one to three storey heights with monolithic reinforced concrete slab and horizontal bands over the load bearing walls at the lintel level. However, implementation of this method of retrofitting is more complex and needs special improvements for foundation also.

Base Isolation

What effectively is done in this scheme is that the superstructure is strengthened nominally and is isolated from ground motion by introducing a flexible layer between the structure and the ground. The various types of base isolation devices are i) Laminated rubber bearing ii) Laminated rubber bearing with lead core

iii) Sliding bearing and iv) Friction pendulum devices. Base isolation modifies the response characteristics so that the maximum earthquake forces on the building are much lower. The seismic isolation eliminates or significantly reduces not only the structural damage but also non-structural damage and enhances the safety of the building content and architectural components (Figure 8 below). This technique is usually employed for buildings with historic importance and critical facilities and is quite expensive as compared to other methods.

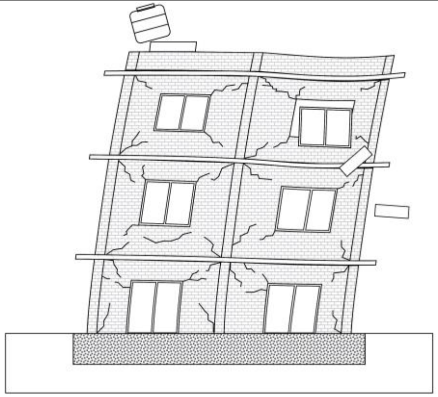
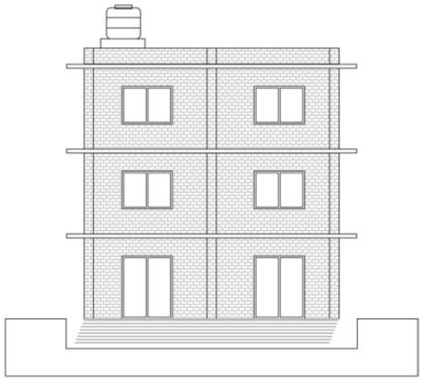
	

Fig 8. (a) Without Base Isolation (Masonry Building)	**Fig 8. (b)With Base Isolation (Masonry Building)**

Use of FRP (Fiber Reinforced Polymer)

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

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

- Increases out-of-plane flexural strength

- Increases in-plane shear strength
- Increases stiffness at service loads
- Results in monolithic action of all units
- Strengthening of entire wall can be accomplished by treating only a fraction of wall surface area
- Adds very little weight to the wall
- Minimum changes in the member size after repair
- Limited access requirements
- Lower installation cost
- Improved corrosion resistant
- On-site flexibility of use

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

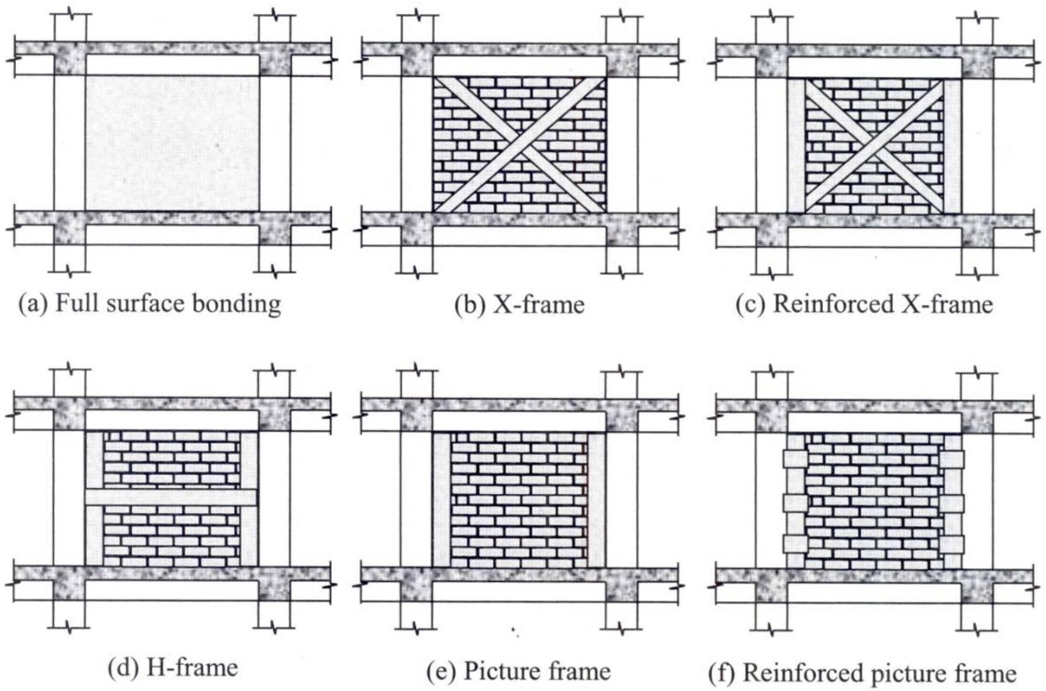


Fig 9. Configurations of FRPLaminates of Masonry Walls

4.7.3.3 Comparison of Common Methods of Retrofitting for Masonry Building

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

Table 3: Comparison of Different Retrofitting Options

	Retrofitting Options					
	Jacketing	**Splint and Bandage**	**Bolting/ Prestressing**	**Confinement with reinforced concrete elements**	**Base Isolation**	**Strengthening with FRPs**
Maximum Nos. of	Suitable upto four	Suitable upto three	Suitable upto two	Suitable upto three storey	Suitable for low to	Suitable for low rise

Storey	storey	storey, preferable for two storey	storey		medium rise buildings with time period upto 0.5sec	buildings upto 2 Stories
Architectural Changes	Extensive	Moderate	Less	Significant	Insignificant	Less
Intervention time	Long	Moderate	Short	Long	Long	Less
Cost	High	Moderate	Low	High	Extensive	High
Safety achieved upto MMI IX	Life safety - Immediate Occupancy	Life safety	Brittle collapse prevention	Life safety	Immediate Occupancy	Life safety

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

4.7.4 Seismic Retrofitting Strategies of Reinforced Concrete Buildings

4.7.4.1 Major Weaknesses Revealed During Earthquakes in Similar Building Typology

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

- absence of ties in beam column joints
- inadequate confinement near beam column joint
- inadequate lap length and anchorage and splice at inappropriate position
- low concrete strength
- improperly anchored ties (90^o hooks)
- inadequate lateral stiffness
- inadequate lateral strength

- irregularities in plan and elevation
- irregular distribution of loads and structural elements
- other most common structural deficiencies such as soft storey effect, short column effect, strong beam-weak column connections etc.

4.7.4.2 Common Retrofitting Methods for the Reinforced Concrete Buildings

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

a) Increasing seismic capacity of the building

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

1) Strengthening of original structural members

These include strengthening of

- Columns (reinforced concrete jacketing, steel profile jacketing, steel encasement, fiber wrap overlays)
- Beams (reinforced concrete jacketing, steel plate reinforcement, fiberwrap overlays)Beam Column joint (reinforced concrete jacketing, steel plate reinforcement, fiber wrap overlays)
- shear wall (increase of wall thickness)
-
- Slab (increase of slab thickness, improving slab to wall connection)
- Infilled partition wall (reinforce infilled walls and anchor them into the surrounding concrete frame members).

2) Introduction of New structural elements

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

- shear walls in a frame or skeleton structure
- Infilled walls (reinforced concrete or masonry located in the plane of existing columns and beams)
- wing walls (adding wall segments or wings on each side of an existing column)

- additional frames in a frame or skeleton structure
- trusses and diagonal bracing (steel or reinforced concrete) in a frame or skeleton structure

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

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

b) Reducing seismic response of the building

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

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

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

Reinforced Concrete Jacketing

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

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

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

Addition of Reinforced Concrete Shear Walls

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

Photo Courtesy: MRB & Associates

Photo Courtesy: MRB & Associates

Fig 10. (a) Jacketing of RC Column	Fig 10. (b) Addition of Shear Wall and Column Jacketing

Steel Bracing

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

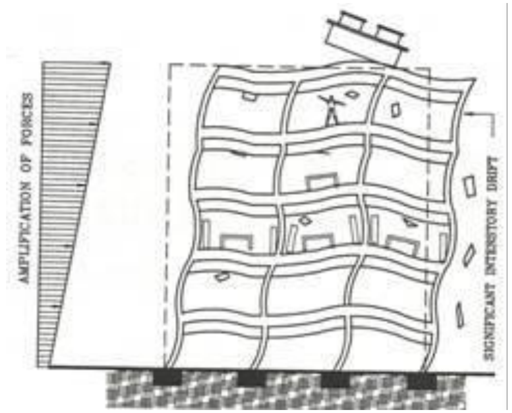
Photo Courtesy: MRB & Associates

Photo Courtesy: MRB & Associates

Fig. 11. Retrofitting by Diagonal Steel Bracing

Base Isolation

In this method superstructure is isolated from ground motion during earthquake shaking by using flexible layer between the structure and the ground as discussed in Section 4.7.3.2 Base Isolation. The only difference is that these isolators are introduced individually beneath column support (Fig 12), while as in masonry building a flexible layer is introduced throughout the wall stretch at base (Fig 8).



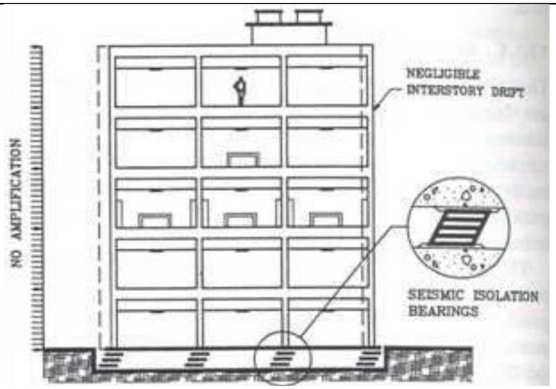


Fig 12. (a) Without Base Isolation (RC Frame Building)	**Fig 12. (b) With Base Isolation (RC Frame Building)**

Use of FRP (Fiber Reinforced Polymer)

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

4.7.4.3 Comparison of Common Methods of Retrofitting for Reinforced Concrete Building

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

Table 4: Comparative Chart of Different Retrofitting Options for RC Frame Buildings

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

The study should consider the structural system of the building, its major structural problems and different available options of retrofitting.

4.7.5 Foundation Intervention

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

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

-Foundation failures may result in severe economic loss resulting in damage to structural and non-structural elements. But, failure of foundation may have smaller effect on the Life-safety and collapse prevention limit as large foundation movements are needed to cause structural collapse.

-Seismic strengthening or upgrade of the foundation may result in transmission of larger seismic forces into the structure. Hence, foundation strengthening may increase the cost of structural upgrade since more structural work is required in response to foundation work. In some cases, foundation upgrade may adversely affect the life safety and collapse prevention limit states. The engineer must balance a range of economic, social and technical concerns, when evaluating these issues.

-However, in general the foundation work will reduce the probability of serious economic damage during an earthquake.

4.8 Cost Estimate

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

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

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

4.9 Comparison of Possible Performance of the Building after Retrofitting

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

4.10 Conclusions and Recommendations

4.10.1 Conclusions

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

- Various retrofit options are compared and studied. Out of which, the most suitable retrofit technique is proposed for the particular building type keeping all factors as mentioned in previous chapters in consideration. The retrofit option should improve the building response with Life Safety as minimum requirement.

- However, the cost of retrofitting may differ to some extent if the actual structural strength and details are found other than that is considered during retrofit design once walls and roof are opened during field implementation.

4.10.2 Recommendations

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

- A time-bound program should be implemented to retrofit the building with incorporation of seismic resistant measures as selected.

- Retrofitting is an advanced process and requires a higher level of expertise than that required for design and construction of new buildings. The process requires lots of destructive interventions such as hammering, drilling in walls, and removal of some parts of building. Such activities may cause additional damage if proper attention is not given during implementation. Hence, use of experienced and skilled labor is strictly emphasized with proper supervision.

- Retrofit design may need revision once structural, architectural and cosmetic elements of the building are removed for implementation and details found other than is assumed for designing. Hence, it is suggested to make clear these things and seek flexibility on design details, actually to be implemented at site, from contractor's side before contract signing.

- During retrofitting process, the elements such as floor cornices, chajjas, cladding, false ceiling that add beauty to the building, have to be removed. These things are to be considered prior to implementation of retrofitting and suggested for designer's advice to retain good aesthetic view of the building after retrofitting.

- Supervision during the retrofitting works is very essential as it is a delicate work. Hence, it is extremely important to have proper supervision at the site during construction work.

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

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18. Reinforced Concrete Structures: R. Park and T Paulay

Annex I: Private and Public Buildings Typology

Type 1 - Adobe, Stone in Mud, Brick-in-Mud (Low Strength Masonry).

These buildings are mud-based constructed buildings and mostly found in rural areas. The vulnerability of these types of buildings mainly depends on the inherent structural strength of the wall material together with the technology of construction. Vertical wooden posts and horizontal wooden elements embedded in walls are the expected key earthquake resistant elements in these buildings. The type of floor and roof used such as flat or sloping, heavy or light, properly fixed with walls or simply rested, braced or un-braced etc. highly influence the vulnerability of such buildings.

Adobe Buildings: These are buildings constructed using sun-dried bricks (earthen) with mud mortar for the construction of the structural walls. The walls are usually more than 350 mm. thick.

Stone in Mud: These are stone-masonry buildings constructed using dressed or undressed stones with mud mortar. They generally have flexible floors and roofs.

Brick in Mud: These are brick masonry buildings with fired bricks in mud mortar.

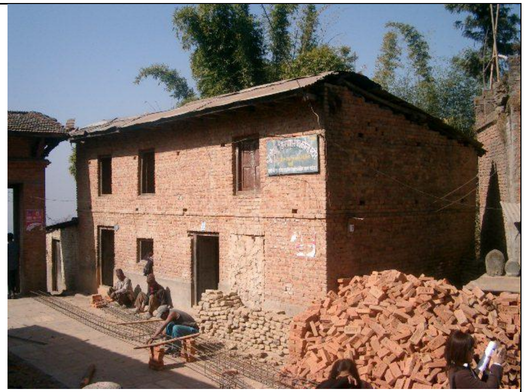
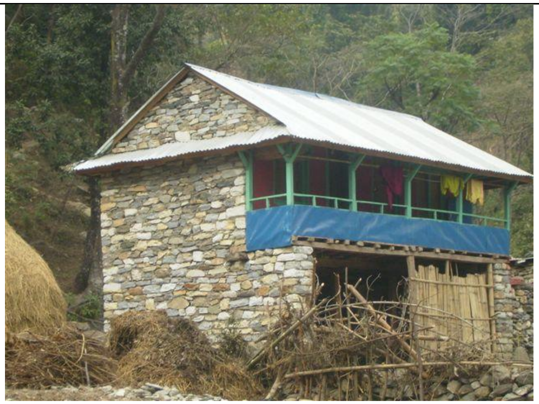
 Photo Courtesy: NSET	 Photo Courtesy: NSET

Fig 1.1 (a) Adobe Building	**Fig 1.1 (b) Brick in Mud Building**
 Photo Courtesy: NSET	**Fig 1.1 (c) Stone in Mud Building**

Type 2 - Brick in Cement, Stone in Cement

These types of buildings are the most common buildings in Nepal. Buildings that are more than 15-20 years old are mostly this type in urban areas. Buildings that are built mostly in rural and outskirts of urban areas are of this type.

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 1.2 (a) Brick in Cement Building	**Fig 1.2 (b) Stone in Cement Building**

Main features of this type of buildings are as follows:

- Foundations are usually openly-excavated strip footings built of stone in mud mortar or brickwork in cement mortar up to the ground-level. The plinth masonry above ground-level to the plinth-level is brickwork in cement mortar, the thickness of walls being about half a brick larger than the superstructure walls.
- The superstructure walls are one brick thick constructed in 1:6 cement sand mortar, in general. Bricks are of a good quality, usually with a crushing strength of more than 7.5 N/mm^2. The construction quality is good with soaking of bricks beforehand and filling of joints with mortar.
- The number of stories usually goes up to three. The floors are of either reinforced concrete or reinforced brick slabs. The roof is also of similar construction although in some cases it is made sloping using RC slabs.
- The use of lintel-level bands was not practiced. Rarely, a peripheral beam was cast with the floor slab. But, however, some newly built buildings do have earthquake resistant features such as horizontal bands at sill, lintel and floor level and vertical band at corners and junction of walls.

Type 3 – Non-Engineered Reinforced Concrete Moment-Resistant-Frames.

This type of building consists of a frame assembly of cast-in-place concrete beams and columns. The floors and roof consist of cast-in-place concrete slabs. Walls consist of infill panels constructed of solid clay bricks. The present trend of building construction in urban areas of Nepal for residential, shop-cum-residential and shop-cum-office-cum-residential buildings is to use reinforced concrete beam-column frames with randomly-placed brick walls in two directions. These buildings are usually built informally. Some of the conspicuous features of such buildings are:

- *Planning:* The column spacing in each direction of the building varies from 3 m to 4.5 m. In most cases, the storey-heights are 2.7 m but sometimes they are up to 3.0 m floor-to-floor. Internal partitions and parapet walls are usually half-a-brick thick while external walls are one-brick thick with relatively big openings for windows.
- *Foundations*: Individual column footings type foundation. The area generally varies from 1.2 m x 1.2 m to 2.0m x 2.0m. The depth varies from 0.9 to 1.2 m below ground level.
- *Columns:* A 230 x 230 mm (9" x 9") column-size is most commonly used for up to five stories and even more, both for face and internal columns. The longitudinal reinforcement commonly used is 4 bars of 16 φ and 2 bars of 12φ of high-strength steel (Fe415) and the ties are usually either 6 φ plain mild steel (Fe250) or 5 φ high-strength twisted steel (Fe500) at 200 mm centers.
- *Beams*: A usual size is 230 x 230 mm (9" x 9"), with a web projecting below a slab with which it is monolithic, with three to four 12 φ bars of high-strength bottom steel and two similar bars at the top. Out of the bottom bars, one or two bars are cranked up, making three to four bars near the supports for the hogging moment.
- *Slabs:* The slabs are usually made of reinforced concrete or reinforced brick concrete (RBC) 75 to 100 mm (3" to 4") thick, with 10 φ high-strength steel at 130 mm centers spanning the shorter dimension and the same at 250 centers in the longer span. Alternate bars are bent up near supports to carry the negative moment.

Photo Courtesy: NSET

Photo Courtesy: NSET

Fig 1.3 Non Engineered Reinforced Concrete Moment Resisting Frame Buildings

The buildings can further be divided into two sub groups, considering the number of stories, as the vulnerability of these types of buildings highly depends on the number of stories.

A: Non engineered reinforced concrete moment resisting frame building with more than three stories.

B: Non engineered reinforced concrete moment resisting frame building less than or equal to three stories.

Type 4 -Engineered Reinforced Concrete Moment-Resistant-Frames

These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs. Lateral forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. These buildings are built with little or extensive input from engineers or designers for earthquakes. Some of the newly constructed reinforced concrete buildings in urban areas of Nepal are likely to be of this type. These buildings are categorized in three groups:

Group I- Good type of engineered RC moment resisting frame building typology: These buildings are properly designed by engineers for expected earthquake. Column size is minimum of 300 mm X 300 mm or more depending on load induced. Shape of this type of building is regular and ductile detailing is fully enforced at site as per IS 13920.

