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# SEISMIC PERFROMANCE OF MASONARY BUILDING WITH TIMBER FRAMES

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In partial fulfillment of the requirements for the degree
of
M.Sc. in Structural Engineering

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This is to certify that the work contained in this thesis titled “**Seismic Performance of Masonry Building with Timber Frames**” submitted by **Mr. Kabir Shakya** (061/MSS/r/104), in partial fulfillment of the requirements of the award of “Master of Science in Structural Engineering” is a record of bona-fide work carried out by him under my supervision and guidance in the Institute of Engineering, Pulchowk Campus, Lalitpur, Nepal. This thesis fulfils the requirements relating to nature and standard of the work for the award of M.Sc. in Structural Engineering and no part of this work has been published or submitted for the award of any degree elsewhere.

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# ABSTRACT

Nepali traditional dwelling houses constructed in brick masonry with traditional technology constitute the major type of structures in the urban nuclei of the Kathmandu Valley. Preservation of these structures against possible earthquakes is of prime concern in view of conservation needs. The present study deals with the seismic evaluation of typical traditional building of brick masonry with timber frames.

The present work of the comprehensive study on the subject is divided into two parts. In the first part, various modeling of connections between different elements of brickwork and timber is studied through analysis of number of analytical test models. Based on the results of the sensitivity study of different joint models, the most efficient and appropriate model adopted in the seismic evaluation of the typical traditional dwelling house, the second part of the work. The seismic evaluation of the typical building is carried out by three dimensional analysis of the structure using finite element method. For the analysis, the brickwork is discretized as the eight nodded solid elements, and the timber beams and columns as frame elements. The flexible timber floor is duly considered. The building is assessed for various load cases including gravity loads and earthquake effects in terms of response spectra.

The results of the analysis show that the traditional floors and the spandrels of the existing structure are the weakest parts which need strengthening to make the structure withstand earthquake excitation. The required improvement and strengthening techniques are proposed mainly with the floor converted into rigid floor diaphragm. The analysis of the modified structure shows considerable improvement in the dynamic characteristics of the building and overall structural response.

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# CHAPTER 1

## INTRODUCTION

### 1.1 Background:

Most parts of Nepal including Kathmandu Valley, by the virtue of active faults in the vicinity, are located in highly seismic prone zone. Moreover, it is believed that Kathmandu Valley used to be a lake which, was drained out later. The presence of soft soil strata, lake deposits and fossils found in Kathmandu Valley provide strong evidences to support the conviction. The structures constructed over these deposited soil strata which are considered to be not very much favorable from seismic view point, are subjected to high amplification effect of ground shaking.

The history has recorded large damages during occurrence of earthquakes in past centuries. Earthquakes of 1833 and 1934 are the most recent large earthquakes resulting into disastrous damages in the history of Nepal. The damage records of past earthquakes denote the vulnerability of the buildings in Nepal. It is recorded that more than 80000 houses were heavily damaged, 104,000 houses were partially damaged during 1934 earthquake. These damages include many historical temples, palaces and other dwellings. Most of the structures in Nepal are constructed with locally available materials like brick, timber and stone. The structures made with these as principal materials suffered the maximum damage because of their heavy weight, low tensile strength, low shear strength, low ductility, lack of adequate structural connections, poor quality of construction and deterioration of strength with the passage of time. Beside these, brittleness and poor energy dissipation capacity also aggravate the suitability of brick masonry against seismic excitation.

The historical and traditional structures are constructed with excessive use of brick masonry and timber elements. Even today, use of masonry walls cannot be avoided in developing countries like Nepal. The timber elements are used in these buildings in the form of beams, columns, joists, doors, windows and other decorative elements. Timber doors, windows and other decorative elements not only provide pleasant aesthetic view but also impart structural stability and in controlling localized stresses. However, the structural strength of these buildings against the possible earthquakes is limited. This situation calls for the need of seismic evaluation of the buildings so that appropriate strengthening techniques can be applied.

A lot of researches have been carried out in other parts of the world to investigate the performance of masonry structures under seismic excitation, however, detailed seismic analysis of masonry building with timber frames, prevalent in Nepal are yet to be carried out. The prominent studies relevant to the type of the structures are carried out by Scrivener (1991), El-Borgi (2004), Juhasova (2002), Giordano (2002), Doudoumis (2005), Hite (2002), Beattie (1999).

The appropriate modeling of the buildings like those of brick masonry is important to asses the performance of the structural system, compared to that of modern buildings. This situation demands for detailed modeling and analysis of the masonry structure in order to obtain the structural response closer to the reality. Therefore, to obtain the most representative model of traditional dwelling houses various analytical test models are prepared and analyzed. After identification of the most appropriate model, same is applied to the building under consideration, and its performance under seismic excitation is studied.

With the appropriate modeling of the structure and detailed analysis the weakness in the building is identified and consequently adequate remedial measures are implemented to improve the structural performance. In view of the need of seismic assessment as the first step of disaster mitigation management the present study is undertaken.

## 1.2 Problem and Need of Study:

Most of the buildings in Kathmandu Valley are constructed with brick masonry with various timber elements. The introduction of RCC structure has brought a revolution in the construction sector in the country. At present people do not desire to construct houses using traditional materials and traditional techniques. New buildings as those of modern materials are preferred rather than preserving the old construction technique. It is an irony that most of the people don’t like to build their house in traditional way. Furthermore, modern technologies, RC and steel are imported technologies, they cannot resemble with our tradition and culture. Hence, it is imperative to think of preserving the existing traditional building in order to give life to our tradition and culture.

The core area of Kathmandu Valley still contains traditional building constructed with masonry and timber. Different NGOs, INGOs have given preferences on the preservation of monumental structures however, the dwellings are still remained as an untouched one. Neither significant researches nor detailed evaluations of the loads or bearing capacities of traditional dwellings have been carried out. Usually, most of the renovation or repair works are done in a very simple way or reconstruction with modern materials without considering seismic effects are observed. The monumental structures possess their own importance; however, the residential buildings cannot be ignored because they are directly related with safety of life. Hence, more emphasis should be given to the safety of these buildings.

A lot of research works have been conducted on new construction materials and technologies but the research works regarding retrofitting, rehabilitation, repair, strengthening of traditional buildings are limited. In general, more than 90% of the populations live in the core area of the cities and about 80% of the buildings are constructed of masonry and timber out of these, most of the buildings are constructed in load bearing masonry walls with timber framing.

In view of structural performance, masonry structures have limited resisting capacity against earthquake. So, it is vital to have a study to address the present status of the structural capacity of the traditional buildings whether they are capable of withstanding the possible future earthquake. The responsibility of a structural engineer is not only limited to construction of modern structures, but also to preserve the traditional structures which reflect the state of civilization, tradition and culture. In this regard, the present study becomes an essential step in the conservation of traditional buildings for our future generations.

## 1.3 Objectives of the study:

One of the principal objectives of study is to determine the appropriate analytical model of the traditional Nepalese non-engineered building composed of load bearing brick masonry walls and timber framing. Another important objective is to assess the effect of in-plane rigidity of the existing timber floor, and to propose necessary modifications. The specific objectives of the study are as follows:

### Specific Objectives:

i. Determination of response of the existing structure under possible earthquake.
ii. Determination of appropriate numerical modeling of traditional masonry building strengthened with timber framings.
iii. To find the effect of rigidity provided by the traditional flooring constructed with timber joist and randomly placed wooden planks covered with compressed mud.
iv. To determine the effect of fixity of timber beam and column joints, timber beam and timber joist joints and the connection between the masonry and timber framings.
v. Identification of critical structural elements.
vi. To propose the modification to the structure in order to withstand the probable future earthquake.