Group II: Average type of engineered RC moment resisting frame building typology: These buildings are designed by engineers for earthquake force and column size is usually 230 mm X 300 mm. However, ductile detailing is partially implemented in this type of building.

Group III: Weak type of engineered RC moment resisting frame building typology: These buildings are either not designed by engineers or designed for vertical load only. Column size is usually 230 mm X 230 mm or 230 mm X 300 mm and ductile detailing is generally not implemented or partially implemented. These buildings have critical deficiencies which can be either of soft storey effect, short column effect, shape irregularity, inadequate distribution of structural elements or lack of ductile detailing.

The seismic performance of this type of construction depends on the interaction between the frame and the infill panels. The combined behavior is more like a shear wall structure than a frame structure. Solidly in-filled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, the diagonal compression struts will not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The shear strength of the concrete columns, after cracking of the infill, may limit the semi ductile behavior of the system.

Photo Courtesy: NSET

Photo Courtesy: NSET

Fig 1.4 Engineered Reinforced Concrete Moment Resisting Frame Buildings

Type 5 - Other

If the building does not fall within one of the categories mentioned above the building may have different seismic behavior depending on its inherent strengths and weaknesses. This is due to use of composite and mixed type of reinforced concrete, masonry units and mortar in the same building.

Photo Courtesy: NSET

Fig 1.5 (a) Stone in Mud in Ground Floor and Brick in Mud in First Floor

Annex II: Seismic Vulnerability Factors and their consequence

Basic Factors Influencing the Seismic Performance of Buildings

Load Path

The general load path of a building is as follows: seismic forces originating throughout the building are delivered through structural connections to horizontal diaphragms; the diaphragms distribute these forces to vertical lateral-force-resisting elements such as shear walls and frames; the vertical elements transfer the forces into the foundation; and the foundation transfers the forces into the supporting soil.

There must be a complete lateral-force-resisting system that forms a continuous load path between the foundation, all diaphragm levels, and all portions of the building for proper seismic performance. If there is a discontinuity in the load path, the building is unable to resist seismic forces regardless of the strength of the existing elements. Mitigation with elements or connections needed to complete the load path is necessary to achieve the selected performance level.

Examples would include a masonry shear wall that does not extend to the foundation, or a column in upper story that does not continue to foundation.

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 2.1 (a) Full Brick Wall on Cantilever Slab Projection	**Fig 2.1 (b) Beam Resting on Beam**

Load Path Problem

Is there any masonry wall in cantilever?

Any column has started from beam? Not continue from foundation?

Is there any masonry wall, which does not continue to foundation?

If yes, there is problem of clear load path!

Adjacent Buildings and Poundings

If buildings are built without sufficient gap and the interaction has not been considered, the buildings may impact each other, or pound, during an earthquake. Building pounding can alter the dynamic response of both buildings, and impart additional inertial loads on both structures. Buildings that are with the same height and have matching floors will exhibit similar dynamic behavior. If the buildings pound, floors will impact other floors, so damage due to pounding usually will be limited to nonstructural components. When the floors of adjacent buildings are at different elevations, floors will impact the columns of the adjacent building and can cause structural damage. Since neither building is designed for these conditions, there is a potential for extensive damage and possible collapse.

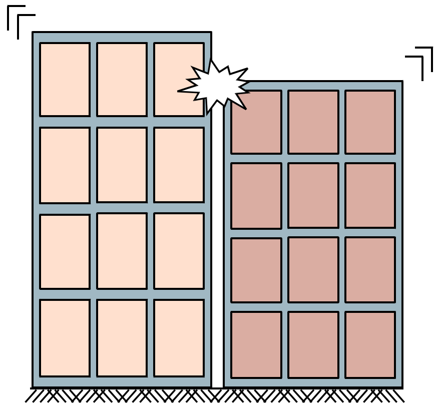


Fig 2.2 (a): Different Floor Height Buildings Suffer More in Pounding

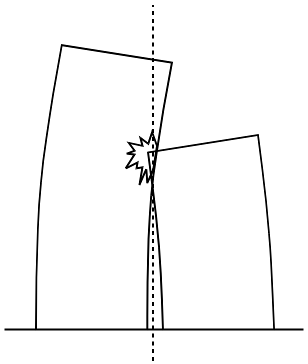


Fig 2.2 (b): Pounding due to Small Gap of Two Buildings

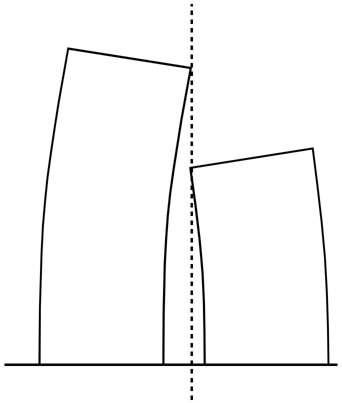


Fig 2.2 (c): Sufficient Gap Between Two Buildings Prevent from Pounding

Photo Courtesy: NSET

Fig 2.2 (d) Sufficient Gap Between Buildings to Avoid Pounding

Photo Courtesy: NSET

Fig 2.2 (e) Buildings Attached to Each Other Without Seismic Gap

(These buildings are liable to suffer in pounding)

If any of the following statements is true, then there will be problem of pounding.
The building is attached to another building and there is no gap between them.
There is a gap between them but the gap is filled with rigid material like concrete or brick.
The gap is made rigid with the use of metal or any other rigid material at the floor levels.When the floor levels of the adjacent buildings are at different levels, there will be further more effect due to pounding.

Configuration

Configuration of buildings is related to dimensions, building form, geometric proportions and the locations of structural components. The configuration of a building will influence the seismic performance of a building, particularly regarding the distribution of the seismic loads.

From past earthquake experiences, it can be stated that the buildings with simple configurations and symmetrical are more resistant to earthquake shaking. Good details and construction quality are of secondary value if a building has an odd shape that is not properly considered in the design. Although a building with an irregular configuration may be designed to meet all code requirements, irregular buildings generally do not perform as well as regular buildings in an earthquake. Typical building configuration deficiencies include an irregular geometry, a weakness in a given story, a concentration of mass, or a discontinuity in the lateral force resisting system.

Vertical irregularities are defined in terms of strength, stiffness, geometry, and mass. These quantities are evaluated separately, but are related and may occur simultaneously. Horizontal irregularities involve the horizontal distribution of lateral forces to the resisting frames or shear walls.

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 2.3 (a) U Shaped Building	**Fig 2.3 (b) L-Shaped School Building**
Irregularities in Shape	

Redundancy

Redundancy is a fundamental characteristic of lateral force resisting systems with superior seismic performance. Redundancy in the structure will ensure that if an element in the lateral force resisting system fails for any reason, there is another element present that can provide lateral force resistance. Redundancy also provides multiple locations for potential yielding, distributing inelastic activity throughout the structure and improving ductility and energy dissipation. Typical characteristics of redundancy include multiple lines of resistance to distribute the lateral forces uniformly throughout the structure, and multiple bays in each line of resistance to reduce the shear and axial demands on any one element.

A distinction should be made between redundancy and adequacy. The redundancy mentioned here is intended to mean simply "more than one." That is not to say that for large buildings two elements is adequate, or for small buildings one is not enough.

Photo Courtesy: NSET

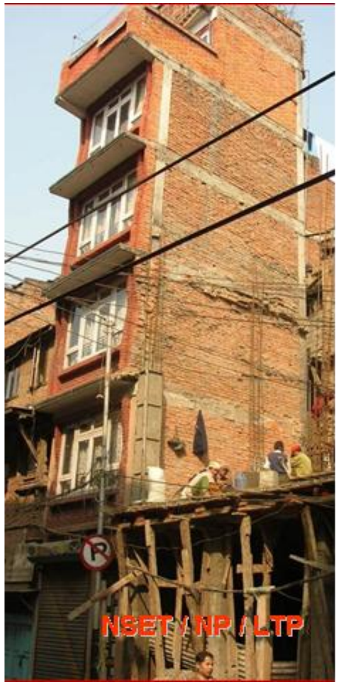


Photo Courtesy: NSET

Fig 2.4 (a) Single Bay RC Frame Building	**Fig 2.4 (b) Slender Building**
Problem due to Inadequate Redundancy	

Is the building structure single bay in one or both direction?

If yes, there is no redundancy in the building.

Weak Story

The story strength is the total strength of all the lateral force-resisting elements in a given story for the direction under consideration. It is the shear capacity of columns or shear walls. If the columns are flexural controlled, the shear strength is the shear corresponding to the flexural strength. Weak stories are usually found where vertical discontinuities exist, or where member size or reinforcement has been reduced. It is necessary to calculate the story strengths and compare them. The result of a weak story is a concentration of inelastic activity that may result in the partial or total collapse of the story.

Soft Story

This condition commonly occurs in buildings in urban areas where ground floor is usually open for parking or shops for commercial purposes. Soft stories usually are revealed by an abrupt change in inter-story drift. Although a comparison of the stiffness in adjacent stories is the direct approach, a simple first step might be to plot and compare the inter-story drifts if analysis results happen to be available.

The difference between "soft" and "weak" stories is the difference between stiffness and strength. A column may be slender but strong, or stiff but weak. A change in column size can affect strength and stiffness, and both need to be considered.

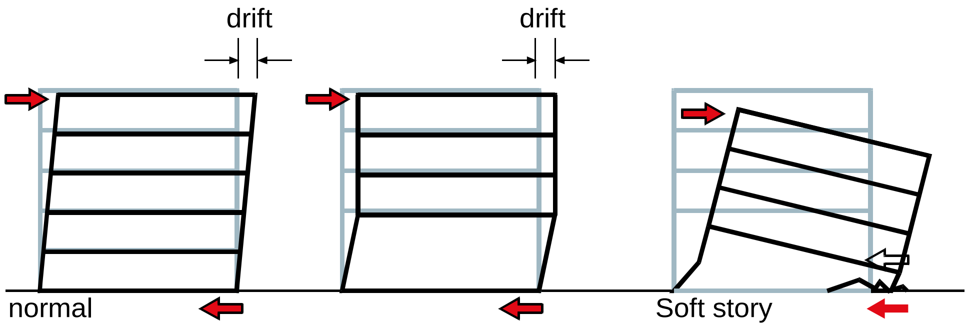


Fig 2.5 (a) Soft Storey due to Excessive Floor Height in Ground Storey

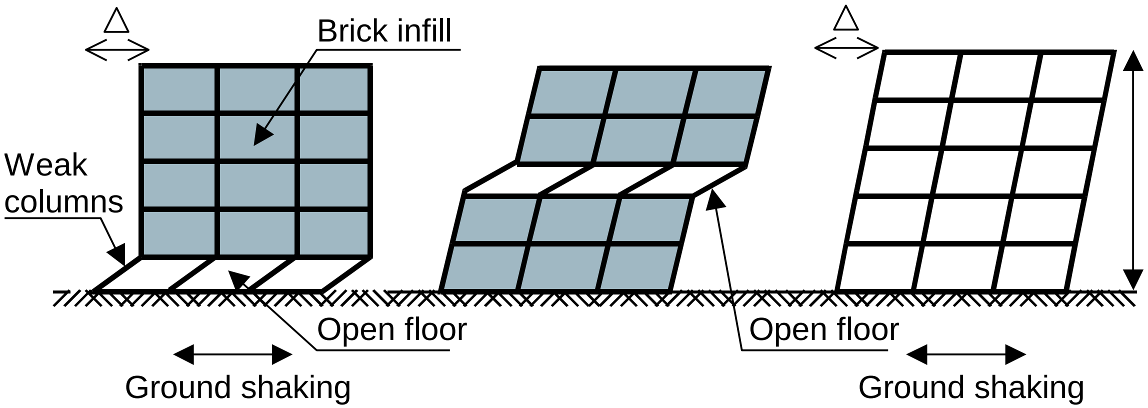


Fig 2.5 (b) Soft Storey due to Open Floors

Photo Courtesy: NSET

Photo Courtesy: NSET

Fig 2.5 (c) Soft Storey Problem due to Lack of Brick Infill in Ground Floor

Is there vertical discontinuity of shear walls or columns in ground or any other story?

Is there open ground or any other story?

Is the column or floor height of any one story is more than that of adjacent story?

If yes, there may be a problem of weak story or soft story.

Geometry

Geometric irregularities are usually detected in an examination of the story-to-story variation in the dimensions of the lateral-force-resisting system. A building with upper stories set back from a broader base structure is a common example. Another example is a story in a high-rise that is set back for architectural reasons. It should be noted that the irregularity of concern is in the dimensions of the lateral-force-resisting system, not the dimensions of the envelope of the building, and, as such, it may not be obvious.

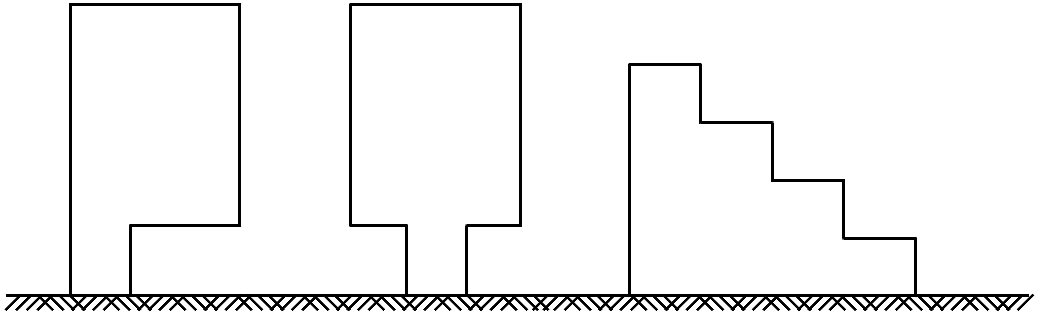


Fig 2.6 (a) Vertical Irregularity in Buildings

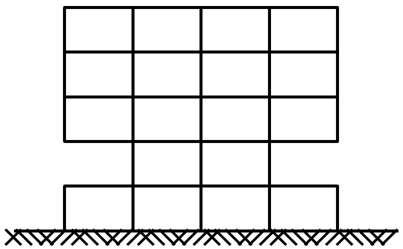


Fig 2.6 (b) Shear Walls in Cantilever

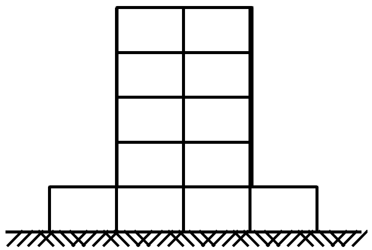


Fig 2.6 (c) Excessive Setback

Photo Courtesy: NSET

Photo Courtesy: NSET

Fig 2.6 (d)Vertically Irregular Building

Are the shear walls or the columns of a storey setback as compared to the adjacent storey?

Are the shear walls or the columns of a storey placed in projected parts as compared to the adjacent stories.

If yes, there is problem of vertical irregularity

Vertical Discontinuities

Vertical discontinuities are usually detected by visual observation. The most common example is a discontinuous columns or masonry shear wall. The element is not continuous to the foundation but stops at an upper level. The shear at this level is transferred through the diaphragm to other resisting elements below.

This issue is a local strength and ductility problem below the discontinuous element, not a global story strength or stiffness irregularity. The concern is that the wall or frame may have more shear capacity than considered in the design.

Is there any column or shear wall that is not continuing to the foundation? If so, that is vertical discontinuities.

Mass

Mass irregularities can be detected by comparison of the story weights. The effective mass consists of the dead load of the structure to each level, plus the actual weights of partitions and permanent equipment at each floor. The validity of this approximation is dependent upon the vertical distribution of mass and stiffness in the building.

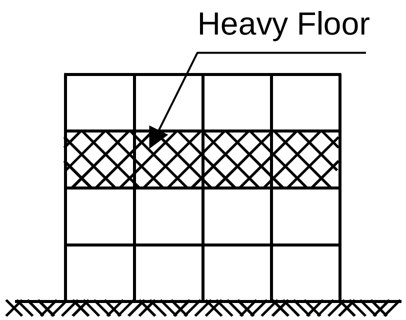


Fig 2.7 Mass Irregularity

Are there heavy walls as compared to the adjacent stories?

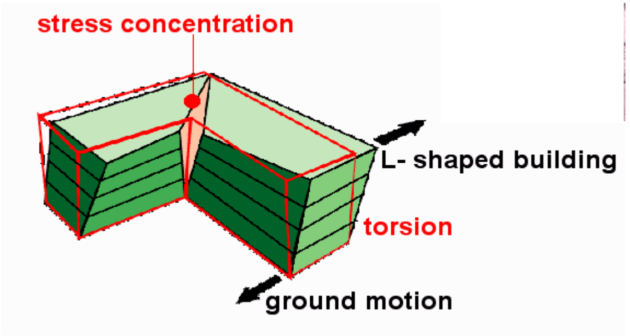
Are there heavy equipments as compared to that in the adjacent stories?

Is the thickness of the floor diaphragm more than that of the adjacent floor?

Is the mass due to all structural and non-structural components in story is less or more than 50% of that of the adjacent stories

Torsion

Whenever there is significant torsion in a building, the concern is for additional seismic demands and lateral drifts imposed on the vertical elements by rotation of the diaphragm. Buildings can be designed to meet code forces including torsion, but buildings with severe torsion are less likely to perform well in earthquakes. It is best to provide a balanced system at the start, rather than design torsion into the system.



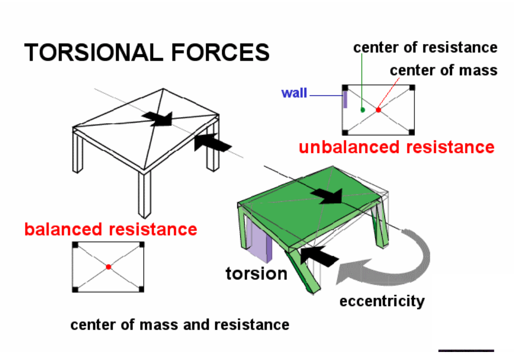


Fig 2.8 Effect of Torsion in Building

Condition of Materials

Deteriorated structural materials may reduce the capacity of the vertical- and lateral-force-resisting systems. The most common type of deterioration is caused by the intrusion of water. Stains may be a clue to water-caused deterioration where the structure is visible on the exterior, but the deterioration may be hidden where the structure is concealed by finishes. In the latter case, the assessment team may have to find a way into attics, plenums, and crawl spaces in order to assess the structural systems and their condition.

Deterioration of Wood

The condition of the wood in a structure has a direct relationship as to its performance in a seismic event. Wood that is split, rotten, or has insect damage may have a very low capacity to resist loads imposed by earthquakes. Structures with wood elements depend to a large extent on the connections between members. If the wood at a bolted connection is split, the connection will possess only a fraction of the capacity of a similar connection in undamaged wood.

Deterioration of Concrete

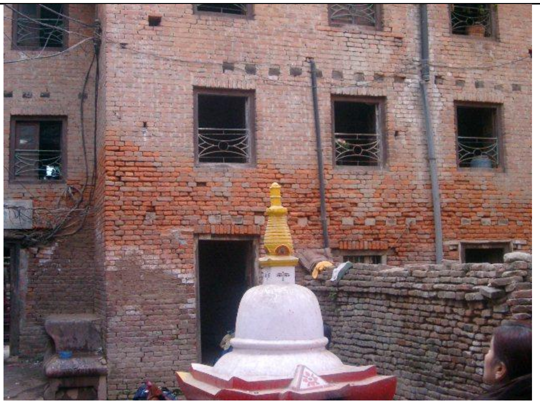
Deteriorated concrete and reinforcing steel can significantly reduce the strength of concrete elements. This statement is concerned with deterioration such as spalled concrete associated with rebar corrosion and water intrusion. Crack in concrete is another problem. Spalled concrete over reinforcing bars reduces the available surface for bond between the concrete and steel. Bar corrosion may significantly reduce the cross section of the bar.

Deterioration is a concern when the concrete cover has begun to spall, and there is evidence of rusting at critical locations.

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 2.9 (a) Delamination due to Seepage of Water	**Fig 2.9 (b) Rusting of Steel Bar**
Problem due to Concrete Deterioration	

Masonry Units and Joints

Deteriorated or poor quality masonry elements can result in significant reductions in the strength of structural elements. Older buildings constructed with lime mortar may have surface re-pointing but still have deteriorated mortar in the main part of the joint. Mortar that is severely eroded or can easily be scraped away has been found to have low shear strength, which results in low wall strength.



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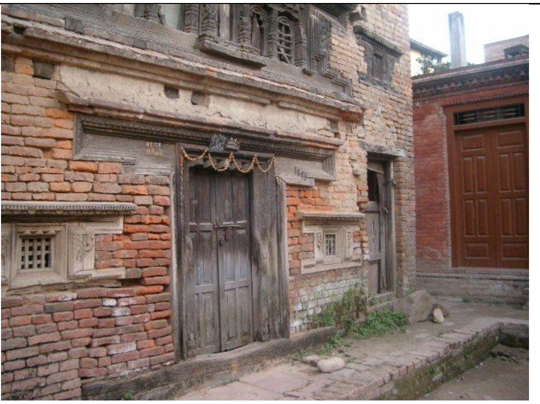


Photo Courtesy: NSET

Fig 2.10 Problem due to Deterioration of Masonry Units and Joints

Unreinforced Masonry Wall Cracks

Diagonal wall cracks, especially along the masonry joints, may affect the interaction of the masonry units, leading to a reduction of strength and stiffness. The cracks may indicate distress in the wall from past seismic events, foundation settlement, or other causes.

Crack width is commonly used as a convenient indicator of damage to a wall, but it should be noted that other factors, such as location, orientation, number, distribution and pattern of the cracks to be equally important in measuring the extent of damage present in the shear walls. All these factors should be considered when evaluating the reduced capacity of a cracked element.

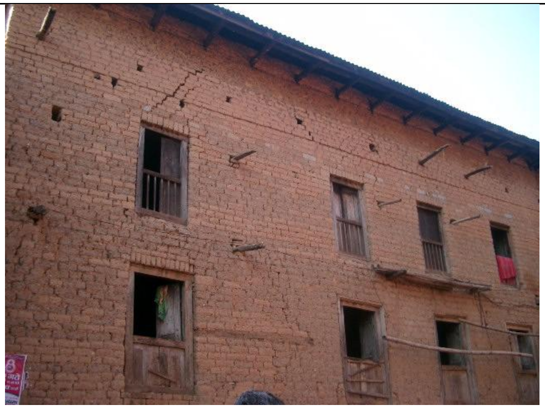


Photo Courtesy: NSET

Photo Courtesy: NSET

Fig 2.11 Problem due to Crack in Brick Wall

Cracks in Boundary Columns

Small cracks in concrete elements have little effect on strength. A significant reduction in strength is usually the result of large displacements or crushing of concrete. Only when the cracks are large enough to prevent aggregate interlock or have the potential for buckling of the reinforcing steel does the adequacy of the concrete element capacity become a concern.

Columns are required to resist diagonal compression strut forces that develop in infill wall panels. Vertical components induce axial forces in the columns. The eccentricity between horizontal components and the beams is resisted by the columns. Extensive cracking in the columns may indicate locations of possible weakness. Such columns may not be able to function in conjunction with the infill panel as expected.

Factors Associated with Lateral Force Resisting System of Different Buildings Influencing the Seismic Performance

Moment Frames

Moment frames develop their resistance to lateral forces through the flexural strength and continuity of beam and column elements. In an earthquake, a frame with suitable proportions and details can develop plastic hinges that will absorb energy and allow the frame to survive actual displacements that are larger than calculated in an elastic-based design.

In modern moment frames, the ends of beams and columns, being the locations of maximum seismic moment, are designed to sustain inelastic behavior associated with plastic hinging over many cycles and load reversals. Frames that are designed and detailed for this ductile behavior are called "Ductile Moment Resisting Frames".

Moment Frames with Infill Walls

Infill walls used for partitions, cladding or shaft walls that enclose stairs and elevators should be isolated from the frames. If not isolated, they will alter the response of the frames and change the behavior of the entire structural system. Lateral drifts of the frame will induce forces on walls that interfere with this movement. Cladding connections must allow for this relative movement. Stiff infill walls confined by the frame will develop compression struts that will impart loads to the frame and cause damage to the walls. This is particularly important around stairs or other means of egress from the building.

Interfering Walls

When an infill wall interferes with the moment frame, the wall becomes an unintended part of the lateral-force-resisting system. Typically these walls are not designed and detailed to participate in the lateral-force-resisting system and may be subject to significant damage. Interfering walls should be checked for forces induced by the frame, particularly when damage to these walls can lead to falling hazards near means of egress. The frames should be checked for forces induced by contact with the walls, particularly if the walls are not full height, or do not completely infill the bay.

Wall Connections

Performance of frame buildings with masonry infill walls is dependent upon the interaction between the frame and infill panels. In-plane lateral force resistance is provided by a compression strut developing in the infill panel that extends diagonally between corners of the frame. If gaps exist between the frame and infill, this strut cannot be developed. If the infill panels separate from the frame due to out-of-plane forces, the strength and stiffness of the system will be determined by the properties of the bare frame, which may not be detailed to resist seismic forces. Severe damage or partial collapse due to excessive drift and p-delta effects may occur.

A positive connection is needed to anchor the infill panel for out-of-plane forces. In this case, a positive connection can consist of a fully grouted bed joint in full contact with the frame, or complete encasement of the frame by the brick masonry.

	
Photo Courtesy: NSET	Photo Courtesy: NSET
Fig 2.12 (a)Separation of Infill Wall from Frame using Flexible Material	**Fig 2.12 (b)Tying of Infill Wall with Frame**

Concrete Moment Frames

Concrete moment frame buildings typically are more flexible than shear wall buildings. This flexibility can result in large inter-story drifts that may lead to extensive nonstructural damage. If a concrete column has a capacity in shear that is less than the shear associated with the flexural capacity of the column, brittle column shear failure may occur and result in collapse.

The following are the characteristics of concrete moment frames that have demonstrated acceptable seismic performance:

- Brittle failure is prevented by providing a sufficient number of beam stirrups, column ties, and joint ties to ensure that the shear capacity of all elements exceeds the shear associated with flexural capacity,
- Concrete confinement is provided by beam stirrups and column ties in the form of closed hoops with 135-degree hooks at locations where plastic hinges will occur.

- Overall performance is enhanced by long lap splices that are restricted to favorable locations and protected with additional transverse reinforcement.
- The strong column/weak beam requirement is achieved by suitable proportioning of the members and their longitudinal reinforcing.

All these detailing result in ductile response of moment-resisting-frame buildings in lateral loading of earthquakes.

Shear Stress Check

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

Axial Stress Check

Columns that carry a substantial amount of gravity load may have limited additional capacity to resist seismic forces. When axial forces due to seismic overturning moments are added, the columns may crush in a non-ductile manner due to excessive axial compression.

Flat Slab Frames

The concern is the transfer of the shear and bending forces between the slab and column, which could result in a punching shear failure and partial collapse. The flexibility of the lateral-force-resisting system will increase as the slab cracks.

Short Captive Columns

Short captive columns tend to attract seismic forces because of high stiffness relative to other columns in a story. Captive column behavior may also occur in buildings with clerestory windows, or in buildings with partial height masonry infill panels.

If not adequately detailed, the columns may suffer a non-ductile shear failure which may result in partial collapse of the structure.

A captive column that can develop the shear capacity to develop the flexural strength over the clear height will have some ductility to prevent sudden non-ductile failure of the vertical support system.

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 2.13 (a)Beam at Mid Height of Column	**Fig 2.13 (b)Ventilator Attached to Frame**
Problem due to Short Column if not Properly Considered	

No Shear Failures

If the shear capacity of a column is reached before the moment capacity, there is a potential for a sudden non-ductile failure of the column, leading to collapse.

Columns that cannot develop the flexural capacity in shear should be checked for adequacy against calculated shear demands. Note that the shear capacity is affected by the axial loads on the column and should be based on the most critical combination of axial load and shear.

Strong Column Weak Beam

When columns are not strong enough to force hinging in the beams, column hinging can lead to story mechanisms and a concentration of inelastic activity at a single level. Excessive story drifts may result in instability of the frame due to P-Δ effects. Good post-elastic behavior consists of yielding distributed throughout the frame. A story mechanism will limit forces in the levels above, preventing the upper levels from yielding.

The alternative procedure checks for the formation of a story mechanism. The story strength is the sum of the shear capacities of all the columns as limited by the controlling action. If the columns are shear critical, a shear mechanism forms at the shear capacity of the columns. If the columns are controlled by flexure, a flexural mechanism forms at a shear corresponding to the flexural capacity.

Beam Bars

The requirement for two continuous bars is a collapse prevention measure. In the event of complete beam failure, continuous bars will prevent total collapse of the supported floor, holding the beam in place by catenary action. Previous construction techniques used bent up longitudinal bars as reinforcement. These bars transitioned from bottom to top reinforcement at the gravity load inflection point. Some amount of continuous top and bottom reinforcement is desired because moments due to seismic forces can shift the location of the inflection point. Because non-compliant beams are vulnerable to collapse, the beams are required to resist demands at an elastic level.

Column Bar Splices

Located just above the floor level, column bar splices are typically located in regions of potential plastic hinge formation. Short splices are subject to sudden loss of bond. Widely spaced ties can result in a spalling of the concrete cover and loss of bond. Splice failures are sudden and non-ductile.

Beam Bar Splices

Lap splices located at the end of beams and in vicinity of potential plastic hinges may not be able to develop the full moment capacity of the beam as the concrete degrades during multiple cycles.

Column Tie Spacing

Widely spaced ties will reduce the ductility of the column, and it may not be able to maintain full moment capacity through several cycles. Columns with widely spaced ties have limited shear capacity and non-ductile shear failures may result.

Stirrup Spacing

Widely spaced stirrups will reduce the ductility of the beam, and it may not be able to maintain full moment capacity through several cycles. Beams with widely spaced stirrups have limited shear capacity and non-ductile shear failures may result.

Joint Reinforcing

Beam-column joints without shear reinforcement may not be able to develop the strength of the connected members, leading to a non-ductile failure of the joint. Perimeter columns are especially vulnerable because the confinement of joint is limited to three sides (along the exterior) or two sides (at a corner).

Joint Eccentricity

Joint eccentricities can result in high torsional demands on the joint area, which will result in higher shear stresses.

Stirrup and Tie Hooks

To be fully effective, stirrups and ties must be anchored into the confined core of the member. 90^{o} hooks that are anchored within the concrete cover are unreliable if the cover spalls during plastic hinging. The amount of shear resistance and confinement will be reduced if the stirrups and ties are not well anchored.

Unreinforced Masonry Shear Walls

Shear Stress Check

The shear stress check provides a quick assessment of the overall level of demand on the structure. The concern is the overall strength of the building.

Proportions

Slender unreinforced masonry bearing walls with large height-to-thickness ratios or large length-to- thickness ratio have a potential for damage due to out-of-plane forces which may result in falling hazards and potential collapse of the structure.

<table>
<tr><td>
Photo Courtesy: NSET</td><td>

Photo Courtesy: NSET</td></tr>
<tr><td>Fig 2.14 (a)Long Unsupported Wall</td><td>Fig 2.14 (b)Slender Wall</td></tr>
<tr><td colspan="2">Problem due to Inadequate Proportions of Load Bearing Walls</td></tr>
</table>

Position of Openings

Openings attached to load bearing masonry walls and too large openings reduce both out-of-plane and in plane stability of the building

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 2.15 (a)Large Window Openings in Masonry Walls	**Fig 2.15 (b)Window Attached to Wall**
Problem due to Openings	

Masonry Lay-up

When walls have poor collar joints, the inner and outer wythes will act independently. The walls may be inadequate to resist out-of-plane forces due to a lack of composite action between the inner and outer wythes. Mitigation to provide out-of-plane stability and anchorage of the wythes may be necessary to achieve the selected performance level.

Solid Walls

When the walls are of cavity construction, the inner and outer wythes will act independently due to a lack of composite action, increasing the potential for damage from out-of-plane forces. Failure of these walls out-of-plane will result in falling hazards and degradation of the strength and stiffness of the lateral force resisting system.

Mitigation to provide out-of-plane stability and anchorage of the wythes is necessary to achieve the selected performance level.

Earthquake Resistant Element

Unreinforced Masonry walls have very low (almost neglible) tension resisting capacity. Hence, the presence of bands at lintel, sill and roof level, corner stitches, vertical reinforcements at corners and junctions of wall mitigate the damage due to tension and shear cracks.

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 2.16 (a)Vertical Reinforcement and Corner Stitch	**Fig 2.16 (b)Presence of Sill Band, Corner Stitch and Lintel Band**
Presence of Earthquake Resisting Element	

Factors Associated with Diaphragms

General

Diaphragms are horizontal elements that distribute seismic forces to vertical lateral force resisting elements. They also provide lateral support for walls and parapets. Diaphragm forces are derived from the self weight of the diaphragm and the weight of the elements and components that depend on the diaphragm for lateral support. Any roof, floor, or ceiling can participate in the distribution of lateral forces to vertical elements up to the limit of its strength. The degree to which it participates depends on relative stiffness and on connections. In order to function as a diaphragm, horizontal elements must be interconnected to transfer shear, with connections that have some degree of stiffness.

An important characteristic of diaphragms is flexibility, or its opposite, rigidity. In seismic design, rigidity means relative rigidity. Of importance is the in-plane rigidity of the diaphragm relative to the walls or frame elements that transmit the lateral forces to the ground.

Diaphragm Continuity

Split level floors and roofs, or diaphragms interrupted by expansion joints, create discontinuities in the diaphragm. It is a problem unless special details are used, or lateral-force-resisting elements are provided at the vertical offset of the diaphragm or on both sides of the expansion joint. Such a discontinuity may cause the diaphragm to function as a cantilever element or three-sided diaphragm. If the diaphragm is not supported on at least three sides by lateral-force-resisting elements, torsional forces in the diaphragm may cause it to become unstable.

 Photo Courtesy: NSET	 Photo Courtesy: NSET
Fig 2.17 (a) Drop in Floor Slab	**Fig 2.17 (b) Large Diaphragm Opening Overlooking a Living Room Below**
Problem due to Discontinuity in Floor Diaphragm	

Openings at Shear Walls and Exterior Masonry Shear Walls

Large openings at shear walls significantly limit the ability of the diaphragm to transfer lateral forces to the wall. This can have a compounding effect if the opening is near one end of the wall and divides the diaphragm into small segements with limited stiffness that are ineffective in transferring shear to the wall. Large openings may also limit the ability of the diaphragm to provide out-of-plane support for the wall.

Plan Irregularities

Diaphragms with plan irregularities such as extending wings, plan insets, or E-, T-, X-, L-, or C-shaped configurations have re-entrant corners where large tensile and compressive forces can develop. The diaphragm may not have sufficient strength at these re-entrant corners to resist these tensile forces and local damage may occur.

Annex III: Vulnerability Factors Identification Checklist

Vulnerability Factors Identification

Appropriate checklists for different types of buildings are given in this section. Checklists available for certain building types are taken from FEMA 310, *Handbook for the Seismic Evaluation of Buildings*, and IS Guidelines for Seismic Evaluation and Strengthening of Existing Building. Checklists for some building types, which are not included in FEMA 310 and IS Guidelines are developed as per Nepal National Building Code. The checklist covers the basic vulnerability factors related to building systems, lateral force resisting systems, connections and diaphragms which will be evaluated mostly based on visual observation.

Structural Assessment Checklist for Type 1 Buildings (Adobe, Stone in Mud, Brick in Mud)

Building System

C NC N/A SHAPE: The building shall be symmetrical in plan and regular in elevation.

C NC N/A PROPORTION IN PLAN: The breadth to length ratio of the building shall be within 1:3. The breadth to length ratio of any room or area enclosed by load bearing walls inside the building shall be also within 1:3. The building height shall be not more than three times the width of the building.

C NC N/A STOREY HEIGHT: The floor to floor height of the building shall be in between 2-3 m.

C NC N/A NUMBER OF STOREYS: The building shall be up to two storeys only.

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

C NC N/A SLOPING GROUND: The slope of the ground where the building lies shall not be more than 20^{o} (1:3, vertical: horizontal)

C NC N/A PLUMBLINE: Walls of the foundation and superstructure shall be true to plumb line and the width of the wall shall be uniform.

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

C NC N/A THROUGH-STONES: In case of stone building, the walls shall have plenty of through-stones extending the whole width of the walls. The maximum spacing of such through-stones shall be within 1.2m horizontally and 0.6m vertically.

C NC N/A WALL THICKNESS: The minimum wall thickness for different storey heights shall not be less than

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

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

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

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

C NC N/A POSITION OF OPENINGS: Openings shall not be located at corners or junctions of a wall. Openings shall not be placed closer to an internal corner of a wall than half the opening height or 1.5 times the wall thickness, whichever is greater. The width of pier between two openings shall not be less than half of the opening height or 1.5 times the wall thickness, whichever is greater. The vertical distance between two openings shall not be less than 0.6m or half the width of the smaller opening, whichever is grater.

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

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

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

C NC N/A TORSION: The estimated distance between the 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.

C NC N/A MASONRY UNITS: There shall be no visible deterioration of masonry units.

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

C NC N/A MASONRY LAY-UP: Filled collar joints of multiwythe masonry walls shall have negligible voids.

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

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

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

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

Lateral Force Resisting System

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

Diaphragms

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

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

Geologic Site

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

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

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

Structural Assessment Checklist for Type 2 Buildings (Brick in Cement Buildings and Stone in Cement Buildings)

Building System

C NC N/A NK 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.

C NC N/A NK REDUNDANCY: The number of lines of vertical lateral load resisting elements in each principal direction shall be greater than or equal to 2. Similarly, the number of lines of shear walls in each direction shall be greater than or equal to 2.

C NC N/A NK 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.

C NC N/A NK MEZZANINES/LOFT/SUBFLOORS: Interior mezzanine/loft/sub-floor levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure.

C NC N/A NK WEAK STORY: The strength of the vertical lateral force resisting system in any storey shall not be less than 70% of the strength in an adjacent story.

C NC N/A NK SOFT STORY: The stiffness of the vertical lateral load resisting system in any storey shall not be less than 60% of the stiffness in an adjacent story above or less than 70% of the average stiffness of the three storey above.