## 1.4 Methodology:

This research work deals with an analysis of typical masonry building strengthened with timber framing located in Kathmandu valley under seismic excitation. Response spectra curve for medium soil from IS 1893:2002 (part 1) is used to find the response of the building under consideration. The methodology adopted for the study of the response of the traditional dwelling house is summarized as:

i. Review of various literature concerning load bearing structures, analytical modeling of structures, failure mechanism, mechanical properties of materials, strengthening techniques etc is done.
ii. Preparation of compilation of drawings of traditional house with timber framing and load bearing masonry wall.
iii. Acquisition of material properties required for the study partly from previous relevant codes and partly from early research works.
iv. One of the main problems is connection between timber elements and brick masonry and beam-column joint and the optimum linkage for this problem is determined by testing an analytical test model.
v. Determination of appropriate analytical model of the proposed building through 12 different test models.
vi. Analysis of different test models considering joists, columns as frame elements and walls as solid elements under different load combinations in which seismic load is considered in the form of Response Spectra.
vii. Identification of the most appropriate test model which best represents the response of the structure closer to the reality.
viii. Application of the most appropriate analytical model to the proposed 4 storied traditional dwelling house constructed with masonry wall with timber framings.
ix. Identification of critical sections as well as dynamic properties of the structure.
x. Improvement of the critical sections using different approaches.

### 1.5 Distribution of Chapters:

The whole study of the work is spread over 4 chapters of the thesis. Chapter 2 presents the review of literature related with the research. The review is broadly classified into 3 areas namely failure mechanism, analytical modeling and seismic resistance and strengthening techniques.

Chapter 3 describes about the structural system, modeling and analysis. It describes the characteristics of typical dwelling house of Kathmandu Valley, presents modeling of brick masonry, timber framing and contact surfaces. Different modelings for connections are considered for evaluation. The influence of timber floors in terms of its in-plane rigidity in the lateral load performance of the structure is carried out. A numerical study on a fictitious structure of masonry framed with timber is carried out for free vibration analysis and then analyzed for various load combinations considering effect of earthquake in terms of response spectra and the results are discussed. A typical building representing traditional construction is analyzed for response spectra and different load combinations and results in terms of stresses and displacements are presented and discussed.

Chapter 4 presents the conclusions of the study. Major conclusions of the study are the effectiveness of rigid links, frame with end offsets, between brick elements and timber framing with friction pendulum elements; substantial improvement in performance due to rigid diaphragm action of floor; fundamental time period of the traditional building is found to be around 0.19 sec.

# CHAPTER 2

## LITERATURE REVIEW

### 2.1 General:

The research work on seismic analysis of traditional dwelling structures consist of two principal components, namely, the modeling technique and the seismic analysis of structures. The first component deals with assessment of various models for rational connection of brick masonry and timer elements, timber beams and columns and timber beams and timber joists. The second one is related with the seismic analysis of the brick masonry building with timber framing. In view of this, the literature review is divided into three broad sections, namely, (i) failure mechanism, (ii) analytical modeling and (iii) seismic resistance and strengthening techniques.

### 2.2 Failure Mechanism:

Lenczner, David (1972) proposed that failure in brickwork under axial compression was normally by vertical splitting due to horizontal tension in bricks. The reason for this type of failure is due mainly to the widely different stain characteristics of the bricks and mortar joints. Broadly speaking, the mortar is less rigid than the brick and under load its tendency is to spread laterally to a greater extent than the brick. Shear strength of brickwork is of great importance when designing for lateral loads on brickwork walls. When a horizontal load is applied in the plane of a brickwork wall and parallel to the bed joints, failure may occur either by horizontal shear at the brick/mortar interface of by diagonal tension. The resistance of brickwork to horizontal shears increases as the normal load between the brick and mortar increases. The shear bond is normally independent of mortar strength. Tensile strength of brickwork is governed by the tensile bond at the brick/mortar interface. Among the factors influencing tensile bond strength, are absorption or suction rate of the bricks, the initial water content, type of mortar, type of brick, thickness of mortar joints and workmanship.

IAEE Committee (1980) estimates that overstressing of structural or non-structural elements produces may different types of damage of the various elements of a building. The typical damages and modes of failure in brick masonry are:

a) Failure of Bearing walls due to racking shear which characterized by diagonal cracks caused by diagonal compression or diagonal tension, bending of wall and cracks at the spandrels.
b) Failure of Non-bearing walls due to overturning of walls, local crushing of the corners and sliding of bed joints.
c) Failure of wall connections due to the seismic force of the roof which can cause the formation of tension cracks and separation of supporting walls, torsion and warping during earthquake in an unsymmetrical buildings. This mode of failure causes excessive cracking due to shear in all the walls.
d) Failure of Roofs and Floors due to improper tied roofing materials, weak roof-support connections, heavy weight and instability.
e) Failure of Ground due to inadequate depth of foundation, differential settlement of foundation and sliding of slopes.

## 2.3 Analytical Modeling:

Doudoumis, I. N., Deligiannidou, J., and Kelesi, A. (2005) describe that masonry infilled timber truss-works is a kind of wall that has been used for the load bearing system of many residential buildings all over the world during the last centuries. In this structural system the walls are composed by a wooden skeleton, with vertical, diagonal and horizontal beam-like elements, that is filled with brick-masonry or stone-masonry with (or without) mortar, or just mortar alone. The static behavior of the wooden skeleton is characterized by the

development of axial forces and bending moments, while the contribution of the infilling material in the system's stiffness, strength, and stress distribution is remarkable.
The wooden elements are modeled with beam-column element, the infilling materials with plane-stress elements, while the boundary conditions between the infilling material and the wooden elements are modeled with proper contact bonds. Elastic or inelastic constitutive laws can be used for the materials and the joint connections of the wooden elements. Numerical applications of the proposed model, with comparisons to other models, are presented for several cases of simple walls without openings as well as for the case of a complex wall with openings, representing the entire face of a building storey.

Doudoumis, I. N., Mitsopoulou, E.N. and Nikolaidis, B.N. (1995) point out that, the development of an analytical macromodel (macroelement) which is suitable for the simulation of the elastic behavior of the infill panels in multistory frames under horizontal seismic actions is presented. The macroelement has 4 external and 4 internal nodes, and consists of an isoparametric plane stress element with proper unilateral contact bonds at the corner nodes. The validity of the macroelement has been tested in comparative parametric studies with proper micromodels discretized with sufficiently "fine" meshes of finite elements. Also comparisons of the proposed macroelement with existing experimental results and widely used macromodels are presented.

Mešić, Esad (2003) explained that the traditional analysis of timber structures is based on the assumption that elements connections are either rigid or hinged. This is an actually simplified approach and it excludes the possible cases of the real behavior of the connections. However, in order to obtain the actual stress and deformations distribution in timber structures, the real deformation characteristics must be included in the analysis. If the relation moment rotation is known for each connection in a structure, the exact solution for the stresses and deformations can be obtained by a nonlinear analysis. The real nonlinear analysis of a structure is possible only in the time domain (time history analysis). If we assume in the analysis of timber structures that the nonlinear deformations occur only in connections, then the stresses in a structure are linearly dependent on the deformation for the most part of the degrees of freedom, and nonlinear response occurs in a relatively small number of degrees of freedom. Because the points of a structure in which the nonlinear deformations occur are known in advance, an analysis can be carried out by the so call Fast Nonlinear Analysis developed by Wilson.

Papa, E. (2001) proposed two models based on Damage mechanics and suitable for the analysis of masonry structures subjected to plane stress conditions are presented. In the first, masonry is considered as a homogeneous, orthotropic material. A limited domain, endowed with a suitable hardening/softening law, is defined according to piecewise linear functions; material damage is taken into account as a function of inelastic deformation. In the second model the overall mechanical properties of masonry are determined based on the properties of the components, for which a unilateral damage model is assumed. The numerical results given by these two models appear to be in good agreement with the experimental data related to masonry panels and buildings.

Andreas J. Kappos et al. (2002) evaluates the relative accuracy of different models, mainly intended for use by practicing engineers, for the analysis of unreinforced masonry buildings and to determine whether, and under what conditions, a simple equivalent frame model can be used for design and/or assessment purposes. Several parametric analyses involving finite element models of two-dimensional and three-dimensional structures have been performed, first in the elastic range, using both refined and coarser planar meshes. They were followed by analyses of the same structures using equivalent frames with alternative arrangements of rigid offsets. Subsequently, two dimensional non-linear static (pushover) analyses of both FE and equivalent models were performed to check the validity of the conclusions drawn form the elastic analysis. The result presented herein shed some further light on the feasibility of using simplified and cost-effective analytical models as a tool for practical design and/or assessment of typical masonry structures.