C NC N/A NK VERTICAL DISCONTINUITIES: All vertical elements in the lateral force resisting system shall be continuous from the root to the foundation.

C NC N/A NK MASS: There shall be no change in effective mass more than 100% from one storey to the next. Light roofs, penthouse, and mezzanine floors need not be considered.

C NC N/A NK TORSION: The estimated distance between the 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.

C NC N/A NK 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, expect for buildings that are of the same height with floors located at the same levels.

C NC N/A NK 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.

C NC N/A NK MASONRY UNITS: There shall be no visible deterioration of masonry units.

C NC N/A NK MASONRY JOINTS: The mortar shall not be easily scraped away from the joints by hand with a metal tool, and there shall be no areas of eroded mortar.

C NC N/A NK UNREINFORCED MASONRY WALL CRACKS: There shall be no existing diagonal cracks in wall elements greater than 1/8" for Life Safety and 1/16" for Immediate Occupancy or out-of-plane offsets in the bed joint greater than 1/8" for Life Safety and 1/16" for Immediate Occupancy.

Lateral Load Resisting System

C NC N/A NK SHEAR STRESS IN SHEAR WALLS: Average shear stress in masonry shear walls, t_{Wall} shall be calculated as per 6.5.2 of IITK- GSDMA guidelines for seismic evaluation and strengthening of buildings. For unreinforced masonry load bearing wall building, the average shear stress, t_{Wall} shall be less than 0.10MPa.

C NC N/A NK HEIGHT TO THICKNESS RATIO: The unreinforced masonry wall height-to-thickness ratios shall be less than the following.

Top storey of multi storey building:	9
First storey of multi storey building:	15
All other conditions:	13

C NC N/A NK MASONRY LAY UP: Filled collar joints of multi wythe masonry walls shall have negligible voids.

C NC N/A NK WALL ANCHORAGE: Walls shall be properly anchored to diaphragms for out of plane forces with anchor spacing of 1.2 m or less.

C NC N/A NK CONNECTIONS: Diaphragms shall be reinforced and connected to transfer of loads to the shear walls.

C NC N/A NK OPENINGS IN DIAPHRAGMS NEAR SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls shall be less than 25% of the wall length.

C NC N/A NK OPENINGS IN DIAPHRAGMS NEAR EXTERIOR MASONRY SHEAR WALLS: Diaphragm opening immediately adjacent to exterior masonry shear walls not be greater than 2.5 m.

C NC N/A NK PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other location of plan irregularities.

C NC N/A NK DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragm opening larger than 50% of the building width in either major plan dimension.

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

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

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

C NC N/A NK GABLE BAND: If the roof is slopped roof, gable band shall be provided to the building.

C NC N/A NK DIAGONAL BRACING: If there is flexible diaphragms such as joists and rafters shall be diagonally braced and each crossing of a joist/rafter and a brace shall be properly fixed.

C NC N/A NK LATERAL RESTRAINERS: For flexible roof and floor, each joists and rafters shall be restrained by timber keys in both sides of wall.

Additional Factors for Stone Buildings

C NC N/A NK NUMBER OF STOREYS: The number of storeys of the stone building shall be limited to 2.

C NC N/A NK UNSUPPORTED WALL LENGTH: The maximum unsupported length of a wall between cross-walls shall be limited to 5m.

C NC N/A NK THROUGH-STONES: In case of stone building, the walls shall have plenty of through-stones extending the whole width of the walls. The maximum spacing of such through-stones shall be within 1.2m horizontally and 0.6m vertically.

Geologic Site

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

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

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

Structural Assessment Checklist for Type 3 and 4 Reinforced Concrete Moment-Resisting-Frame Buildings

Building System

C NC N/A NK 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.

C NC N/A NK REDUNDANCY: The number of lines of vertical lateral load resisting elements in each principle direction shall be greater than or equal to 2.

C NC N/A NK 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.

C NC N/A NK MEZZANINES/LOFT/SUBFLOORS: Interior mezzanine/loft/sub-floor levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure.

C NC N/A NK WEAK STORY: The strength of the vertical lateral force resisting system in any storey shall not be less than 70% of the strength in an adjacent story.

C NC N/A NK SOFT STORY: The stiffness of vertical lateral load resisting system in any storey shall not be less than 60% of the stiffness in an adjacent story or less than 70% of the average stiffness of the three storey above.

C NC N/A NK VERTICAL DISCONTINUITIES: All vertical elements in the lateral force resisting system shall be continuous from the root to the foundation.

C NC N/A NK MASS: There shall be no change in effective mass more than 100% from one storey to the next. Light roofs, penthouse, and mezzanine floors need not be considered.

C NC N/A NK TORSION: The estimated distance between the 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.

C NC N/A NK 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, expect for buildings that are of the same height with floors located at the same levels.

C NC N/A NK FLAT SLAB FRAMES: The lateral-force-resisting system shall not be a frame consisting of columns and a flat slab/plate without beams.

C NC N/A NK SHORT COLUMNS: The reduced height of a columns due to surrounding parapet, infill wall, etc. shall not be less than five times the dimension of the column in the direction of parapet, infill wall, etc. or 50% of the nominal height of the typical columns in that storey.

C NC N/A NK 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.

C NC N/A NK CRACKS IN BOUNDARY COLUMNS: There shall be no existing diagonal cracks wider than 3 mm in concrete columns that encase masonry infills.

Lateral Load Resisting System

C NC N/A NK SHEAR STRESS IN RC FRAME COLUMNS: The average shear stress in concrete columns τ_{col}, computed in accordance with 6.5.1 of IITK- GSDMA guidelines for seismic evaluation and strengthening of buildings shall be lesser of 0.4MPa and 0.10 $\sqrt{f_{ck}}$

C NC N/A NK AXIAL STRESS IN MOMENT FRAMES: The maximum compressive axial stress in the columns of moments frames at base due to overturing forces alone (Fo) as calculated using 6.5.4 equation of IITK- GSDMA guidelines for seismic evaluation and strengthening of buildings shall be less than $0.25f_{ck}$

C NC N/A NK NO SHEAR FAILURES: Shear capacity of frame members shall be adequate to develop the moment capacity at the ends, and shall be in accordance with provision of IS: 13920 for shear design of beams and columns.

C NC N/A NK CONCRETE COLUMNS: All concrete columns shall be anchored into the foundation.

C NC N/A NK STRONG COLUMN/WEAK BEAM: The sum of the moments of resistance of the columns shall be at least 1.1 times the sum of the moment of resistance of the beams at each frame joint.

C NC N/A NK BEAM BARS: At least two longitudinal top and two longitudinal bottom bars shall extend continuously through out the length of each frame beam. At least 25% of the longitudinal bars located at the joints for either positive or negative moment shall be continuous throughout the length of the members.

C NC N/A NK COLUMNS BAR SPLICES: Lap splices shall be located only in the central half of the member length. It should be proportions as a tension splice. Hoops shall be located over the entire splice length at spacing not exceeding 150 mm centre to centre. Not more than 50% of the bars shall preferably be spliced at one section. If more than 50 % of the bars are spliced at one section, the lap length shall be 1.3Ld where Ld is the development length of bar in tension as per IS 456:2000

C NC N/A NK BEAM BAR SPLICES: Longitudinal bars shall be spliced only if hoops are located over the entire splice length, at a spacing not exceeding 150mm. The lap length shall not be less than the bar development length in tension. Lap splices shall not be located (a) within a joint, (b) within a distance of 2d from joint face, and (c) within a quarter length of the member where flexural yielding may occur under the effect of earthquake forces. Not more than 50% of the bars shall be spliced at one section.

C NC N/A NK COLUMN TIE SPACING: The parallel legs of rectangular hoop shall be spaced not more than 300mm centre to centre. If the length of any side of the hoop exceeds 300mm, the provision of a crosstie should be there. Alternatively, a pair of overlapping hoops may be located within the column. The hooks shall engage peripheral longitudinal bars.

C NC N/A NK STIRRUP SPACING: The spacing of stirrups over a length of 2d at either end of a beam shall not exceed (a) d/4, or (b) 8 times the diameter of the smallest longitudinal bar; however, it need not be less than 100 mm. The first hoop shall be at a distance not exceeding 50 mm from the joint face. In case of beams vertical hoops at the same spacing as above shall also be located over a length equal to 2d on either side of a section where flexural yielding side of a section where flexural yielding may occur under the effect of earthquake forces. Elsewhere, the beam shall have vertical hoops at a spacing not exceeding d/2.

C NC N/A NK JOINT REINFORCING: Beam- column joints shall have ties spaced at or less than 150 mm.

C NC N/A NK STIRRUP AND TIE HOOKS: The beam stirrups and column ties shall preferably be anchored into the member cores with hooks of 135^0

C NC N/A NK JOINT ECCENTRICITY: There shall be no eccentricities larger than 20% of the smallest column plan dimension between girder and column centerlines. This statement shall apply to the Immediate Occupancy Performance Level only.

C NC N/A NK WALL CONNECTIONS: All infill walls shall have a positive connection to the frame to resist out-of-plane forces.

C NC N/A NK INTERFERING WALLS: All infill walls placed in moment frames shall be isolated from structural elements.

Diaphragms

C NC N/A DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors. In wood buildings, the diaphragms shall not have expansion joints.

C NC N/A PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only.

C NC N/A DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragms openings larger than 50% of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only.

Geologic Site

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

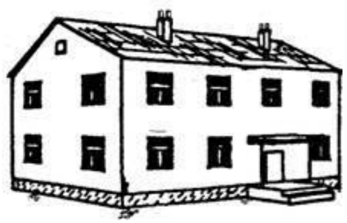
C NC N/A NK LIQUEFACTION: Liquefaction susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils.

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

Annex IV: Damage Grades of Buildings

Classification from European Macro-seismic Scale (EMS 98)

Table 4.1 Classification of Damage to Masonry Buildings

 **Grade 1: Negligible to slight damage**	**Structural damage : No** **Non-structural damage: Slight** • Hair-line cracks in very few walls. • Fall of small pieces of plaster only. • Fall of loose stones from upper parts of buildings in very few cases.
 **Grade 2: Moderate damage**	**Structural damage : Slight** **Non-structural damage: Moderate** • Cracks in many walls. • Fall of fairly large pieces of plaster. • Partial collapse of chimneys.

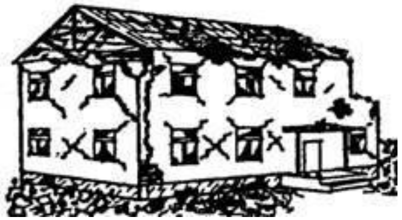
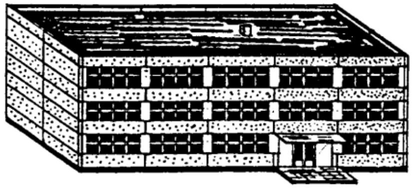
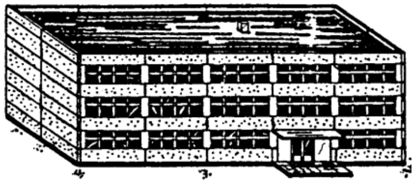
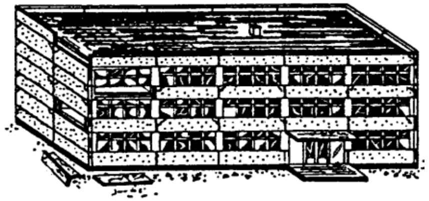
 **Grade 3: Substantial to heavy damage**	**Structural damage: Moderate** **Non-structural damage: Heavy** • Large and extensive cracks in most walls. • Roof tiles detach. • Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).
 **Grade 4: Very heavy damage**	**Structural damage: Heavy** **Non-structural damage: Very heavy** • Serious failure of walls; partial structural failure of roofs and floors.

Table 4.2 Classification of Damage to RC Frame Buildings

 **Grade 5: Destruction**	**Structural damage: very heavy** • Total or near total collapse.
 **Grade 1: Negligible to slight damage**	**Structural damage : No** **Non-structural damage: Slight** • Fine cracks in plaster over frame members or in walls at the base. • Fine cracks in partitions and infills.
 **Grade 2: Moderate damage**	**Structural damage : Slight** **Non-structural damage: Moderate** • Cracks in columns and beams of frames and in structural walls. • Cracks in partition and infill walls; fall of brittle cladding and plaster. • Falling of mortar from the joints of wall panels.

 **Grade 3: Substantial to heavy damage**	**Structural damage: Moderate** **Non-structural damage: Heavy** • Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. • Spalling of concrete cover, buckling of reinforced bars. • Large cracks in partition and infill walls, failure of individual infill panels.
 **Grade 4: Very heavy damage**	**Structural damage: Heavy** **Non-structural damage: Very heavy** • Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns. • Collapse of a few columns or of a single upper floor.
 **Grade 5: Destruction**	**Structural damage: very heavy** • Collapse of ground floor or parts (e.g. wings) of buildings.

Annex V: Modified Mercally Intensity Scale (MMI Scale)

Intensity	Description of Effect
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.

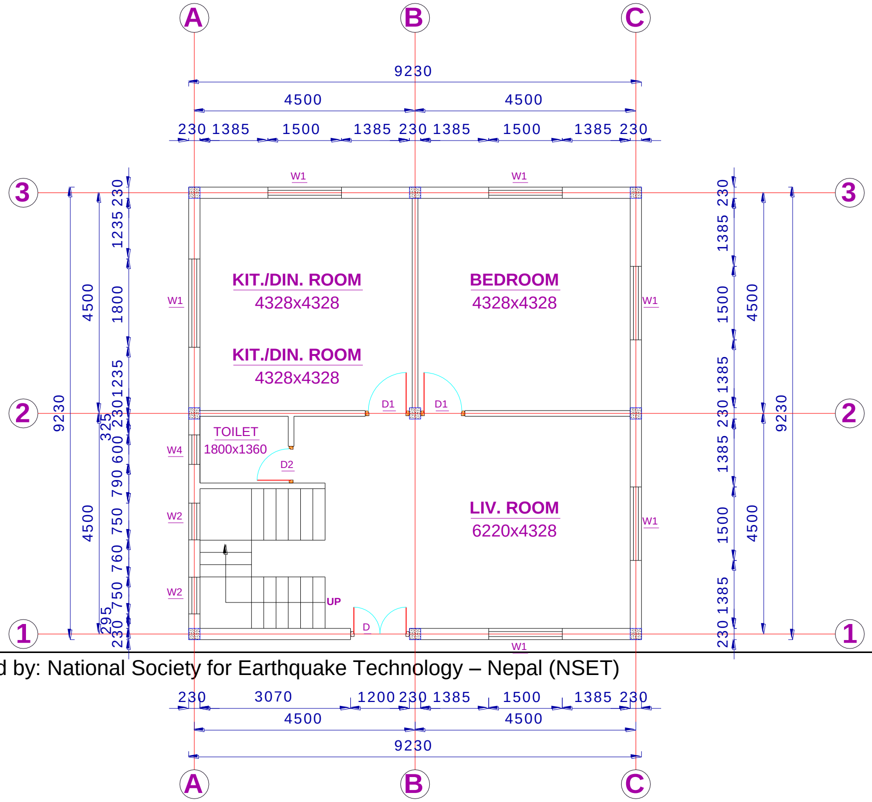
IX	**Destructive Intensity** • People would be forcibly thrown/fall down. Chaos, fear and confusion will be extreme. • Most building structures would be destroyed and intensely damaged. Bridges and high structures would fall and be destroyed. Posts, towers and monuments may bend or completely be destroyed. 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.
XI	**Devastative Intensity** • Severe damage even to well built buildings, bridges, water dams and railway lines; highways become useless; underground pipes destroyed.
XII	**Extremely Destructive Intensity (Landscape changes)** • Practically all structures above and below ground are greatly damaged or destroyed.

ANNEX VI : Example 1: Seismic Evaluation of Reinforced Concrete Moment Resisting Frame Building

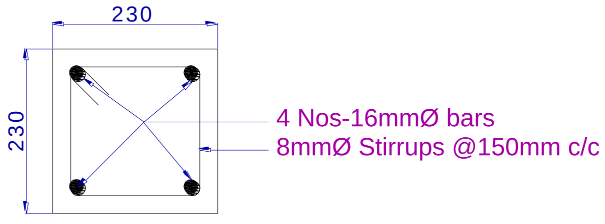
1.1 Building Description

Building Type	:	Residential Building
No. of Stories	:	Three
Storey Height	:	3 m
Floor/Roof	:	RCC 125 mm thick Slab
Parapet Wall Height	:	1 m
Earthquake Zone	:	1 (NBC 105)
		Seismic Zone V according to IS code
Importance Factor	:	1.0 (Residential Building)
Building Dimension	:	9.0 m X 9.0 m
		Two bay each of 4.5 m span both direction
Lateral load resisting element	:	9 Columns of 230 mm X 230 mm size reinforced with 4
		nos. 16 mm dia verttical bars and 8 mm dia. Stirrups @ 150 mm c/c throughout the length of column
		Beam in every floor is of size 230 mm X 350 mm including slab thickness reinforced with
		4 nos. 16 mm dia. (Top bars)
		3 nos. 16 mm dia. (Bottom bars)
		8 mm dia. Stirrup @ 150 mm c/c throughout

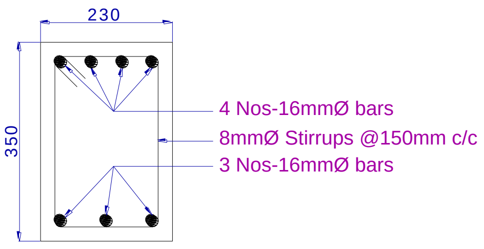
1.2 Building Drawing



Ground Floor Plan

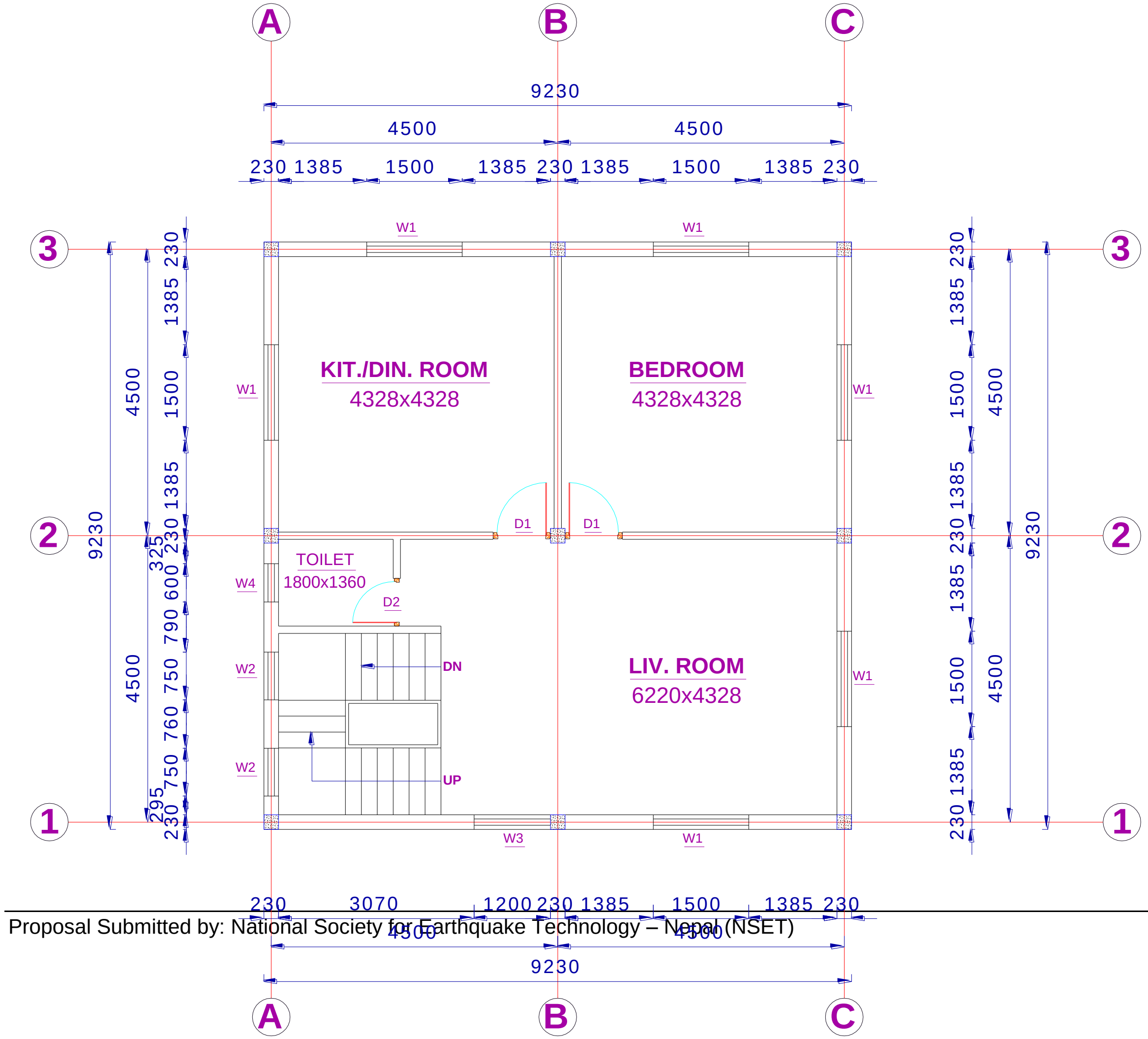


Column Detail



Beam Detail

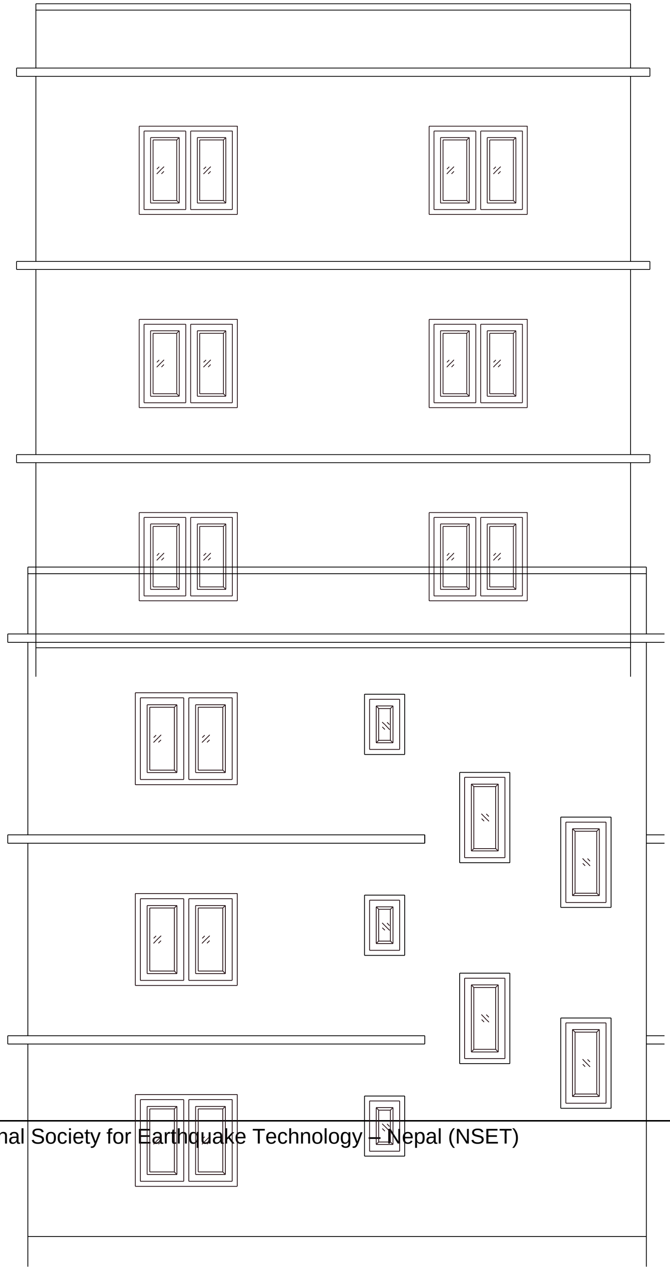
1st and 2nd Floor Plan



Front Elevation

Side Elevation

Back Elevation



Side Elevation

The following is a sample of quick check calculations based on FEMA 310 for the seismic evaluation of building and IITK-GSDMA Guidelines for seismic evaluation and strengthening of buildings.

1.3 Assumptions:

- Unit weight of RCC = 25 kN/m^3
- Unit weight of brick = 19 kN/m^3
- Live load = 2.5 kN/m^2
- Weight of plaster and floor finish = 1.0 kN/m^2
- Grade of concrete = M20 for all other structural elements
- Grade of steel = Fe 415
- Lateral load is solely carried by frame elements. Stiffness of the walls is not considered.

1.4 Calculation for Shear Stress check

Table 6.1.1 Summary of lumped load calculation

Level	Dead Load	Live load	25% Live Load	Seismic weight
3	659.54	121.50	30.38	689.92
2	833.91	202.50	50.63	884.53
1	833.91	202.50	50.63	884.53
				2458.98

1.5 Calculation of base shear (Using IS 1893: 2002)

The total design lateral force or design seismic base shear is given by

$V_b = A_h W$

Where,

W = Seismic weight of the building = 2458.98 kN

A_h =The design horizontal seismic force coefficient = Z I S_a / 2 R g
Where A_h will not be taken less than Z/2
Z = Zone factor = 0.36 (for Seismic Zone V)
I = Importance factor = 1.0
R = Response Reduction Factor = 3 for Ordinary RC Moment Resisting Frame
S_a/g =Average response acceleration coefficient, that depends upon natural period and damping of the structure
$T_a = 0.09h / \sqrt{d}$ The approximate fundamental natural period of vibration of building in seconds
h = Height of building in m = 9m
d = Base dimension of the building at the plinth level in m along the consideration direction of the lateral force.
When d = 9.0m T_a = 0.27 sec
For medium soil
$S_a/g = 2.5$ for $0.10 \leq T \leq 0.55$
A_h = 0.15
Base shear V_b = 368.85 kN

1.6 Distribution of base shear and calculation of shear stress in RC Columns

The design base shear (V_b) is distributed along the height of the building as per the following expression:

$Q_i = V_b (W_i h_i / \sum W_i h_i)$

Where Q_i = Design lateral force at floor i
W_i = Seismic weight of floor i
h_i = Height of floor i measured from base

Table 6.1.2 Base Shear Distribution

Floor	Total weight W_i (kN)	Height h_i(m)	$W_i h_i$	Q_i (kN)	Storey Shear V_i (kN)
3	689.92	9.00	6209.24	161.63	161.63

2	884.53	6.00	5307.19	138.15	299.77
1	884.53	3.00	2653.60	69.07	368.85
	3055.02		14170.03	368.85	

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings, 6.5.1)

Average shearing stress in columns is given as

$\tau_{col} = (n_c/(n_c-n_f))*(V_j/A_c) < $ min of 0.4Mpa and 0.1 $\in f_{ck}$

For ground storey columns,

n_c = Total no of columns resisting lateral forces in the direction of loading

n_f = Total no. of frames in the direction of loading

A_c = Summation of the cross-section area of all columns in the storey under consideration

V_j = Maximum storey shear at storey level 'j'

Table 6.1.3 Shear Stress at Storey Levels

Storey	n_c	n_{f2}	n_{f1}	A_c (m^2)	Storey shears Vj (kN)	Shear stress	
						$\tau_{col\ 1}$ (Mpa)	$\tau_{col\ 2}$ (Mpa)
3.00	9.00	3.00	3.00	0.48	161.63	0.51	0.51
2.00	9.00	3.00	3.00	0.48	299.77	0.94	0.94
1.00	9.00	3.00	3.00	0.48	368.85	1.15	1.15

But $\tau_{col} >$ min of 0.4Mpa and 0.1 $\in f_{ck}$

Hence, the check is not satisfied

1.7 Calculation of Shear capacity of column using capacity design method

Checking Shear Capacity of Center Column

Shear capacity of column required = $1.4(M^l+M^r)/h_{st}$

The Longitudinal Beam of size 230 x 350 is reinforced with 4-16& (804 mm^2, i.e 1.1%) at top and 3-16 (603 mm^2, i.e 0.83 %) at bottom.

Where,

b = 230 mm;   d=317 mm

The hogging and sagging moment capacities are evaluated as 76 kN-m and 57 kN-m respectively.

The shear force in column corresponding to these moments

$V_u = 1.4 (M_u^{bl} + M_u^{br})/h_{st} = 1.4 \times (76 + 57)/3.0 = 62.1$ kN

Center Column is of size 230mm x 230mm

b = 230mm;   d = 192 mm

A_s = 804 mm^2 (4-16&)

f_{ck} = 20 N/mm^2

f_y = 415 N/mm^2;

From SP:16 Table 61, for P_t = 1.52 %, τ_c = 0.56 N/mm^2

Shear capacity of concrete section = 0.56 * 230 * 230 / 1000 = 29.62 kN

Shear to be carried by stirrups V_{us} = 62.1 – 29.62 = 32.48 kN

From table 62, SP -16: for 8mm dia. stirrups @ 150mm c/c

For rectangular stirrups

V_{us} / d = 2.42 kN/cm

V_{us} provided = 2.42 * 19.2 = 46.5 kN > 32.48 kN

Hence, the check is satisfied for Center Column

1.8 Check for Confining Links in Column

The area of cross section, A_{sh}, of the bar forming rectangular hoop, to be used as special confining reinforcement shall not be less than

$A_{sh} = 0.18\ S\ h\ (f_{ck}/f_y)\ (A_g/A_k-1)$ as per IS 13920: 1993

Where,

h = longer dimension of the rectangular confining hoop measured to its outer face

A_k = area of confined concrete core in the rectangular hoop measured to its outside dimensions.

The size of inner core h = 230-60+16 = 186 (Considering cover of 30mm)

A_g = 230 * 230 = 52900
A_k = 186*186=34596
Hence,
50= 0.18 S 186 (20/415) (52900/34596 – 1.0)
S required = 58.6 mm
But need not be less than 75 mm

1.9 Axial Stress check

1.9.1 The Axial Stress due to GravityLoads as per FEMA 310

Permissible axial stress = 0.1 f_c' = 2.0 N/mm^2
The axial stress due to gravity loads in center column Ground Floor = 440KN
The axial stress due to gravity loads in column
= Load on column (N) / Cross section Area of Column
= 440*1000 / (230*230)
= 8.32 N/mm^2 > 2.0 N/mm^2
Hence, the check is not satisfied for Center Column

1.9.2 Axial Stress in Moment Frames

Axial force in columns of moment frames at base due to overturning forces,
F_o = 2/3 [V_B/n_f] [H/L]
Where,
n_f = Total no. of frames in the direction of loading = 3
VB= Base shear = 368.85 KN
H = height above the base to the roof level = 9 m
L = Total length of the frame =9 m

F_o = 2/3 [368.85/3] [9/9] = 81.97 KN
Axial stress σ = 81.97*1000/230/230 = 1.55 MPa
$\sigma_{all} = 0.25\ f_{ck} = 0.25 * 20 = 5$ MPa
Therefore,
$\sigma < \sigma_{all}$

Hence, the check is satisfied

1.10 Check for Strong Column Weak Beam

1.10.1. Checking Capacity of Center Column at Ground Floor

The Longitudinal Beam of size 230 x 350 is reinforced with 4-16& (804 mm^2, i.e 1.1%) at top and 3-16 (603 mm^2, i.e 0.83 %) at bottom.
Where,
b = 230 mm; d=317 mm
The hogging and sagging moment capacities are evaluated as 76 kN-m and 57 kN-m respectively.
Factored column axial load = 705 kN (1.2DL + 1.2LL+1.2EQL)
$P_u / f_{ck} * b * D$ = (705*1000)/ (20 * 230 * 230) = 0.67
The column is reinforced with 4-16
A_{sc}= 804 mm^2; p_t = 1.52%
p_t/f_{ck} = 1.52 / 20 =0.076
Using SP-16; Chart 45
Moment carrying capacity of column is negligible as the axial load is very high
Σ Mb = 76 + 57 = 133 KN-m >> Σ Mc

Hence, strong column weak beam requirement is not satisfied for Center Column

1.10.2. Checking Capacity of Center Column of Peripheral Frame at Ground Floor

The Longitudinal Beam of size 230 x 350 is reinforced with 4-16& (804 mm^2, i.e 1.1%) at top and 3-16 (603 mm^2, i.e 0.83 %) at bottom.

Where,

b = 230 mm;   d=317 mm

The hogging and sagging moment capacities are evaluated as 76 kN-m and 57 kN-m respectively.

Factored column axial load = 500 kN (1.5 DL+1.5EQL)

$P_u / f_{ck} * b * D$ = (500*1000)/ (20 * 230 * 230) = 0.47

The column is reinforced with 4-16mm&

A_{sc}= 804 mm^2; p_t = 1.52%

p_t/f_{ck} = 1.52 / 20 =0.076

Using SP-16; Chart 45

$M_u / f_{ck} * b * D^2 = 0.075$

M_u = 18.25 KN-m

Σ Mb = 133 KN-m

Σ Mc = 18.25 + 18.25 = 36.5 KN-m << 1.1Σ Mb

Hence, strong column weak beam requirement is not satisfied for center column of peripharal wall

1.11 Check for Out-of-Plane Stability of Brick Masonry Walls

Wall type	Wall thick ness	Recommended Height/ Thickness ratio (0.24<Sx≤0.35)	Actual Height/ Thickness ratio in building	Comments
Wall in first storey,	230 mm	18	2650/230=11.52	Pass

	115 mm	18	2650/115 = 23.04	Fail
All other walls	230 mm	16	2650/230=11.52	Pass
	115mm	16	2650/115 = 23.04	Fail

1.12 Pushover Analysis

1.12.1 Genaral

Seismic Evaluation of existing RC Building is generally performed by Pushover Analysis to verify the adequacy of the structural system. Pushover Analysis is the available method which is a simplified method of Non-Linear Static Process. One of the Non-Linear Static Processes is the capacity spectrum method that uses the interaction of the capacity (Pushover) curve and a reduced response spectrum to estimate maximum displacement. This method provides a graphical representation of the global force-displacement capacity curve of the stucture (i.e. Pushover) and compares it to the response spectra representations of the earthquake demand, is a very useful tool in the evaluation and retrofit design of existing concrete buildings. The procedure help demonstrate how buildings really work by identifying modes of failure and the potential for progressive collapse. In order to provide reliable seismic performance, a building must have a complete lateral force resisting system, capable of limiting earthquak-induced lateral displacements to levels at which the damage sustained by the building's element will be within acceptable levels for the intended performance objective as shown in fig below.

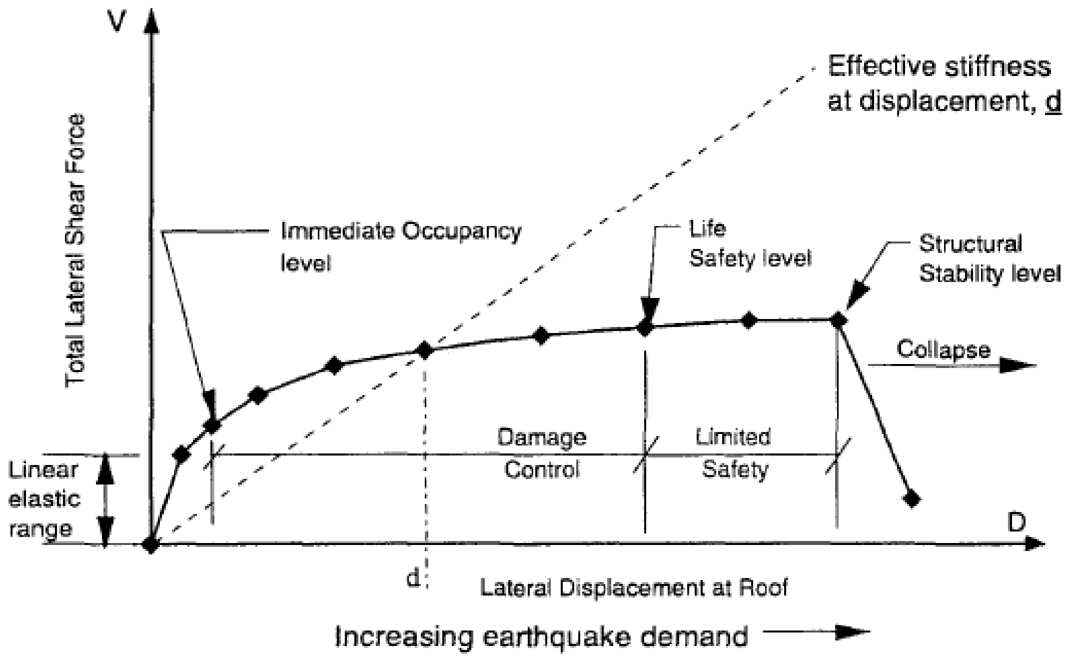


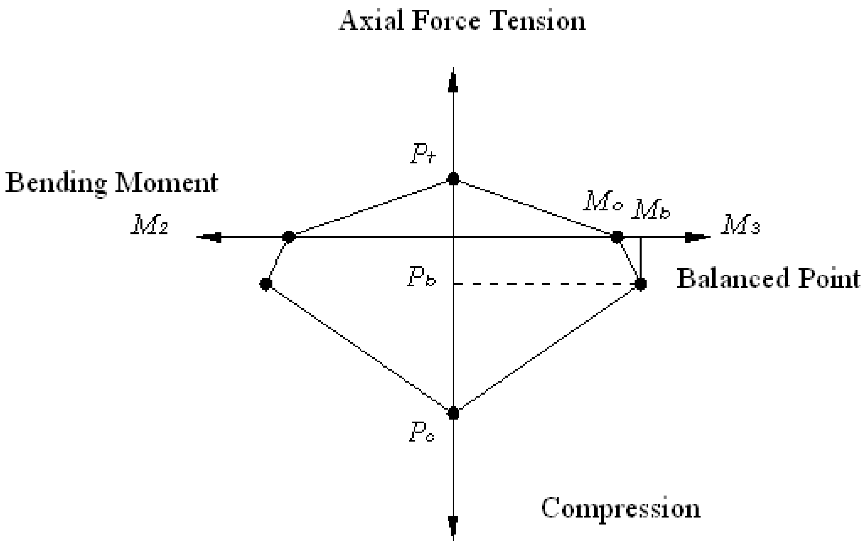
Fig 6.1.1 Typical Capacity Curve

1.12.2 Pushover Analysis of the Building

Pushover Analysis is carried out to determine the structural response of the building. For this, hinge properties for the RC members of the building is calculated using the method given in the book "Reinforced Concrete Structures", R. Park and T. Paulay. Hinge properties are given in Table 6.1.4 below. References of hinge properties are given in Fig 6.1.2, 6.1.3 and 6.1.4.