Tena-Colunga, Arturo and Abrams, Daniel P., Members, ASCE (1996) discussed the influence of floor flexibility on the seismic response of building structures through comparison of the computed seismic response for structures with flexible diaphragms and counterpart structures with rigid diaphragms. Case studies of three existing buildings with flexible diaphragms and analogous systems with rigid diaphragms are presented to illustrate these differences in their paper. The paper discusses that structures with flexible

diaphragms can experience higher accelerations and displacements than structures with rigid diaphragms, and their fundamental periods of vibration can be significantly longer.
Tena-Colunga, Arturo and Erri, M. (1992) developed a discrete linear-elastic, multi-degree-of-freedom (MDOF) dynamic model for the dynamic analysis of unreinforced masonry structures with flexible diaphragms. The discrete MDOF dynamic model represents the dynamic response of the structure in a given direction by a reduced number of discrete masses associated to translational degrees of freedom acting in that direction. The discrete model considers the flexibility of the diaphragms and the rotations of the walls, which are included into the global translational degrees of freedom through static condensation.

Timothy L. Phillips et al. (1993) founded that the current analysis and design procedures for light-frame wood buildings don not give consideration to the complex three-dimensional structural response of the buildings. So, they constructed a full scale single-storey wood house and tested under lateral loads at various stages of loading to evaluate the structural response and load-sharing characteristics. Results of the study indicate that the roof diaphragm affected the distribution of lateral load to the shear walls of the building. The roof diaphragm behaved nearly like a rigid diaphragm. Load distribution among the shear walls is a function of wall stiffness and position within the building.

## 2.4 Seismic Resistance and Strengthening Techniques:

Beattie, Graeme J. (1999) illustrated that the lateral load resistance of lightly reinforced masonry walls has always been thought to be quite different from that of timber framed walls. Timber walls were expected to respond to earthquake loading in a ductile fashion while masonry walls were expected to behave in a brittle manner. These differences made it difficult to design structures that incorporate both of these common forms of construction. Since the introduction of the non-specific design standards, it is now recognized that lightly reinforced masonry walls have a limited amount of ductility, which reduces the bracing demand. Additional ductile capacity is available in the connections between masonry walls and the ceiling. This was quantified experimentally. The distribution of lateral force between masonry and timber rammed walls was examined in some typical structures by computer modeling. It was found that the stiffness of a masonry wall system incorporating the bolted and nailed joint, is not significantly different from that of a timber framed wall system when there is bolted and nailed joint at the ceiling to wall junctions. The distribution of lateral load between walls may be obtained using tributary areas when these joints are used. Diaphragm action must also be considered when there is continuous ceiling "chords" perpendicular to the bracing walls. Without the presence of diaphragm action, the loads can be distributed on the basis of tributary area.

Tomaževič, Miha (1999) explained that masonry buildings are typical shear-wall structures. Masonry shear-walls in two orthogonal directions of the buildings, which are linked together with floors, represent the basic resisting elements for both vertical gravity loads and horizontal seismic loads. Consequently, basic principles, hypotheses and mathematical models used for seismic resistant design of reinforced concrete shear walls and shear-wall structures can also be applied to masonry buildings. However, mathematical models developed for seismic resistance verification of R.C. structures should be modified to take into account the specific mechanical characteristics of masonry and constituent materials, as well as specific structural characteristics of various types of masonry construction.

Sucuoğlu, Haluk and Erberik, Altuğ (1997) evaluated the seismic performance of a three-storey unreinforced masonry building which survived the 1992 Erizincan Earthquake without damage. Mechanical properties of the masonry walls have been determined experimentally by using identical brick and mortar used in construction. An accurate material model is developed for masonry and employed in a computer program for the non-linear dynamic analysis of masonry buildings. The analytical results based on measured material properties indicated that masonry buildings which satisfy basic seismic code requirements possess remarkable lateral strength, stiffness and energy dissipation capacity. Accordingly, a simple design approach is rendered suitable for unreinforced masonry under seismic excitations, provided that realistic material properties are employed in the design.

IAEE Committee (1980) emphasized that the seismic strengthening of existing damaged or undamaged buildings can be a definite requirement in some areas. Since its cost may go to as high as 50 to 60% of the cost of rebuilding, the justification of such strengthening must be fully considered. The main items related to seismic strengthening are modification of roofs, substitution of strengthening of floors, planar modifications and strengthening of walls and strengthening of foundations.

Tomaževič, Miha (1999) explained that in seismic areas, the basic criterion for repair and strengthening is based on the correlation of the expected seismic loads with the resistance of structural system, i.e. on seismic resistance verification. If the assessed resistance of the structure is not sufficient to resist the earthquake of expected intensity within acceptable limits of damage, the structure needs seismic strengthening. Seismic resistance analysis will define the causes of potential damage and indicate parts of the structure that need to be strengthened.

P.G. Asteris et al. (2005) described the Earthquake Resistant Design and Rehabilitation of Masonry Historical Structures in 8 Steps. Based on the FEM, a basic methodology for the earthquake resistant design and rehabilitation of damaged masonry historical structures has been developed and presented as a contribution to the solution of this complex problem. The steps involved in the earthquake resistant design and rehabilitation of masonry historical structures includes preparation of detailed architectural and structural drawings, determination of material characteristics, structural simulation, actions of loadings, , analysis, determination of failure criterion, repairing and/or Strengthening decisions and reanalysis if necessary.

# CHAPTER 3

## STRUCTURAL SYSTEM, MODELING AND ANALYSIS

### 3.1 General:

Most of the traditional buildings in Kathmandu Valley are based on a structural system made up of brick masonry wall with timber floors. Burnt clay bricks, mud mortar and timber of different types are mostly used for the construction of these buildings. These buildings not only represent the traditional technology and the contemporary architecture but also reflects the living style of the community the civilization and their different functional use. Earthquake resistance capacity of brick masonry material is limited and recognizing this, various elements of timbers are generally used as supplementary elements in the structural system of the buildings. Another significant feature of the traditional buildings is construction details which are assumed to be effective in undertaking the lateral loads. Beside this, strengthening of the brick masonry wall is consummated by providing timber frames in most of the buildings. Composite action of masonry with timber framing have been studied in many countries however, seismic analysis of timber framed masonry wall buildings, prevalent in Nepal has yet to be carried out. The present chapter describes the detailed seismic analysis incorporating various analytical modeling, and presents the results related with the effectiveness of timber framing in walls, and improved timber flooring with necessary joint details to improve the structure's seismic performance.

The prominent studies on the composite action of masonry, timber and other elements are available in various literatures. Doudoumis et al. (2005), conducted a detailed analytical study on composite behavior of masonry-infilled timber truss-works. They have presented the response of structures for several cases of simple walls without openings as well as for the case of a complex wall with openings, representing the entire face of a building storey. Other significant literatures related to composite behavior of masonry and timber are presented by Doudoumis et al. (1995), Hite et al. (2002) and Beattie (1999).

In order to identify the optimum type of modeling of the connections and the structural system, a one storey building is considered as a test structure. Following the results obtained in the analysis of that test structure a typical traditional dwelling house of Kathmandu Valley is studied for its performance during an earthquake.

### 3.2 Typical traditional dwelling House:

The traditional dwelling house as shown in figure 3.1 is usually rectangular in plan and stretched over 3 to 4 storeys high. The length of the plan is usually about 6 to 8m with facades of various widths but most commonly 4 to 8m. Usually, the house is raised vertically over 3 storeys with a partition wall running up the height, creating front and back rooms. At the upper storey the partition wall is sometimes replaced by a timber frame system so as to create a large continuous space. In most of the buildings, numbers of timber frames are provided at certain interval usually 3m to 4m, parallel to the façade. In this type of building sometimes the timber frames are replaced by brick walls in order to create rooms. The staircase is usually single flight to one side of the plan. The typical inter storey height is between 2.2m to 2.5m. Usually the bathroom is located at the ground floor and the kitchen is on the top floor usually directly under the sloped roof normally used as attic floor. The first floor is traditionally used as bedrooms, while the second floor is used as living room and visitors' reception.

During the construction, the modern construction materials like concrete, steel etc were not available. The most common building material is brick bonded with mud mortar. There are typically two types of bricks: (i) ordinary bricks of size 210mm x 105mm x 50mm and (ii) vitrified fired bricks with same dimensions but with a trapezoidal cross section, so that the mud bed-joint is partially covered externally by the bricks. The second type of brick is normally used in sophisticated construction for the fair facing of external walls. Because of vitrified surface and the overlapping over the joints, this wall construction is substantially impermeable to rain water. The wall is constructed with 3 solid brick leaves of thickness 450mm at the ground floor and 300mm at the top floor.

The floors of the traditional buildings are mainly constructed of timber beams; timber joists and randomly placed wooden planks or bamboo chirpat covered by compressed mud. The layout of timber joists of size 100mm x 70mm spaced closely at intervals of 150mm to 200mm are placed usually normally to the façade. The spandrels above windows are very shallow, just the height of 3 to 5 courses of bricks. The timber frames consist of columns, having cross-section 100mm x 100mm to 150mm x 150mm surmounted by capitals typically known as “Meth” which bear double beams as shown in figure 3.2. The two adjacent timber frames are usually connected only at the level of the beam. In most of the cases the columns at the upper level are not placed directly over the lower one and this causes some eccentricity while transferring load from upper storey to lower. In seismic analysis, this connection is very weak for lateral loads, as all connections are very flexible; the only lateral restraint, when present, is represented by the shear and bending strength of the masonry piers at the edges of the façade.

Window openings vary within a storey depending upon the use of rooms. It also depends on period of construction, normally older buildings have smaller squared windows. There is a practice to connect the adjacent doors and windows with timber member s embedded within the wall, at the sill and lintel or at intermediate level. This provides the structural integrity and to some extent helps in increasing the structural stiffness. Another feature of Nepalese traditional construction is the provision of pegs or shear licks, commonly known as “chukka” in joints at each side of wall, to restrain floor joists from sliding over walls. Continuous wall footings are provided in almost all of the traditional buildings and can be considered as a rigid foundation.

### 3.3 Need of Strengthening:

The traditional masonry buildings did not performed very well in the past earthquakes. Although there were rare cases of complete collapse most of the buildings suffered serious damages. Those damages include collapse of roof, separation of walls, substantially large cracks at the corners of openings and so on. The earthquake resistance capacity of brick is limited and because of flexible type of floor, inadequate connection between timber elements and brick masonry, timber beam and timber column, timber beams and timber joists increase the vulnerability against the lateral load. The structural detailing at the roof of the traditional buildings was comparatively poor and this may be one of the reasons behind excessive damage at the roof of almost all buildings. However, low tensile strength, low shear strength, low ductility, deterioration of strength with passage of time, changes made during use over the years such as making new openings, addition of new parts including dissymmetry in plan and elevation are the major factors which affect the seismic response of the old masonry buildings and in addition being located on highly seismic prone zone call for strengthening of the existing masonry building.

The structural response of the building can be improved by providing adequate joint and connection detailing, increasing the number of shear locks, making the floor rigid against the lateral loads. But before prescribing any remedial measures to the existing structure it is of utmost importance to find out the present capacity of the building. The level of vulnerability is governed by modes of failure; locations of crucial elements in the structure etc and it can be detected through the detailed structural analysis. Therefore, the required strengthening approaches should be preceded by detailed structural analysis.

### 3.4 Modeling of Masonry structures:

Assessment of structure largely depends on the adopted modeling so an appropriate modeling is a must to obtain the reliable response of a structure. Degree of desired accuracy and objective of study governs the type of modeling and accordingly micro-modeling or macro-modeling can be adopted.

### 3.4.1 Modeling of brick masonry wall:

The brick masonry can be modeled either as a heterogeneous material or as a homogeneous material. In micro-modeling brick masonry is considered as a 2-phase heterogeneous material and brick and mortars are treated separately whereas in macro-modeling brick masonry is considered as isotropic and homogeneous material. The desired level of accuracy and objective of study governs the selection of micro-modeling and macro-modeling. On adoption of macro-modeling the analysis of structures gets simplified but in the same time micro-modeling increases complication in structure and hence causes dramatic increment in analysis time and cost. If the performance of a whole system is the priority rather than individual wall or section, with emphasis on material behavior, macro modeling proves to be more economical and effective.

Sometimes micro-modeling of a structure may not be fruitful because of its accuracy, it may vary with very small amount in comparison to macro-modeling. So in such a case, micro-modeling only consumes a lot of time with no significant improvement in structural behavior. Doudoumis et al, (1995), experienced a similar case while they conducted a study for the simulation of the infill panels in multistory frames under horizontal seismic action. They prepared and tested the identical structures with micro-modeling and macro-modeling and found only 2% difference in the response.

Solid elements are used to model brick masonry walls in order to consider three dimensional stresses and deflections. A solid element is an eight nodded element, with six quadrilateral faces as shown if figure 3.3 for modeling three dimensional structures of solids. It is based on an isoparametric formulation that includes nine optional incompatible bending modes. The incompatible bending modes significantly improve the bending behavior of the element if the element geometry is of a rectangular form. Improved behavior is exhibited even with non-rectangular geometry. The stress components and the local coordinate system in a solid model are presented in figure 3.4. For better performance it is best to have an aspect ratio of 1 for solid model but in no case it should be greater than 10.

### 3.4.2 Modeling of Contact Surfaces:

In actual structures, the surfaces of timber beams and columns come into contact with the brick masonry walls but frame elements, normally represented by single centre lines in analysis packages like SAP2000, STAAD, Microfeap etc, are used to model beams and columns in analysis due to which the center lines of the beams and columns come into contact with adjacent masonry walls which is modeled either as shell elements or solid elements. Such type of modeling cannot reflect the actual structural condition. So in order to overcome such type of contact problem any other element which can represent the actual condition, must be introduced between the center of frame element and surfaces of shell or solid elements. To find the appropriate contact element number of analytical test models are obtained and tested.

A fictitious 2D portal frame of size 2.5m x 2m, as shown if figure 3.5, consists of 2 timber columns of size 120mm x 120mm and 1 timber beam of size 120mm x 150mm with infilled brick masonry is taken as a test model. To conduct the test the fictitious material properties are taken as $E_{timber}$ = 12454 N/mm$^2$, $\rho_{timber}$ = 0.865t/cm$^3$, $E_{brick}$ = 1200 N/mm$^2$ and $\rho_{brick}$ = 19 KN/m$^3$. The beams and columns are divided into five equal parts with six nodes in each beam and column. The nodes are provided with different contact elements in different test models. The models used for test are described below:

Model 1 – Bare Frame (Portal frame without infill)
Model 2 – Bare Shell (Only wall without any frames)
Model 3 – Composite (Frames with infill wall but the center lines of beam and columns are connected with wall)
Model 4 – Rigid Links (Frame with infill wall and rigid links are used to connect the center lines of beam and columns to wall)
Model 5 – Offset (Frame with infill wall and frame element with offset is used to connect the center line of beam and columns to wall)
Model 6 – Gap (Frame with infill wall and gap element is introduced between center line of beam/columns and wall)

A 10KN fictitious load is applied horizontally at top and analyzed the structures with different structural configurations for horizontal displacement at different locations using SAP2000 and the result of which is presented in table 3.1. Figure 3.6 presents the displacement at different levels of test models.

It is clearly seen from the table 3.1 and figure 3.6 that, the displacements of model 1 (Bare frame) and model 3 (composite) are maximum and minimum respectively and these displacement are considered as two extreme values. The displacement of model 6 (Gap) are not within these two extreme values but model 4 (Rigid link) and model 5 (Offset) satisfy the limiting condition. But under little modification in section properties of rigid links and change in number of rigid links results significant variation in response unlike variation in model 5. Due to the consistency in the result obtained, model 5 is considered as an appropriate model to incorporate the contact problem between beam/column and wall.

Kappos et al., (2002), conducted a study to evaluate the simplified models for lateral load analysis of unreinforced masonry buildings. In this research wok, masonry spandrels and piers are replaced with frame elements with end offsets and compared its response with the model having spandrels and piers modeled with shell elements and found no significant deviation in the result. The research work concluded that the relatively accurate result can be obtained using frame elements with end offsets instead of using shell elements to represent the masonry walls.

### 3.4.3 Modeling of timber beam-column joint:

In most of the cases, the timber beam column joints are modeled either as fixed joint or hinged joint but in reality the joint behavior doesn’t reflect in reality with either of them. Normally the timber joints are flexible which is responsible for the non-linear deformation of connections within the timber frames. Although it is possible to incorporate the large deformation resulting from the second order effects, the study is limited to small deformation due to which the non-linear behavior can be limited to the connections, can be termed as localized non-linearity.