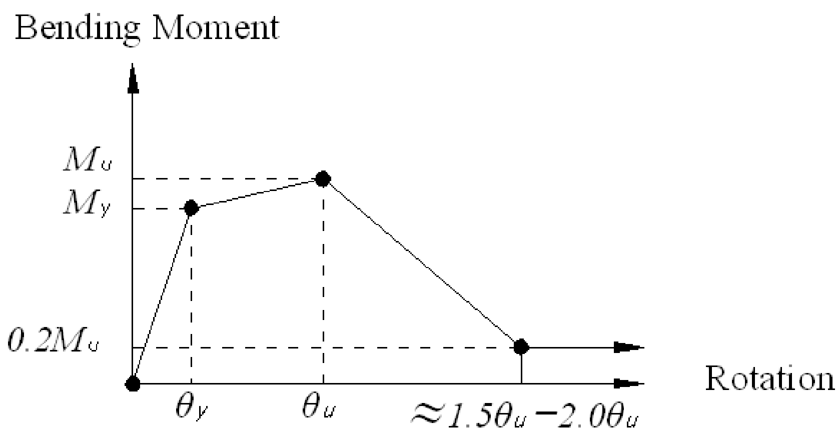
Table 6.1.4 Calculated Plastic Hinge Properties for RC Members of the Frame

Flexural Hinge	Properties	M_y (Negative) (KNm)	M_y (Positive) (KNm)	θ_y (rad)	M_u/M_y	θ_u/θ_y
	M-θ Beams	87	66	0.012	1.05	7
	Properties	P_b (KN)	P_c/P_b	P_t/P_b	M_o (KNm)	M_b/M_o
	P-M Columns	420	3.2	0.79	29	1.86
Shear Hinge						
	Properties	V_u (KN)		Properties	V_u (KN)	
	V-Δ Columns	87		V-Δ Beams	141	



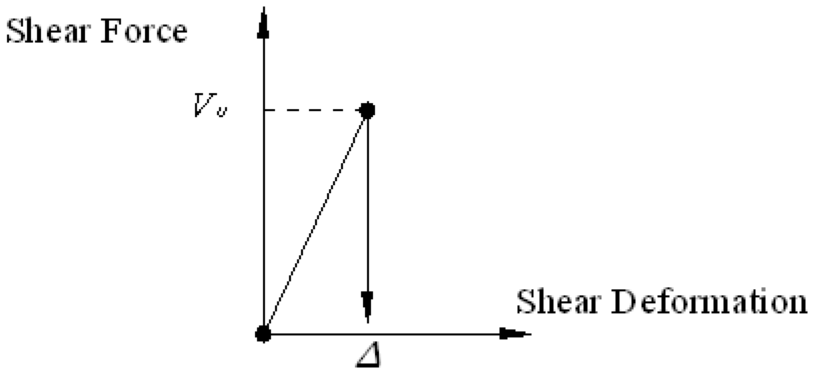
(a) ***P-M***

Fig 6.1.2 Typical Axial load Moment (PMM) Hinge Assigned to Column Members



(b) M-θ

Fig 6.1.3 Typical Moment Rotation (M-θ) Hinge Assigned to Beam Members



(c) V-Δ

Fig 6.1.4 Typical Shear Force Deformation (V-Δ) Hinge Assigned to Beam and Column Members

1.12.3 Results of Pushover Analysis

Capacity Spectrum i.e. Spectral acceleration Vs Spectral displacement curve and Base Shear Vs Spectral displacement curve for the building is plotted as shown in Fig. below. Analysis results show that the capacity of the existing building does not meet the seismic demand of the region. Hence retrofitting is recommended for the building under consideration.

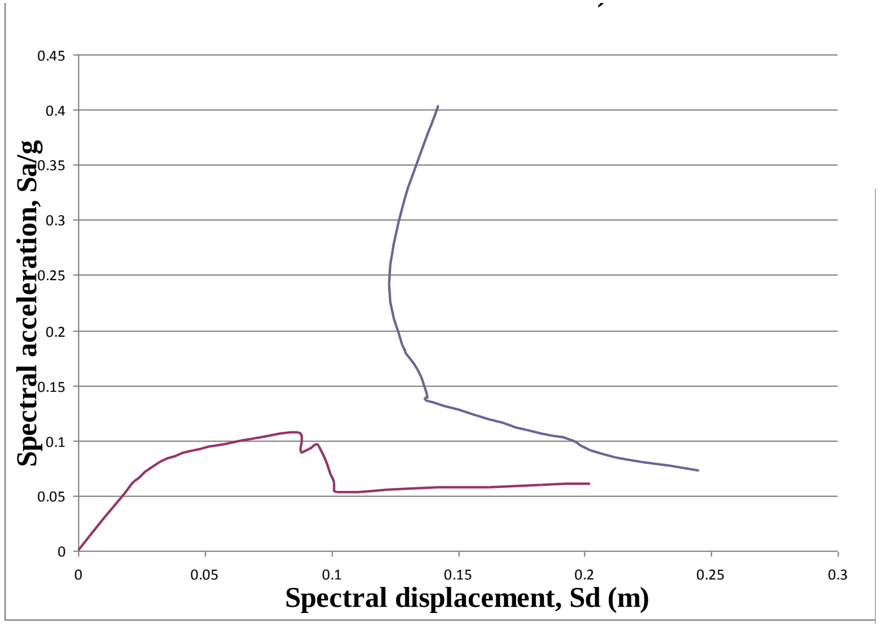


Fig 6.1.5 Capacity Spectrum (Comparison of Demand and Capacity Curve)

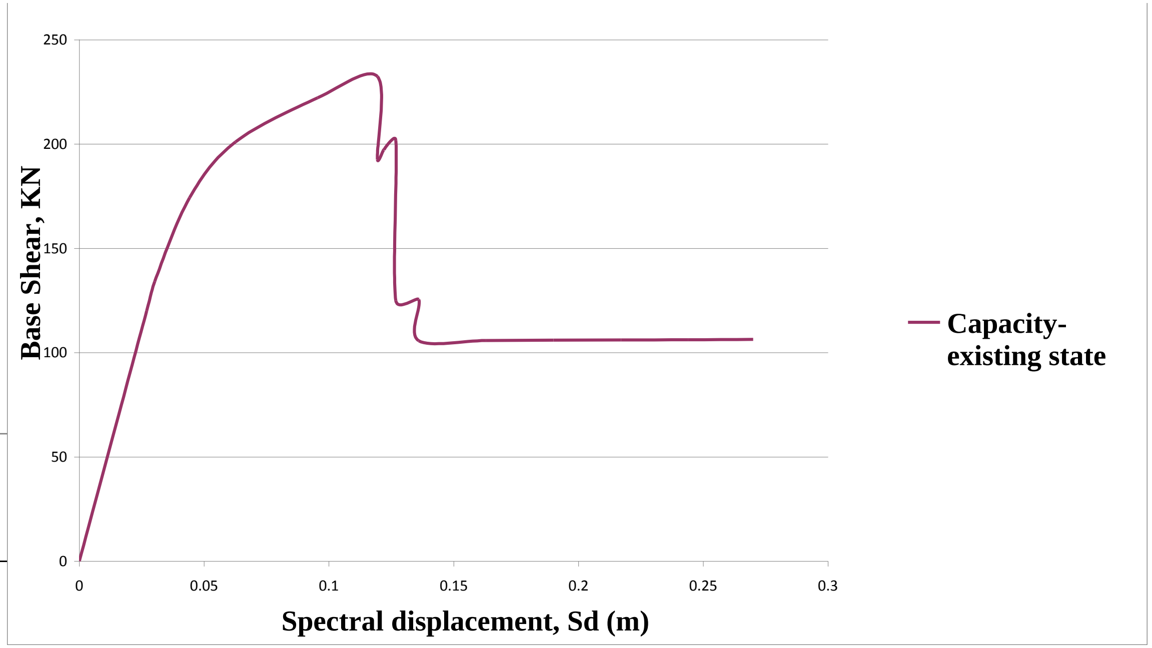


Fig 6.1.6 Capacity Curve of Existing Building

ANNEX VI : Example 2: Seismic Evaluation of Brick Masonry Building

The analysis and design presented here is approximate and is in very simplified version. The goal of this exercise is just to give an orientation for the retrofit design of unreinforced masonry building.

2.1 Building Description

Building Type	:	School Building
No. of Stories	:	Two
Storey Height	:	9’10” (3 m)
Wall	:	Brick in 1:5 Cement Sand mortar
Floor/Roof	:	RCC 100 mm thick Slab
Parapet Wall Height	:	0.9’ (1 m)
Earthquake Zone	:	1 (NBC 105)
Importance Factor	:	1.5 (Educational Building)
Building Dimension	:	29’9”(9.068 m) X 35’10” (10.922 m)

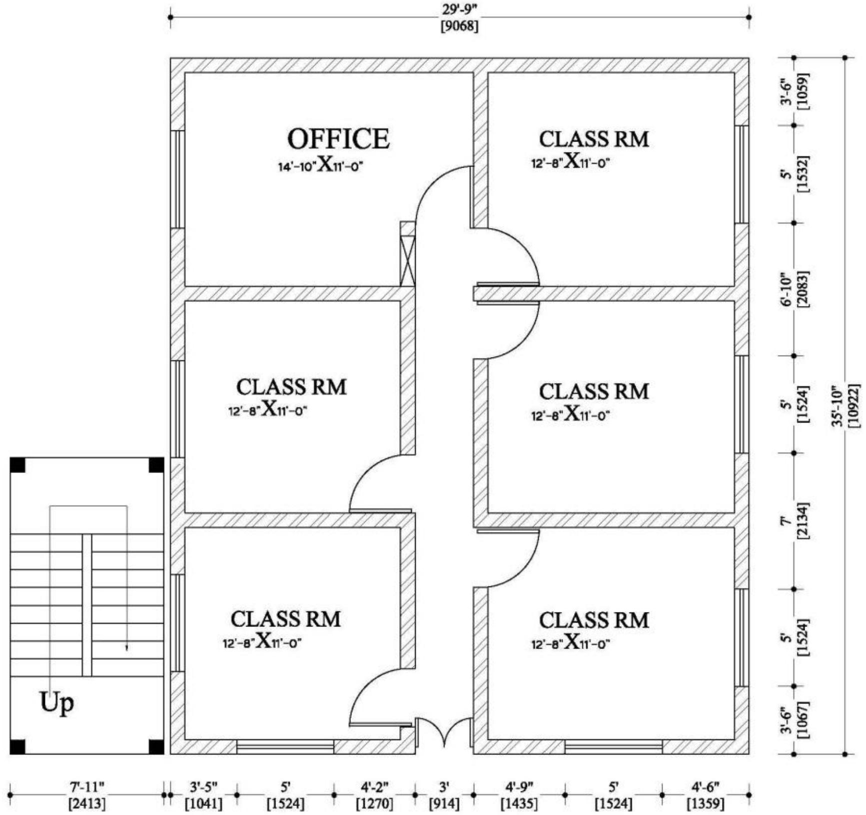
2.2 Design Loads

Dead Loads

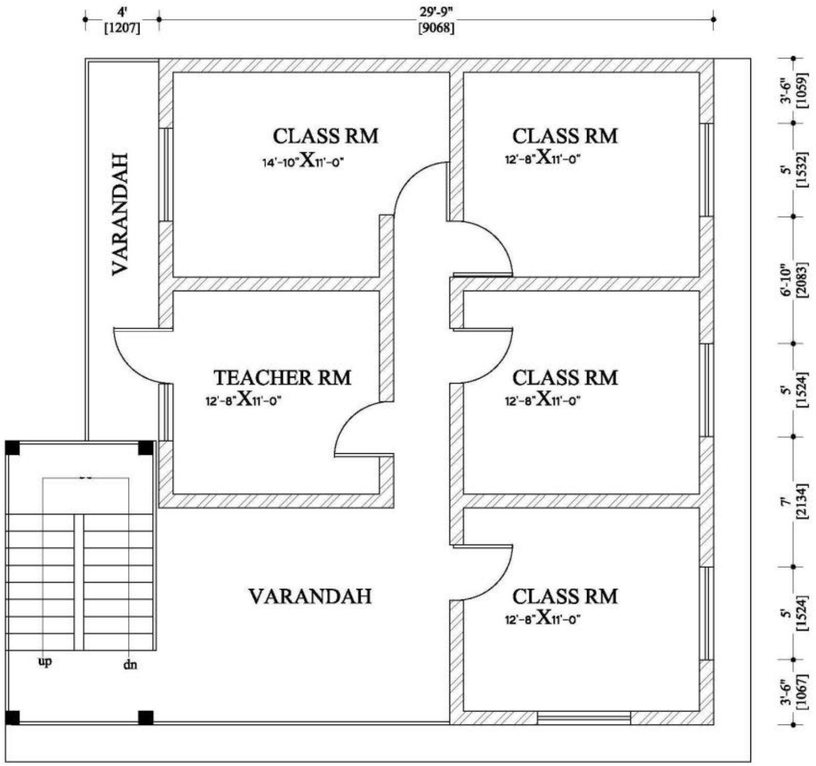
Masonry Wall	:	19 kN/m^3

RCC Slab	:	25 kN/m^3
Live Loads		
Floor Live Load	:	3 kN/m^2 *(IS : 875 (Part 2) – 1987 Table 1)*
Roof Live Load	:	1.5 kN/m^2

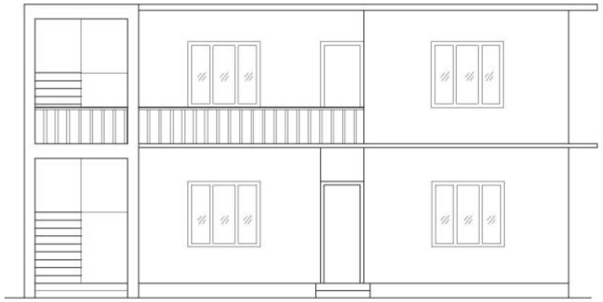
2.3 Building Drawings



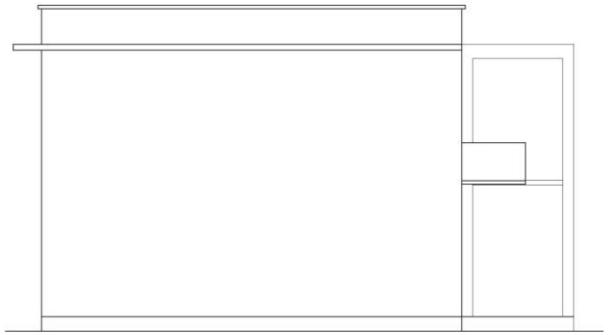
Ground Floor Plan



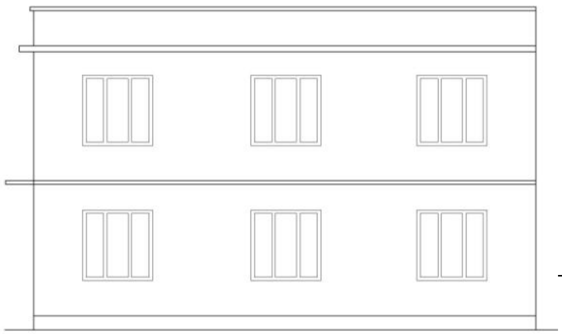
First Floor Plan



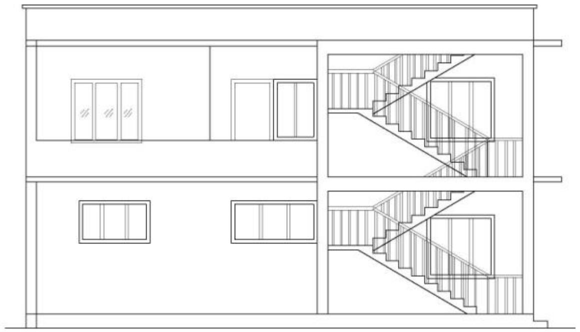
FRONT ELEVATION



BACK ELEVATION



RIGHT SIDE ELEVATION



LEFT SIDE ELEVATION

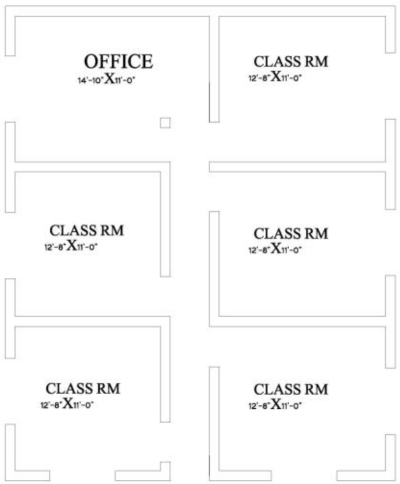
2.4 Load Calculation

Table 6.2.1 Unit Weight of the Elements

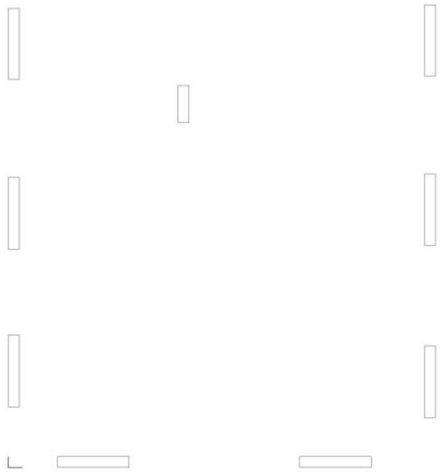
S.No.	Description	Thk. (m)	Density kN/m^3	Finishing Thk. (m)	Density kN/m^3	Intensity kN/m^2
1	Self Wt. of Slab	0.1	25	0.05	20	3.5
2	Wall (9" Thk.)	0.23	19	0.025	20	4.87

Table 6.2.2 Load Calculation For Ground Floor

Floor	Description	Wt.	Height	Area	Centroid		WEIGHT
		(KN/M3).	H (m)	(m2)	X (m)	Y (m)	(KN)
G.F	Walls	19	3.0	12.263	4.523	5.718	699.00
G.F	Walls above window	19	0.6	2.967	4.447	4.359	33.82
G.F	slab	3.5		119.158	3.883	5.323	417.05
G.F	sloped slab						
G.F	Live Load	3		119			357
G.F	Live Load	3		119			357



Ground Floor Walls

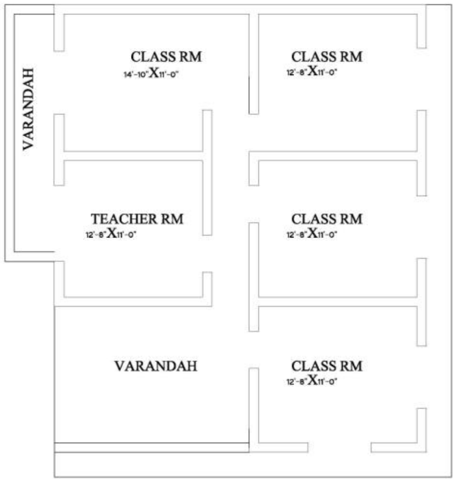


Ground Floor Windows

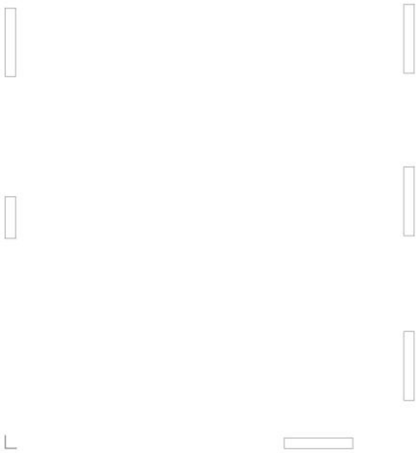
Table 6.2.3 Load Calculation For First Floor

Floor	Description	Wt. (KN/M3).	Height H (m)	Area (m2)	Centroid X (m)	Centroid Y (m)	WEIGHT (KN)
1 F	Walls	19	3.00	10.890	4.870	6.353	620.75
1 F	Walls below window	19	0.90	1.955	6.056	5.109	33.44
1 F	Walls above window	19	0.60	1.955	6.056	5.109	22.29
1 F	Parapet wall	19	1.00	2.983	0.268	4.978	56.68

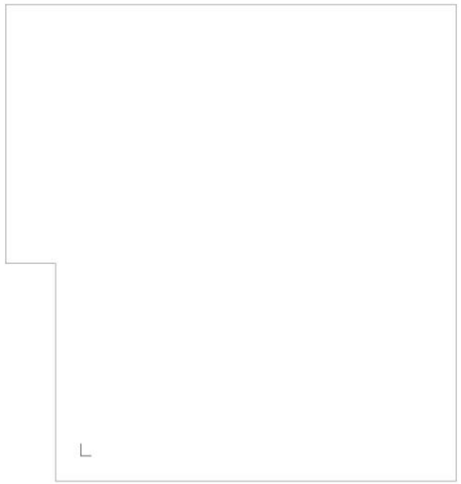
1 F	slab	3.5		119.158	3.883	5.323	417.05
1 F	slope slab						



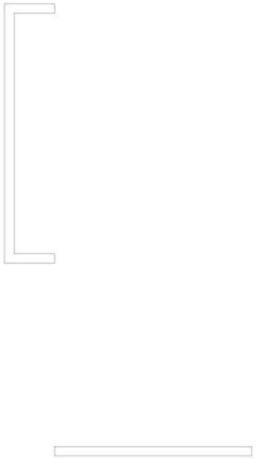
First Floor Walls



First Floor Windows



Ground Floor Slab



First Floor Parapet Walls

2.5 Lumped Mass Calculation

Table 6.2.4Load Calculation for 1st Lump

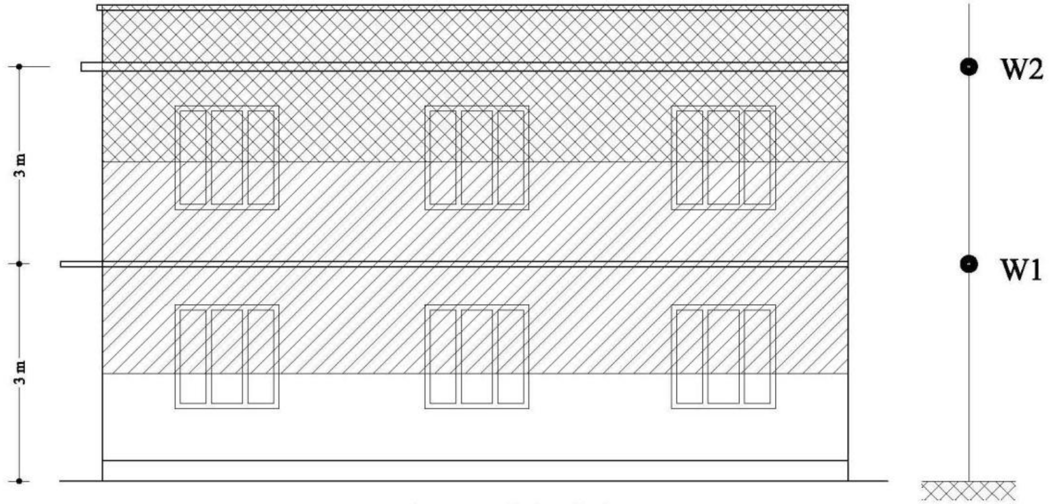
	WEIGHT	X	Y	W*X	W*Y

	(KN)	(m)	(m)	(KN-m)	(KN-m)
G.F Walls	349.50	4.52	5.72	1581	1998
F.F Walls	310.38	4.87	6.35	1512	1972
GF wall above windows	33.82	4.45	4.36	150	147
FF wall below windows	33.44	6.06	5.11	202	171
Parapet Wall (1st Floor)	56.68	0.27	4.98	15	282
G.F slab	417.05	3.88	5.32	1619	2220
Dead load	1200.88			5080	6791
Live Load (25%)	89.37				
Mass Center		4.23	5.65	m	

Table 6.2.5 Load Calculation for 2nd Lump					
	WEIGHT	X	Y	W*X	W*Y
	(KN)	(m)	(m)	(KN-m)	(KN-m)
1F Wall	310.38	4.87	6.35	1512	1972

1F walls above window	22.29	6.06	5.11	135	114
1F slab	417.05	3.88	5.32	1619	2220
Dead load	749.72	4.36	5.74	3266	4306
live load	(Roof Live Load not Considered in Lumped Mass Calculation)				
Mass Center		4.36	5.74	m	

Lumping the Mass in the Storey Levels.



Lumped Mass Calculation

2.6 Calculation of Earthquake Load *(Referring NBC 105)*

Lateral Force Coefficients

Design Horizontal Seismic Coefficient for the Seismic Coefficient Method

The design horizontal seismic force coefficient, C_d shall be taken as :

C_d = CZIK

Where C is the basic seismic coefficient for the fundamental translational period in the direction under consideration.

Basic Seismic Coefficient

The basic seismic coefficient, C, shall be determined for the appropriate site subsoil category using the fundamental structural period in accordance with the code for the direction under consideration.

For the purposes of initial member sizing, the following approximate formulae for fundamental structural period may be used :

(a) For framed structures with no rigid elements limiting the deflection :

T1 = 0.085 H ¾ for steel frames

T1 = 0.06 H ¾ for concrete frames

(b) For other structures :

T1 = 0.09 H/√D

6. 2.7 Quick Calculations for Critical Checks

The following is a sample of quick check calculations based on FEMA 310, IS 1893: 2002 & Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings for the seismic evaluation of building under consideration.

2.7.1. Calculation for Shear Stress check

2.7.1.1 Summary of Lumped Load Calculation

Table 6.2.6 Lumped Weights of the Building at the Storey Levels

Storey	Dead Load (kN)	25% of Live Load (kN)	Total W_i (kN)

2	749.72	0	749.72
1	1200.88	89.37	1290.25
Summation			2039.50

2.7.1..2 Calculation of Seismic Base Shear (Using IS 1893: 2002)

The total design lateral force or design seismic base shear is given by

$V_b = A_h W$

Where,

A_h = design horizontal seismic coefficient = $(ZI/2R)*(S_a/g)$

d = Base dimension of the building at the plinth level in m = 9.07 m and 10.92 m

h = Height of building in m, = 6m

$T = 0.09 * h/ d^{0.5}$ = 0.18 sec for d = 9.07 m

= 0.16 sec for d = 10.92 m

S_a/g = 2.5 (for soft soil, 0.1 [T [0.55)

Z = Seismic zone factor = 0.36

I = Importance factor = 1.5 (For Educational Building)

R = Response reduction factor = 1.5; Un-reinforcement load bearing masonry wall building

Hence, A_h = (0.36 x 1.5 x 2.5)/ (2 x 1.5) = 0. 45 kN and

For the Assumed Building,

Using NBC Code,

Z = 1 (zone 1)

I = 1.5 (Educational Building)

T = (0.09 X H) / (D^0.5) = (0.09 * 6)/(9.068^0.5) = 0.18

C = 0.08 for Subsoil Type III

K = 4 (for structures of minimal ductility)

C_d = CZIK = 0.08 X 1.5 X 1 X 4 = 0.48

Now lets take Base Shear Coefficient **A_h = 0.45**

Total Base Shear **Vb** = 2039.50 X 0.45 = **917.78 kN**

Here, Linear Distribution of Base Shear is adopted as per NBC Code,

i.e. $Q_i = V_b$ X $[W_i h_i / \Sigma W_i h_i]$

Where Q_i = Design lateral force at floor i

W_i = Seismic weight of floor i

h_i = Height of floor i measured from base

Table 6.2.7 Storey Shears at Different Stories of the Building.

Storey	Total W_i (kN)	H_i (m)	W_ih_i (kN m)	Q_i (kN)	Storey Shear (kN)
2	749.72	6	4498.32	493.42	493.42
1	1290.25	3	3870.75	424.58	917.78
Summation	2039.50		8369.07	917.78	

(Using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings, 6.5.1)

Shear stress in Shear walls is given as

$\tau_{wall} = (V_j/A_w)$

For unreinforced masonry load bearing wall building, the average shear stress, τ_{wall} shall be less than 0.1 Mpa

Where

V_j = Storey shear for piers

A_w = Area of shear wall in the direction of the loading

Average Shear stress in X direction walls

Storey	Storey Shear (Vj)	Area of Shear Wall (Aw)	Stresses
1	917.78	7.010	0.13

Average Shear stress in Y direction walls

Storey	Storey Shear (Vj)	Area of Shear Wall (Aw)	Stresses
1	917.78	6.010	0.15

Hence, the check is not satisfied. (As $t_{wall} > 0.1$ Mpa)

2.7.2. Check For Torsion

2.7.2.1 Checking Eccentricity Between Centre of Mass and Centre of Stiffness at Ground Floor

Table 6.2.8 Calculation of Stiffness Center (Walls Along Dir^n. X)

Col No.	Col ID	D (mm)	B (mm)	Area (mm^2)	I_{yy} (mm^4)	Y_i (mm)	$I_y * Y_i$	$K_{ey}=(12EI/H^3)/(1+3D^2/H^2)$	$K_{ey} * Y_i$

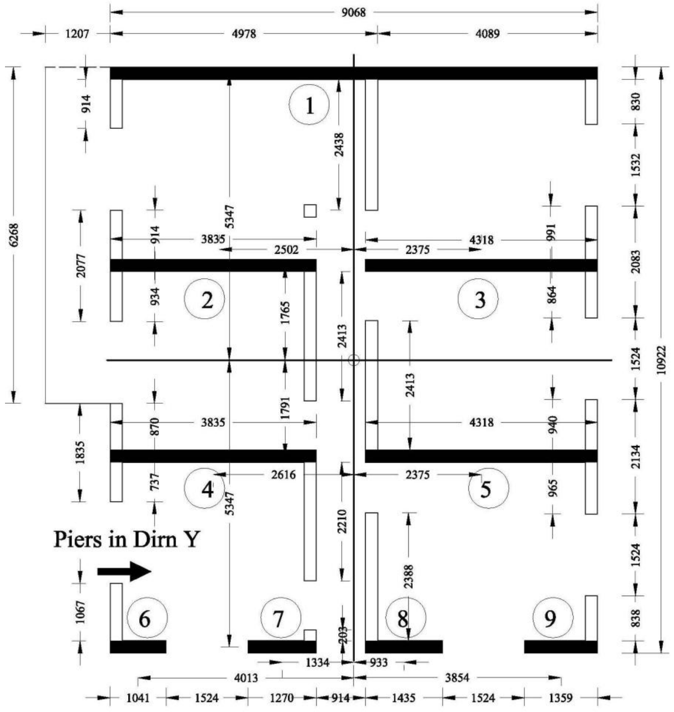
1	Rect. - 1	9068	229	2076572	1.423E+13	10807.5	1.538E+17	4977529.351	5.3795E+10
2	Rect. - 2	3835	229	878215	1.076E+12	7226.5	7.778E+15	1812217.714	1.3096E+10
3	Rect. - 3	4318	229	988822	1.536E+12	7226.5	1.11E+16	2116180.012	1.5293E+10
4	Rect. - 4	3835	229	878215	1.076E+12	3670.5	3.951E+15	1812217.714	6651745119
5	Rect. - 5	4318	229	988822	1.536E+12	3670.5	5.639E+15	2116180.012	7767438733
6	Rect. - 6	1041	229	238389	2.153E+10	114.5	2.465E+12	157168.3707	17995778.5
7	Rect. - 7	1270	229	290830	3.909E+10	114.5	4.476E+12	252639.8734	28927265.5
8	Rect. - 8	1435	229	328615	5.639E+10	114.5	6.457E+12	332304.6978	38048887.9
9	Rect. - 9	1359	229	311211	4.79E+10	114.5	5.484E+12	294619.0231	33733878.1
	Summation =			6979691	1.962E+13		1.823E+17	13871056.77	9.6721E+10

Y

Stiffness Center = 6972.872 mm

$E = 22360\ N/mm^2$

$H = 3000$ mm



Ground Floor Plan

Table 6.2.9 Calculation of Stiffness Center (Walls Along Dir^n. Y)

Col No.	Col ID	B (mm)	D (mm)	Area (mm^2)	I_{xx} (mm^4)	X_i (mm)	$I_x * X_i$	$K_{ex}=(12EI/H^3)/(1+3D^2/H^2)$	$K_{ex} * X_i$
1	Rect. - 1	229	1143	261747	2.85E+10	114.5	3.263E+12	197280.4977	22588617
2	Rect. - 2	228	2667	608076	3.604E+11	4864	1.753E+15	1062570.962	5.168E+09
3	Rect. - 3	229	1067	244343	2.318E+10	8953.5	2.076E+14	166999.9054	1.495E+09
4	Rect. - 4	229	2077	475633	1.71E+11	114.5	1.958E+13	696985.1444	79804799
5	Rect. - 5	229	2642	602376	3.504E+11	3721	1.304E+15	1046706.446	3.895E+09
6	Rect. - 6	229	2083	477007	1.725E+11	8953.5	1.544E+15	700651.8336	6.273E+09
7	Rect. - 7	229	2642	602376	3.504E+11	4864	1.704E+15	1046706.446	5.091E+09
8	Rect. - 8	229	1835	420215	1.179E+11	114.5	1.35E+13	552106.3994	63216183
9	Rect. -	229	2439	556092	2.757E+11	3721	1.026E+15	918414.1572	3.417E+09

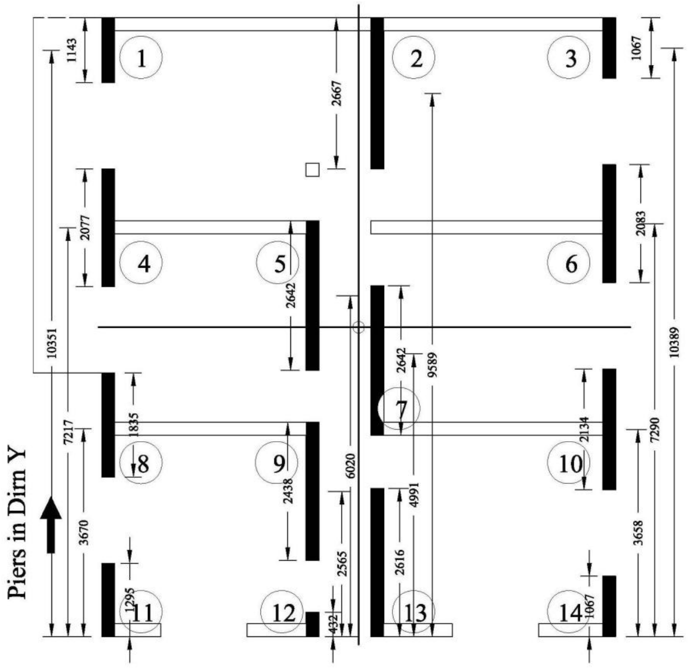
	9								
10	Rect. - 10	229	2133	488457	1.852E+11	8953.5	1.658E+15	731321.3911	6.548E+09
11	Rect. - 11	229	1295	296555	4.144E+10	114.5	4.745E+12	264182.707	30248920
12	Rect. - 12	229	432	98496	1.532E+09	3721	5.7E+12	14331.26592	53326640
13	Rect. - 13	229	2616	596448	3.401E+11	4864	1.654E+15	1030219.465	5.011E+09
14	Rect. - 14	229	1067	244343	2.318E+10	8953.5	2.076E+14	166999.9054	1.495E+09
	Summation =			5972164	2.441E+12		1.111E+16	8595476.527	3.864E+10

X

Stiffness Center = 4495.8 mm

E = 22360 N/mm^2

H = 3000 mm



Ground Floor Plan

Now,

The Location of centre of stiffness at ground floor CS (K_x, K_y) = (4.49 m, 6.97m)

2.7.2.2 Calculation of Mass Center

Referring the calculation done in table 6.2.4,

Lumped mass in Ground Floor (M_1) = 1200.88 kN

Mass Center in that storey X_1 = 4.32 m

Mass Center in that storey Y_1 = 5.65 m

Similarly, referring the calculation done in table 6.2.5,

Lumped mass in First Floor (M_2) = 749.72 kN

Mass Center in that storey X_2 = 4.36 m

Mass Center in that storey Y_2 = 5.74 m

Now, Effective Mass Center can be calculated as,

$$X_{eff} = \Sigma M \times X / \Sigma M = [M_1 \times X_1 + M_2 \times X_2]/(M_1 + M_2)$$
$$= [1200.88 \times 4.32 + 749.72 \times 4.36]/(1200.88 + 749.72)$$
$$= 4.33 \text{ m}$$

Similarly,

$$Y_{eff} = \Sigma M \times Y / \Sigma M = [M_1 \times Y_1 + M_2 \times Y_2]/(M_1 + M_2)$$
$$= [1200.88 \times 5.65 + 749.72 \times 5.74]/(1200.88 + 749.72)$$
$$= 5.68 \text{ m}$$

Location of effective mass center at ground floor (W_x, W_y) = (4.33 m, 5.68 m)

Calculated eccentricity along X direction $e_x = \left| 4.49 - 4.33 \right| = 0.16$ m

Calculated eccentricity along Y direction, $e_y = \left| 6.97 - 5.68 \right| = 1.29$ m

Permissible ecc. along X direction e_x (30% of 9.07 m length along X-dir) = 2.72 m

Permissible ecc. along Y direction, e_y (30% of 10.92 m length along Y-dir) = 3.27 m

Hence, the check is satisfied.