In order to conduct the analysis, it is necessary to define the load deformation relationship and it can be done through the introduction of a spring element with complicated properties, at the connections of frame elements. But using the software like SAP2000, it is beyond the capacity of the software to reflect the non-linear behavior of connection through the modeling of single spring element with complex force-displacement relationship; however, same behavior can be obtained using combination of several non-linear elements with simple force-displacement relationship. These elements are available in SAP2000, and are termed as Nlink elements. Here, in this thesis work three different Nlink elements viz. gap, hook and plastic1 (Wen Plasticity Property) are used to model the timber beam-column connection.

## Hook Element:

Independent hook properties can be provided for each deformational degree of freedom. The opening and closing of a hook for one deformation does not affect the behavior of the other deformations. The Hook element works only under tension and the nonlinear force-deformation relationship is shown in figure 3.7, and is given by:

$f = k\,(d - open) \qquad \text{if } d - open > 0$ -------------- (3.1)

$= 0 \qquad \text{otherwise}$ -------------- (3.2)

Where,

k = spring constant

Open = initial hook opening, which must be zero or positive.

## Gap Element:

Independent gap properties can be assigned for each deformational degree of freedom. The opening and closing of a gap for one deformation does not affect the behavior of the other deformations. The Gap element works only under Compression and the nonlinear force-deformation relationship is shown in figure 3.8, and is given by:

$f = k\,(d + \text{open})$ if $d + \text{open} > 0$ ------------------- (3.3)

$= 0$ otherwise ------------------- (3.4)

Where,

k = spring constant

Open = initial gap opening, which must be zero or positive.

## Plastic1 Element:

Independent uni-axial plasticity properties can be defined for each deformational degree of freedom. The plasticity model is based on the hysteretic behavior proposed by Wen (1976). The yielding at one degree of freedom does not affect the behavior of the other deformations. The nonlinear force-deformation relationship is shown in figure 3.9 and is given by:

$F = \text{ratio}\ k\ d + (1-\text{ratio})\text{yield}\ z$ -------------------- (3.5)

Where, k is the elastic spring constant, yield is the yield force, ratio is the specified ratio of post-yield stiffness to elastic stiffness (k), and z is an internal hysteretic variable. This variable has a range of $|z| \le 1$, with the yield surface represented by

$|z| = 1$.

The initial value of z is zero, and it evolves according to the differential equation:

$$\dot{z} = \frac{k}{\text{yield}} \begin{cases} \dot{d}(1 - |z|^{\text{exp}}) & \text{if } \dot{d}\,z > 0 \\ \dot{d} & \text{otherwise} \end{cases} \qquad \text{--------------------- (3.6)}$$

where, exp is an exponent greater than or equal to unity. Larger values of this exponent increase the sharpness of yielding as shown in figure 3.9.

The actual behavior of a connection between a beam and a column in the timber frame structures can be modeled through a series of four Nlink elements placed between end joints of a beam and a column and one fictitious joint between the actual joints. The fictitious joint enables the realistic behavior of the actual connection. In this way, a single complex spring element can be replaced by four Nlink elements, which for a given degree of freedom give the same force-displacement relationship as a single spring.

Figure: 3.10 shows the arrangement of four Nlink elements in which first of all, element 4, whose initial stiffness is equal to the stiffness of the connection, introducing mainly from the friction between the connected members, gets engaged in transferring the load through a connection. When the loading overcomes the friction, the element 4 begins to yield and yields until the opening of the gap element closed, that is, until the timber engages connectors. After closing the opening of the gap element, it starts to transfer the load to the element 3. The plastification of the connection starts immediately after yielding of element 3. Under the reversal loading the system behaves in the same manner but hook element comes into account instead of gap element. The behavior of the joint solely depends on the parameters of the Nlink elements and hence large effort is required to find out these parameters.

## Friction-Pendulum Isolator Property:

Many structures have contact surfaces between components of the structures that can only take compression. During the time the surfaces are in contact, it is possible for tangential friction forces to develop between the surfaces. The maximum tangential surface forces, which can be developed at a particular time, are a function of the normal compressive force that exists at that time. If the surfaces are not in contact, the normal and the surface friction forces must be zero. Therefore, surface slip displacements will take place during the period of time when the allowable friction is exceeded or when the surfaces are not in contact. The contact surface element is shown in figure 3.11. During the time of contact, the force-displacement relationships for the friction gap element are:

Normal Force : $f_n = kd_n$ --------- (3.7)

Maximum allowable Slip Force : $fa = \mu| f_n |$ --------- (3.8)

Tangential Surface Forces : $f_s = k\ (d_s - y_s)$ --------- (3.9)

or, $f_s = \text{sign}\ (f_s)\ f_n$ --------- (3.10)

where, the coefficient of friction is designated by $\mu$ and the surface slip deformation in the s direction is $y_s$.

This is a bi axial friction-pendulum isolator that has coupled friction properties for the two shear deformations, post- slip stiffness in the shear directions due the pendulum radii of the slipping surfaces, gap behavior in the axial direction, and linear effective- stiffness properties for the three moment deformations. This element can also be used to model gap and friction behavior between contacting surfaces. The friction forces and pendulum forces are directly proportional to the compressive axial force in the element. The element cannot carry axial tension. The non-linear force-displacement relationship is presented in fig 3.12.

### 3.4.4 Floor System:

The floor of traditional dwelling houses are made of timber joists with dimensions 100mm x 70mm, running from wall to wall spaced closely at intervals of 150mm to 200mm. Above the joists usually planks or bamboo chirpat is placed and covered with compressed mud. The layout of the floor joists is usually normal to the façade. Typically at every two or three joists, pegs are inserted at each side of the wall to restrain floor joists from sliding over walls. This type of floor system possesses very few strength against the lateral loads and hence can be considered as flexible floor diaphragm. Diaphragms are considered flexible when the diaphragm deflection exceeds twice the storey drift, as per National Building Code of India 1983.

Flexible diaphragms generally have multiple dominant modes, and each of diaphragms within the structure tends to respond in each of their principal directions largely in an independent fashion. Structures with flexible diaphragm can experience higher accelerations and displacements than structures with rigid diaphragm and their fundamental periods of vibration can be significantly longer. These complications involved in the flexible system makes the traditional buildings especially located in the most seismic prone zone, Kathmandu valley, more vulnerable against probable earthquake.

The risk of damage in the structure can be significantly reduced by transforming flexible floor into rigid floor system. A rigid floor diaphragm is assumed to distribute the lateral forces to the vertical resisting elements in direct proportion to the relative rigidities of the vertical elements. A rigid diaphragm can be constructed either with a plywood decking or with 1 or 2 in. sheathing laid at 45° to the supporting joists, called diagonal sheathing. For heavy lateral loads, a specially constructed double sheathed diaphragm of plank or plywood can be used. The load- carrying capacity and stiffness of diaphragm depends not only on plywood thickness and nailing but also on whether or not they are blocked. Blocking consists of light weight nailers, framed between the joists or other primary structural supports, for the specific purpose of connection the edges of the plywood sheets, as shown in figure 3.13. The blocking in diaphragm allow nailing of sheets at all edges for better shear transfer and with same nailing spacing, the capacity of diaphragm can be increased by several times greater than those without blocking. Beside this, connection details are other parts which can enhance the performance of floor diaphragm. The detailing of connection is imperative to tie the lateral force resisting system together into an integrated unit. Some of the connection details are presented in figure 3.14 a) to 3.14 e)

A procedure devised by the American Plywood Association (APA) and approved by ICBO, was based essentially on a long and extensive series of tests conducted by the American Plywood Association (APA) Research Center, the Forest Products Laboratory, and the Oregon Forest Research Center is used to design wood diaphragm and in order to design the wood diaphragm in isolation, an example is presented Annex 1.

### 3.5 Ideal Structure for analytical test:

#### 3.5.1 General:

To determine the comparative performance of building with different structural elements, mainly brick wall or wall with timber frames with adoption of different connections, a fictitious simple structure is taken into consideration for analysis. A one storey brick masonry building of size 6m x 6m x 2.3m having four rooms and number of openings of different size as shown in figure 3.15 is taken as a test model. A wall thickness of 230mm with mud mortar is used as structural as well as partition walls and the timber frame with 100mm x 100mm sized columns and 100mm x 150mm sized beams are used. The floor of test model is provided with 100mm x 120mm timber joists spaced closely at intervals of 150mm over which wooden planks are placed randomly covered by compressed mud.

The modulus of elasticity of timber used to conduct this study is taken from the test conducted by Mann, (1978). In his test report the load deflection curve and mechanical properties of Sal wood are presented after number of lab tests and according to which the modulus of Elasticity of Sal wood is found to be 19800 N/mm$^2$. The load-deflection curve of Sal wood is shown in figure 3.16. IAEE Committee II (1980) also presented typical stress-strain curves and load-deflection curves for Sal and Deodar type of timber (Figure 3.17 and figure 3.18). These curves are presented in this thesis work only to get ideas about the nature of stress-strain curve of timber. The material properties used for the analysis is tabulated in table 3.2.

In order to find out structural model which can represent the structural behavior closer to reality, 12 different analytical models are modeled and are described briefly as follows:

Model 1: Bare Frame (Bare Frame with rigid joints)
Model 2: Wall and Frame (Walls with embedded beams and columns with all rigid connections)
Model 3: Wall and frame with rigid links (Walls with embedded beams and columns but the columns are connected to walls with rigid links.)
Model 4: Wall and frame with rigid links and NLlinks (Walls with embedded beams and columns but the columns are connected to walls with rigid links and non-linear links (Gaps and Hooks) is introduced at the boundary between rigid links and walls.)
Model 5: Wall and frame with rigid links and friction elements (Walls with embedded beams and columns but the columns are connected to walls with rigid links and

friction pendulum isolator is introduced at the boundary between rigid links and walls.)

Model 6: Wall and frame with springs and friction elements (Walls with embedded beams and columns but the columns are connected to walls with rigid links and friction pendulum isolator is introduced at the boundary between rigid links and walls. Also spring elements are introduced to model the joint between beams and joists.)

Model 7: Wall and frame with diaphragm (Walls with embedded beams and columns but the columns are connected to walls with rigid links and friction pendulum isolator is introduced at the boundary between rigid links and walls. Also very stiff spring elements are introduced to model the joint between beams and joists and diaphragm constraint is applied at floor level.)

Model 8: Wall and frame with flexible springs and friction elements (Walls with embedded beams and columns but the columns are connected to walls with rigid links and friction pendulum isolator is introduced at the boundary between rigid links and walls. Also flexible spring elements are introduced to model the joint between beams and joists.)

Model 9: Wall and frame with springs but without columns (Walls with embedded beams only but with no columns. But flexible spring elements are introduced to model the joint between beams and joists and diaphragm constraint is applied at floor level.)

Model 10: Wall and frame with stiff beam column joint (Walls with embedded beams and columns but the columns are connected to walls with rigid links and friction pendulum isolator is introduced at the boundary between rigid links and walls. Also beam column joints are modeled with 4 stiff Nlink elements and flexible spring elements are introduced to model the joint between beams and joists.)

Model 11: Wall and frame with Flexible Beam column joint (Walls with embedded beams and columns but the columns are connected to walls with rigid links and friction pendulum isolator is introduced at the boundary between rigid links and walls. Also beam column joints are modeled with 4 flexible Nlinks elements and flexible spring elements are introduced to model the joint between beams and joists.)

Model 12: Wall and frame with Beam column joint different plastic properties (Walls with embedded beams and columns but the columns are connected to walls with rigid links and friction pendulum isolator is introduced at the boundary between rigid links and walls. Also beam column joints are modeled with 4 flexible Nlinks elements but with varying stiffness properties of plastic1 and plastic2 elements and flexible spring elements are introduced to model the joint between beams and joists.)

In these models brick masonry is discretized with solid elements, timber beams and columns with frame elements, and different connections, contacts etc are modeled with NLlink elements. Dynamic properties, in terms of time periods of each and every models are found separately using SAP2000 and compared these values with those prescribed by IS 1893:2002 (part 1), cl. 7.6.2. The comparison of time period is given in table 3.3. To find the response of the structures under seismic excitation, a response spectra analysis is performed for every models using Response Spectra curve as shown in figure 3.19, given in IS 1893:2002 (part 1) for medium soil. The zoning factor of Z=0.36 (for the severest zone), and the response reduction factor, R = 1.5 are used in the analysis. To have fair idea about the response of test models, they are subjected to a combination of gravity and seismic load which are as follows:

i. Dead Load + Imposed Load
ii. Dead Load + Imposed Load + Response Spectra in x-direction
iii. Dead Load + Imposed Load + Response Spectra in y-direction
iv. Dead Load + Imposed Load - Response Spectra in x-direction
v. Dead Load + Imposed Load - Response Spectra in y-direction

### 3.5.2 Results and discussion on test models:

Since, model 1 (Bare frame) and model 2 (Wall and Frame) are too ideal cases from the real structure; these cannot represent the true behavior of the structure. These are obtained only to compare the test results considering their behavior as extreme cases for having maximum and minimum displacements. It is seen from the table 3.3 that the displacement in y-direction of model 3 (Wall and Frame with Rigid Links) is significantly larger than the extreme case displacement of model 2. The displacements of model 4 (Wall and frame with Rigid links and NLlinks) lie between the two extreme cases, however, this case does not consider the friction between timber frames and masonry walls, which is believed to take place. To overcome the problem related to friction, friction-pendulum isolators are introduced at the joints between rigid links and masonry walls as described in model 5 (Wall and frame with Rigid links and friction elements). It is to be noted that in this model the beams and joists are rigidly connected at their center lines contrary to the reality the timber joists are located over the timber beams. This problem is overcome in model 6 (Wall and frame with Springs and friction elements) with adoption of spring elements to represent the location of joists over the beam. To access the effectiveness of rigid diaphragm over the flexible one, model 6 is improved to model 7 (Wall and frame with Diaphragm) by providing a rigid floor diaphragm action.

In model 6 and model 7 very stiff springs ($k = 10^7$ KN/m) are used to model the connection between beams and joists. Again to check the reliability of spring stiffness; the spring stiffness are reduced to 1000 KN/m and model 8 (Wall and frame with flexible spring and friction elements) is obtained and reanalyzed with the new value of spring stiffness. But even under such a large variation in spring stiffness, only marginal variation in the response of the structure is observed, which can be clearly seen in table 3.3. Therefore, it can be pointed out that the alteration spring stiffness rarely influences the structural response.

It is indispensable to check that whether embedment of timber columns improved the performance of the structure or not. Hence, model 9 (Wall with springs but without column) which consists of the masonry walls without timber columns but with beams and joists connected through springs is developed to make a distinguished comparison. The displacements of the structures as shown in table 3.3, indicates that the performance of the masonry structure strengthened with timber frames is better than that of without timber columns as predicted.

In most of the cases, the joint between timber beam and column is modeled as either rigid or hinged joint. But both of these are ideal cases and the reality lies in between these two extreme conditions. Therefore to obtain more realistic structural response, model 10 (Wall and Frame with stiff beam column joint) is created in which the joints between beams and columns are remodeled using 4 NLlink elements (1 Gap, 1 Hook and 2 Plastic elements) which have been already described in section 3.4.3.

Again to make sure about the stiffness values of link elements used at the beam column joints, model 11 (Wall and frame with flexible beam column joint) is obtained after alteration in stiffness values of NLlink elements. When the response of model 10 and model 11 are compared the significant variation is found and hence after careful inspection and studying the experimental procedures of timber testing and their results, appropriate stiffness values are worked out to the NLlink elements. Again to make distinguished comparison, model 11 is improved to model 12 (Wall and frame with beam and column joint having different plastic properties) considering stiffness values as 10000 KN/m, 10000KN/m, 6000 KN/m and 16000 KN/ for gap, hook, plastic1 and plastic2 elements respectively. After analysis the results are worked out and compared its result with the two extreme values as well as result of other models. Being consideration of almost all of the parameters involved in real structure and also due to its response which lies in between the extreme cases, model 12 is believed to be the most representative analytical model however, due to accumulation of lots of assumptions in providing link elements' properties, model 12 is not considered as an appropriate analytical model to conduct the study.

The displacements of model 4, model 6, model 8, model 9, model 10, model11, model12 are within the two extreme case displacements except for those of model 3, model 5 and model 7. The reduction of displacement in model 3 and model 5 compared to that of Model 2 indicates the inadequacy of the analytical model whereas for model 7, it indicates the benefit of rigid floor diaphragm over the flexible one.