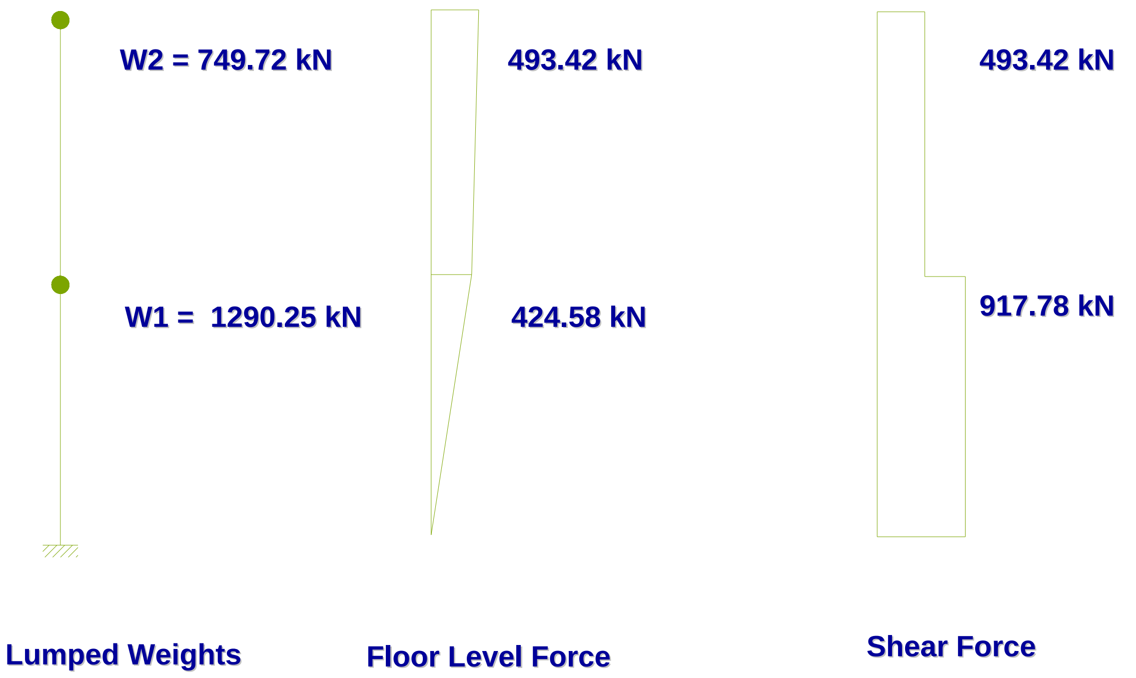
2.8 Stress Calculation of the building

2.8.1 Out of Plane Bending of the Wall,

Here, Linear Distribution of Base Shear is adopted as per NBC Code,

i.e. $Q_i = V_b \text{ X } [W_i \, h_i / \Sigma W_i h_i]$

Referring table 6.2.4,



Lateral coefficient in 2nd storey,

$$C = 493.42/749.72 = 0.63 > 0.45$$

Lateral coefficient in 1st storey,

$$C = 424.58/1290.25 = 0.33 < 0.45$$

Check Stress for, C = 0.66 in the 2nd storey

Special care should be taken for upper storey walls, particularly the top one.

2.8.2 Stress check at Lintel Level

The stress is checked for the horizontal bending of the wall,

Maximum Span of wall in the building = 5 m

Wall below lintel level = 2.2 m

Load carried by the lintel level band,

q = (2.2/2 + 0.8/2) x 4.87 x 0.63 = 4.6 kN/m

Bending Moment,

$M = wl^2/10 = 4.6 \times 5^2/10 = 11.5$ kN m

So a bandage is required to resist the calculated moment in the lintel level of the wall.

2.8.3 Stress check below Lintel Level,

Let's consider the unit width of the wall,

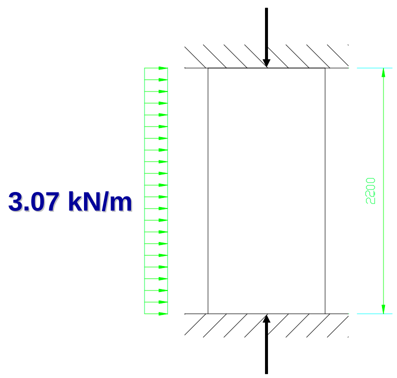
Lateral load = 1*4.87*0.63 = 3.07 kN/m

$M = wl^2/12 = 3.07*2.2^2/12 = 1.24$ kNm/m strip

Bending Stress, $f_b = M/Z$

$= 1.24*10^6/(230^2*1000/6)$

= 0.14 MPa



Vertical load on wall at mid height of wall below Lintel level,

= (2.2/2+0.8)*4.87+4.77*3.5 (wall+slab)

= 25.94 kN

Slab trapezoidal load is considered,

Slab Area = (4.521+(4.521-3.353))*(3.353/2)/2 = 4.77 m2

Stress due to Vertical Load f_a = 4.77 x 1000 / (4521 x 230) = 0.0046 MPa

Combined Vertical Stress on the Wall,

$f = f_a + f_b$ and $f_a - f_b$

$= 0.14 + 0.0046 = 0.1446$ (Compression)

$= 0.14 - 0.0046 = 0.1354$ (Tension)

Permissible Bending Stress for M1=0.07 N/mm2

As the tensile stress exceeds the permissible value, some extra bandage should be provided below the lintel level also.

2.8.4 In plane Analysis of the Piers

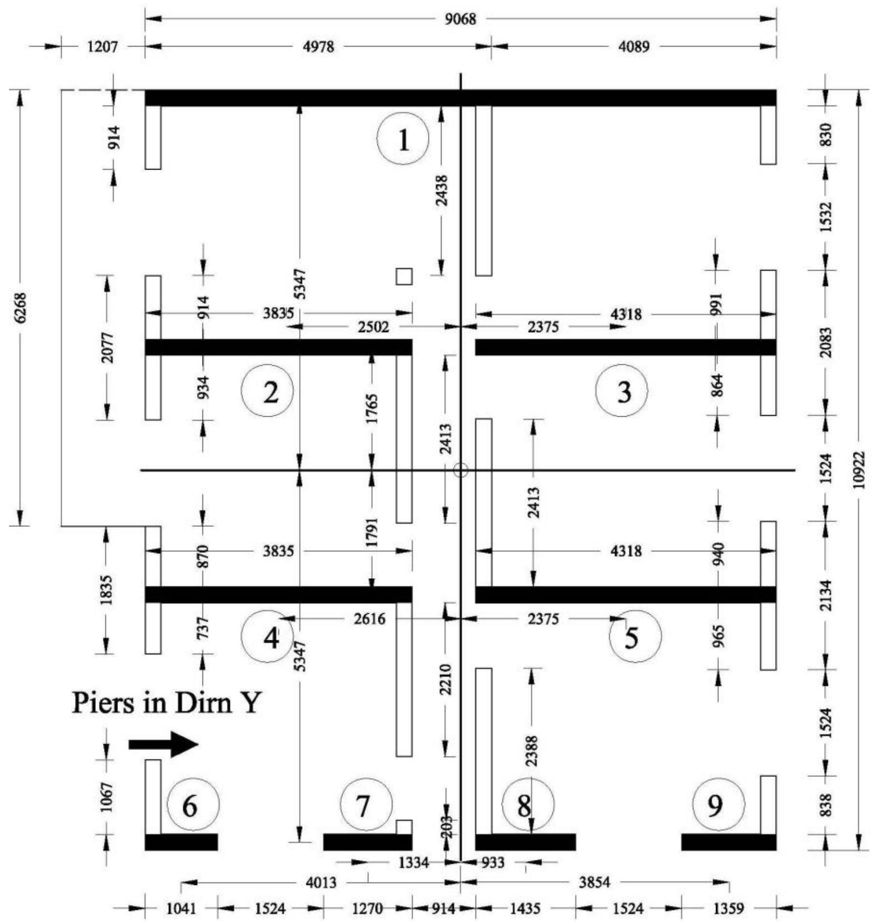
Effect of cross walls is ignored in pier analysis. It can be considered by considering effective areas of piers at L or T sections. The commonly used rules for establishing flange width of L or T section can be used in the case.

The analyses have been done without the consideration of the torsion. However most of the buildings are torsionally active and it is strongly advised to analyze the buildings considering torsion as well.

As the floor is the rigid RCC slab so due to rigid diaphragm action, it is assumed that the loads are distribute proportionate to the stiffness of the pier sections. And it is also assumed that, the effective height of the pier section will be the equivalent height of the door or window whichever is present in that pier section.

Table 6.2.10 Pier Analysis (In Direction X)

Pier No.	Length	width	Height	Area	MI	Stiffness	Prop.	Lateral Load	M	Z	F_b=M/Z
P1	9.068	0.23	3	2.086	14.292	0.277	0.294	269.453	404.18	3.152	0.13
P2	3.835	0.23	3	0.882	1.081	0.098	0.103	94.944	142.42	0.564	0.25
P3	4.318	0.23	3	0.993	1.543	0.115	0.122	111.695	167.54	0.715	0.23
P4	3.835	0.23	3	0.882	1.081	0.098	0.103	94.944	142.42	0.564	0.25
P5	4.318	0.23	3	0.993	1.543	0.115	0.122	111.695	167.54	0.715	0.23
P6	1.041	0.23	1.37	0.239	0.022	0.042	0.045	41.138	28.18	0.042	0.68
P7	1.27	0.23	1.37	0.292	0.039	0.060	0.063	58.191	39.86	0.062	0.64
P8	1.435	0.23	1.37	0.330	0.057	0.073	0.077	70.759	48.47	0.079	0.61
P9	1.359	0.23	1.37	0.313	0.048	0.067	0.071	64.956	44.49	0.071	0.63
Sum						0.944	1.000	917.775			



Ground Floor Plan

Table 6.2.11 Overturning Moment

Piers	Centroid	MI	Propor.	Q1	Q2	M
Pier in Grid 4	4.534	14.29	0.27	116.31	135.08	1159.43
Pier in Grid 3	4.555	14.21	0.27	115.63	134.29	1152.60
Pier in Grid 2	4.555	14.21	0.27	115.63	134.29	1152.60
Pier in Grid 1	4.672	9.47	0.18	77.06	89.50	768.15
Summation		52.17	1	424.62	493.15	

Piers	Pier Section							
	1	2	3	4	5	6	7	8
Pier in Grid 4	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36
Pier in Grid 3	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36
Pier in Grid 2	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36
Pier in Grid 1	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36	-0.36

Table 6.2.12 Vertical Stresses

Piers	Wall Load	Openings	Wall Load	Slab	Roof Slab	Slab Load	Total (kN)	fa (Mpa)
Pier in Grid 4	61.66	0.00	300.30	11.65	11.65	81.52	381.82	0.18
Pier in Grid 3	52.59	3.84	237.44	23.29	23.29	163.04	400.48	0.21
Pier in Grid 2	52.59	3.84	237.44	23.29	23.29	163.04	400.48	0.21
Pier in Grid 1	61.66	8.04	261.15	11.65	11.65	81.52	342.67	0.29

Table 6.2.13 Combination of Stresses at the Bottom of Pier

Grid	End	Bending	Overturn	Vertical	Net Stress	X	Total T
Grid 4	A	-0.13	-0.37	-0.18	-0.68	2860.73	102967.11
	B	0.13	0.37	-0.18	0.31		
Grid 3	A	-0.25	-0.37	-0.21	-0.83	246.77	660.76
and	B	0.25	-0.02	-0.21	0.02		
Grid 2	C	-0.23	0.06	-0.21	-0.39	2161.12	97015.34
	D	0.23	0.37	-0.21	0.39		
Grid 1	A	-0.68	-0.36	-0.29	-1.33	99.47	1603.62

	B	0.68	-0.25	-0.29	0.14		
	C	-0.64	-0.12	-0.29	-1.06	313.10	12479.60
	D	0.64	-0.01	-0.29	0.35		
	E	-0.61	0.07	-0.29	-0.84	531.59	30144.51
	F	0.61	0.17	-0.29	0.49		
	G	-0.63	0.29	-0.29	-0.63	725.02	59667.93
	H	0.63	0.38	-0.29	0.72		

Table 6.2.14 Pier Analysis (In Direction Y)

Pier No.	Length	width	Height	Area	MI	Stiffness	Prop.	Lateral Load	M	Z	F_b=M/Z
P1	1.143	0.23	1.37	0.263	0.029	0.050	0.062	56.783	38.90	0.050	0.78
P2	2.667	0.23	2.134	0.613	0.364	0.095	0.117	107.341	114.53	0.273	0.42
P3	1.059	0.23	1.37	0.244	0.023	0.044	0.054	49.550	33.94	0.043	0.79
P4	2.077	0.23	1.37	0.478	0.172	0.123	0.152	139.635	95.65	0.165	0.58
P5	2.642	0.23	2.134	0.608	0.353	0.093	0.115	105.910	113.01	0.268	0.42
P6	2.083	0.23	1.37	0.479	0.173	0.123	0.153	140.162	96.01	0.166	0.58

P7	2.642	0.23	2.134	0.608	0.353	0.093	0.115	105.910	113.01	0.268	0.42
P8	1.835	0.23	1.37	0.422	0.118	0.104	0.129	118.263	81.01	0.129	0.63
P9	2.438	0.23	2.134	0.561	0.278	0.083	0.103	94.221	100.53	0.228	0.44
P10	2.134	0.23	1.37	0.491	0.186	0.127	0.158	144.637	99.08	0.175	0.57
P11	1.295	0.23	1.37	0.298	0.042	0.062	0.076	70.138	48.04	0.064	0.75
P12	0.432	0.23	2.134	0.099	0.002	0.002	0.002	1.972	2.10	0.007	0.29
P13	2.616	0.23	2.134	0.602	0.343	0.092	0.114	104.422	111.42	0.262	0.42
P14	1.067	0.23	1.37	0.245	0.023	0.044	0.055	50.232	34.41	0.044	0.79
Sum						0.808	1.000	917.775			

Table 6.2.15 Overturning Moment

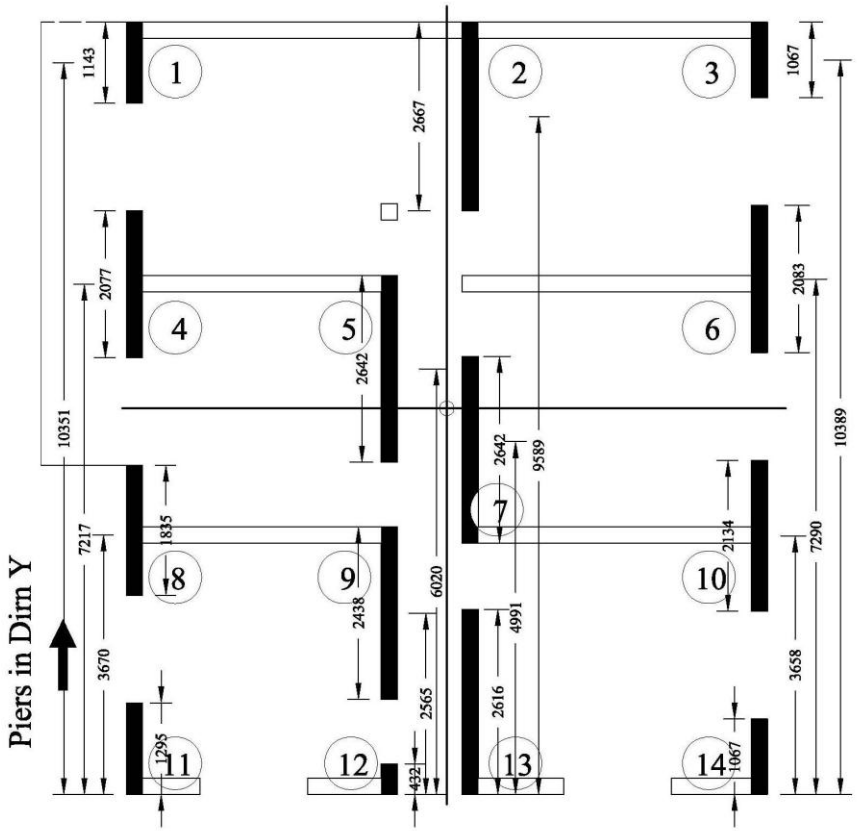
Piers	Centroid	MI	Propor.	Q1	Q2	M
Pier in Grid A	5.416	16.37171	0.25	107.15	124.44	1068.10
Pier in Grid B	3.350	11.04113	0.17	72.26	83.92	720.33
Pier in Grid C	5.323	21.98968	0.34	143.92	167.14	1434.62
Pier in Grid D	5.449	15.47732	0.24	101.29	117.64	1009.75

Summation		64.87984	1	424.62	493.15	

Piers	Pier Section							
	1	2	3	4	5	6	7	8
Pier in Grid A	0.35	0.27	0.17	0.05	-0.05	-0.19	-0.28	-0.36
Pier in Grid B	0.22	0.19	0.13	-0.03	-0.09	-0.26		
Pier in Grid C	0.35	0.18	0.11	-0.06	-0.19	-0.37		
Pier in Grid D	0.36	0.29	0.19	0.05	-0.05	-0.19	-0.29	-0.36

Table 6.2.16 Vertical Stresses

Piers	Wall Load	Openings	Wall Load	Slab	Roof Slab	Slab Load	Total (kN)	fa (Mpa)
Pier in Grid A	74.27	12.60	300.33	17.29	17.29	121.05	421.38	0.31
Pier in Grid B	41.91	11.52	148.02	21.77	21.77	152.36	300.38	0.24
Pier in Grid C	63.35	11.52	252.42	21.77	21.77	152.36	404.77	0.22
Pier in Grid D	74.27	12.60	300.33	17.29	17.29	121.05	421.38	0.29



Ground Floor Plan

Table 6.2.17 Combination of Stresses at the Bottom of Pier

Grid	End	Bending	Overturn	Vertical	Net Stress	x	Total T
Grid A	1	-0.75	-0.36	-0.31	-1.42	125.25	2186.22
	2	0.75	-0.28	-0.31	0.15		
	3	-0.63	-0.19	-0.31	-1.12	248.65	7635.71
	4	0.63	-0.05	-0.31	0.27		
	5	-0.58	0.05	-0.31	-0.84	447.81	22735.30
	6	0.58	0.17	-0.31	0.44		
	7	-0.78	0.27	-0.31	-0.82	647.68	61013.35
	8	0.78	0.35	-0.31	0.82		
Grid B	1	-0.29	-0.26	-0.24	-0.79	(No Tension Zone)	
	2	0.29	-0.09	-0.24	-0.03		
	3	-0.44	-0.03	-0.24	-0.71	416.58	16050.43
	4	0.44	0.13	-0.24	0.34		
	5	-0.42	0.19	-0.24	-0.47	599.32	27841.33
	6	0.42	0.22	-0.24	0.40		
Grid C	1	-0.42	-0.37	-0.22	-1.01	14.36	18.73
	2	0.42	-0.19	-0.22	0.01		
	3	-0.42	-0.06	-0.22	-0.71	392.29	13898.90

	4	0.42	0.11	-0.22	0.31		
	5	-0.42	0.18	-0.22	-0.47	698.55	43800.92
	6	0.42	0.35	-0.22	0.55		
Grid D	A	-0.79	-0.36	-0.29	-1.43	167.99	4124.99
	B	0.79	-0.29	-0.29	0.21		
	C	-0.57	-0.19	-0.29	-1.04	232.52	6102.48
	D	0.57	-0.05	-0.29	0.23		
	E	-0.58	0.05	-0.29	-0.82	476.97	26136.71
	F	0.58	0.19	-0.29	0.48		
	G	-0.79	0.29	-0.29	-0.79	673.80	66468.65
	H	0.79	0.36	-0.29	0.86		