Due to lack of experimental results, the stiffness values of different NLlink elements are assumed. Being lots of assumptions within a structural system and also due to involvement of number of NLlink elements added the complication in model 12. Meanwhile, the omission of NLlinks at the beam-column joint makes the model 8 comparatively simplified analytical model without losing other parameter except flexibility at beam-column joint, and also due to marginal variation in the responses which can be clearly seen in table 3.3, of the model 8 and the model 12, model 8 is considered as a most representative and practical analytical model for modeling a masonry building strengthened with timber frames.

The decision made for the selection of model is based on the deflection criterion only meanwhile the maximum stresses, shown in table 3.4, induced at various locations are traced out for getting ideas about the fluctuation of stresses within the system in accordance to the changes in the modeling of the structure. It is comparatively difficult to predict the stress pattern in the various parts of any structure under seismic excitation for different structural configuration but it is rather easier to predict the qualitative nature of deflection unlike stresses and it is the reason why the deflection is given a priority over stress for the selection of an appropriate analytical model.

## 3.6 Typical traditional structure and its analysis

### 3.6.1 Description of Structure:

A traditional dwelling house of rectangular plan shape and over four storeys high constructed with mud mortar brick masonry and timber frames as shown in the figures in Annex 2 is selected for the study. The depth and width of plan are 6.553m and 5.486m respectively and the building rises upto the height of 9.449m. The height of the attic floor is 2.743m and remaining floors are of 2.134m height each. A 457mm thick wall is provided at the periphery of the building. Two types of partition walls having 228mm and 101mm wall thickness running upto the height create front and back rooms with lobby and staircase at the left side of the building.

There are all together 5 timber frames, 4 of them are provided at the either facades and 1 at the middle as shown in figures in Annex 2. The timber frame consists of 101mm x 101mm timber columns and 152mm x 101mm timber beams. The timber joists of size 101mm x 127mm are provided over the timber beams perpendicular to the façade at intervals of 152. Timber planks covered with compressed mud is provided above the joists to construct a traditional floor.

Squared shaped 914mm x 914mm window are provided at the ground floor and attic floor. Rectangular windows of size 0.914m x 1.829m are provided at 1st and 2nd floors and rectangle shaped doors of different sizes are provided within the building. The sizes and locations of doors and windows are clearly mentioned in Annex 2.

### 3.6.2 Modeling and Analysis:

The masonry walls are modeled with solid elements, timber frames are modeled with frame elements and different connections, contacts, friction etc are modeled with the help of NLlinks elements. The wall section is divided into 4 layers in order to provide proper connection between masonry wall and timber frames. Similarly, the masonry walls are divided into number of elements in vertical plane in order to maintain the aspect ratio of solid elements below 10. The floor of the building is considered as flexible floor. An analytical model of the dwelling house under consideration is shown in figure 3.20. The material properties assigned to the analytical model are presented in table 3.2.

Using SAP2000 version 8, free vibration analysis is performed for 6 modes of vibrations and in order to consider the effect of earthquake on the structure, a response spectra analysis is done using response spectra curve, shown in figure 3.19, for medium soil and 5% damping from IS 1893:2002 (part 1). The result of free vibration analysis, in terms of time period and natural frequencies are presented in table 3.5.

Although the result of the test models described in section 3.5.2 recommends the application of analytical model confirming to that of model 8 (Wall and frame with flexible spring and friction elements) to the real structures, the building under consideration is modeled in two different structural configurations; one with rigid beam column joint according to model 8 and another with beam column joint connected with 4 NLlinks elements according to model 12 (wall and frame with beam column joint different plastic properties). These two models are created and analyzed separately in order to be more confident in adopting the most appropriate model and to have fair idea of comparative results between these two models. The time periods and frequencies in different modes, maximum deflections and maximum stresses induced in the structures are presented in table 3.5, table 3.6 and table 3.7 respectively. It is to be noted that on comparing time periods, deflections and stresses induced in the structure between these two models, they are different by a nominal margin. Therefore, due to this result and the reasons mentioned in section 3.5.2, the building modeled according to model 8 is recommended for further analysis.

### 3.6.3 Computation of Fundamental Natural Time period:

IS 1893:2002 has given a following relation to calculate the appropriate fundamental natural time period of vibration

$T = 0.09H/\sqrt{d}$ ---------------------------------------- (3.11)

where,

H = Height of building in m

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force.

The fundamental natural period of vibration of a structure can also be obtained from the free vibration analysis in SAP2000. The time period calculated from equation 3.11 and time period obtained after free vibration analysis is tabulated in table 3.10.

The time period obtained from these two methods are quite different and hence considering the time period obtained from the free vibration analysis being closer to reality a relation between height of building under consideration and fundamental natural time period is formulated and is given by

$T = 0.0299\ H^{0.8202}$ -------------------------------- (3.12)

The time periods calculated using equation 3.12 is tabulated in table 3.10 and are similar to those obtained from free vibration analysis. While formulating the equation 3.12, the building under consideration is analyzed in 4 different stages varying the number of storey from 1 storey to 4 storey. The first mode of vibration of the building in free vibration analysis and from equation 3.11 is in the same direction.

### 3.6.4 Results and Discussion:

On analyzing the structure modeled as per model 8, for gravity loads, stresses and displacements are considerably small, as predicted and hence the building is safe under gravity loads. Even under the action of gravity load only, comparatively large tensile bending stresses of localized nature are observed at the spandrels above and below the doors and windows but are well below the limiting values.

The structure is found to be vulnerable in out of plane as well as in in-plane tensile bending while the earthquake load is considered along with gravity load. The compressive stresses induced in the structure and the displacements are within the limiting value. Figure 3.21 and figure 3.22 show the deflected shape of the building under earthquake excitation in x-direction and y-direction respectively. The figure 3.23 shows the displacement of structure at different floor levels under different load combinations. In some locations especially partition walls and spandrels, the shear stresses induced are significantly high, however, below the failure stress level. These stresses can be considered as of localized nature. The maximum shear is observed in the partition wall at the ground floor. Figure 3.24(c) shows the shear stresses induced in the structure.

Meanwhile almost all part of the spandrels and some localized portions of partition walls as shown in figure 3.24(a) are beyond the ultimate limit of tensile bending stress and hence it can be said that the failure in the structure occurs due to tensile bending. The tensile stresses in walls are found to be larger at the inner portions of the walls rather than the outer faces, this is because of the presence of the timber elements of doors and windows at the outer faces. It is clearly seen in the figure 3.25 that the outer faces of walls are less stressed whereas the inner faces are highly stressed and fail under tensile bending. Moreover, the discontinuities within the structure where the cross-section and accordingly stiffness of the walls are drastically changed seem to be overstressed. In the case of 101mm thick partition wall; the sections are overstressed due to high slenderness ratio.

**Improvement in Structure:**

The floor of the building is substantially flexible and the flexible diaphragms generally possess multiple dominant modes, each of diaphragms within the structure tends to response in each of their principal directions largely in an independent fashion. The presence of flexible floor diaphragm in the structure increases the complication and it can be dramatically reduced by introducing rigid floor diaphragm in the structure and it can be obtained following the procedure described in the section 3.4.4. After making the floor rigid, the structure is remodeled with the provision of the rigid floor diaphragm and then reanalyzed.

A lot of enhancement in the structure is observed after providing rigid floor diaphragm in the structure. The most important point to be noted is the mode shapes which are compared in figure 3.26. It is clearly seen from these figures that the haphazard types of mode shapes obtained in the building with flexible floor diaphragm is improved after providing rigid floor diaphragm and followed a well defined pattern of mode shape which can be predicted beforehand. The compressive stresses and shear stresses induced in the structure are within the permissible limit and the displacements as well as time periods of structure are reduced compared to the previous one. The provision of diaphragm controlled the random distribution of localized stresses and hence the partition walls which were highly stressed at some locations are now seems to be almost uniformly stressed and also the stress levels on the spandrels at the top two storey fell below to the limiting values whereas the spandrels at bottom two storey i.e. ground floor and first floor are overstressed and still fails under tensile bending. This situation again calls for modification in the structure.

### Further Improvement in the structure:

Since, the tensile bending stresses are crucial at the spandrels; the window opening sizes are reduced by adding brick masonry at the bottom of the windows. This improvement made the increase in height of spandrels and hence to some extent it reduced the discontinuities increasing structural stiffness. The improved model with added brick masonry wall and rigid floor diaphragm is reanalyzed and time periods, stresses and deflections of the modified building are studied.

This improvement in the structure brought the deflections, compressive stresses, shear and tensile bending stresses at almost all parts of the building under control but at very few locations of the topmost storey exceeded the permissible tensile bending stress. These stresses are of localized type and their failure cannot be considered as a structure failure. The partial damage at the roof level can be ignored and as a whole it can be said that the building is safe against the probable considered future earthquake. The tensile bending stress contour, as a sample, is shown in figure 3.27 and the deformed shapes in figure 3.28 and figure 3.29. The deflections at the floor levels of the improved model are shown in figure 3.30. The time periods and frequencies of existing structure, structure provided with rigid floor diaphragm and the modified structure is compared in table 3.8 and this comparison shows the reduction in time period with modification in the structure. Similarly, table 3.9 presents the comparison of stresses induced at various parts of the same structures. The stress contour in the modified building is shown in figure 3.31.

# CHAPTER 4

## CONCLUSION

### 4.1 General:

This research work is conducted to find out the structural response of a traditional Nepalese building constructed of brick masonry in mud mortar strengthened with timber frames. The response of the structure in terms of natural time periods, deflections and stresses induced at the various locations under seismic excitation is determined and the critical sections in the building are identified. In order to make the building capable of withstanding the probable future earthquake, necessary strengthening measures are applied and the modified buildings are reanalyzed.

Since the existing condition of the floor cannot provide rigid floor diaphragm action, first modification was made on floors of the building making them rigid. The modified structure performs better than the existing one but still lacks fulfilling the required purpose and hence another modification is made reducing the windows opening sizes. This modification makes the existing structure enhanced, and also better the overall response of the structure both under gravity load and seismic excitation.

### 4.2 Major Conclusions:

Some of the important conclusions made after conducting this study are as follow:

1. An introduction of timber frames in masonry building improves the structural response.
2. Due to lack of experimental results, the analytical structural model is prepared according to model 8 (Wall and frame with Flexible springs and friction elements). If the reliable properties of NLlink elements are available, the model constructed

according to that of model 12 (Wall and frame with beam column joint having different plastic properties) is believed to give the most reliable response.

3. Although timber frames enhances the structural performance under seismic excitation, the major contributing element to withstand external load is brick masonry.
4. The presence of doors and windows contribute in controlling the localized stresses.
5. The existing flooring in building is substantially flexible and causes random distribution of stresses.
6. Haphazard type of mode shapes of structure is obtained in flexible floor diaphragms unlike in rigid floor diaphragms.
7. Introduction of rigid floor can control the random distribution of localized stresses, simplifies the deflection pattern of the building and hence, contributes greatly in enhancing the structural performance.
8. The spandrels and locations at connection between the walls of the existing building fail under tensile bending stress.
9. The wall connections and spandrels where geometric discontinuities dominate the geometric of a building are the most crucial places under seismic excitation.
10. Smaller the openings in the wall better the performance of the building.
11. The natural time period of the existing building does not follow the relationship given by IS 1893:2002 (part 1). The relation between the height of the building in m and its fundamental natural period in seconds is well defined by the relation $T = 0.0.0299H^{0.8202}$.
12. Using very simple techniques and locally available materials, the traditional buildings constructed of mud mortar strengthened with timber frames can be made safe against probable future earthquake.

## 4.3 Future Works:

1. The study is limited to linear phase only; it can be studied under material non-linearity and geometric non-linearity.
2. The separation of connection of the walls is ignored but it can be included in future work.
3. Due to lack of experimental results, the beam column joints are considered as rigid joint; if experimental results are available in future, the currently proposed 4 NLlink elements or any other better model can be implied at the joint between the timber beam and column.
4. The study is limited upto superstructure of the building, the future study can be conducted for the building including substructure also taking the soil-structure interaction into account.

## Table 3.1: Displacement of joints (m)

| Joint No. | Bare Frame | Bare Shell | Composite | Rigid Link | Offset | Gap |
|---|---|---|---|---|---|---|
| 5 | 0.00166 | 1.885E-06 | 2.031E-06 | 4.049E-06 | 2.063E-06 | 0.00E+00 |
| 4 | 0.00557 | 3.703E-06 | 3.693E-06 | 6.866E-06 | 3.894E-06 | 2.924E-06 |
| 3 | 0.01026 | 5.953E-06 | 5.579E-06 | 9.973E-06 | 6.065E-06 | 5.763E-06 |
| 2 | 0.01423 | 8.747E-06 | 7.819E-06 | 1.346E-05 | 8.588E-06 | 9.078E-06 |
| 1 | 0.016 | 1.748E-05 | 1.122E-05 | 1.915E-05 | 1.242E-05 | 1.266E-05 |

## Table 3.2: Material Properties

| | | | |
|---|---|---|---|
| **Brick Masonry** | | | |
| | | | |
| Unit Weight | | 19.5 | KN/m$^3$ |
| Young's Modulus of Elasticity, E | | 2000 | N/mm$^2$ |
| Poisson's ratio, $\nu$ | | 0.100 | |
| Ultimate Compressive Stress, $f_{cu}$ | | 7.260 | N/mm$^2$ |
| Ultimate Tensile Stress, $f_{tu}$ | | 0.225 | N/mm$^2$ |
| Ultimate Shear Stress, $f_{su}$ | | 0.500 | N/mm$^2$ |
| | | | |
| **Timber** | | | |
| | | | |
| Unit Weight | | 8.50 | KN/m$^3$ |
| Young's Modulus of Elasticity, E | | 19800 | N/mm$^2$ |
| Poisson's ratio, $\nu$ | | 0.12 | |
| Permissible Compressive Stress parallel to grain | Inside Location | 15.00 | N/mm$^2$ |
| | Outside Location | 13.30 | N/mm$^2$ |
| | Wet Location | 10.90 | N/mm$^2$ |
| Permissible Compressive Stress perpendicular to grain, $f_c$ | Inside Location | 12.90 | N/mm$^2$ |
| | Outside Location | 10.00 | N/mm$^2$ |
| | Wet Location | 8.20 | N/mm$^2$ |
| Permissible Tensile and Bending Stress along Grain, extreme fibre stress | Inside Location | 16.90 | N/mm$^2$ |
| | Outside Location | 14.00 | N/mm$^2$ |
| | Wet Location | 11.20 | N/mm$^2$ |
| Permissible Shear stress, All Location | Horizontal | 0.94 | N/mm$^2$ |
| | Along Grain | 1.34 | N/mm$^2$ |

**Table 3.3: Responses of Test Models (Natural Time Period and Displacement)**

| Model No. | Description | Time Period (sec) | | | Time Period from IS 1893:2002 $T = 0.09*H/\sqrt{d}$ | $\Delta x$ | $-\Delta x$ | $\Delta y$ | $-\Delta y$ |
|---|---|---|---|---|---|---|---|---|---|
| | | Mode 1 | Mode 2 | Mode 3 | | | | | |
| | | | | | | | | | |
| 1 | Bare Frame | 0.2329 | 0.2187 | 0.1813 | 0.0845 | 2.859 | -2.859 | 3.166 | -3.164 |
| 2 | Wall and Frame | 0.0394 | 0.0366 | 0.0343 | 0.0845 | 0.014 | -0.047 | 0.095 | -0.095 |
| 3 | Wall and frame with rigid links | 0.0392 | 0.0362 | 0.0340 | 0.0845 | 0.083 | -0.115 | 0.017 | -0.017 |
| 4 | Wall and column with Rigid Links and NLlinks | 0.0388 | 0.0397 | 0.0355 | 0.0845 | 0.102 | -0.117 | 0.110 | -0.111 |
| 5 | Wall and frame with Rigid Links and Friction elements | 0.0420 | 0.0395 | 0.0388 | 0.0845 | 0.110 | -0.123 | 0.064 | -0.106 |
| 6 | Wall and frame with Springs & Friction elements | 0.0521 | 0.0459 | 0.0451 | 0.0845 | 0.174 | -0.136 | 0.213 | -0.215 |
| 7 | Wall and frame with Diaphragm | 0.0425 | 0.0419 | 0.0415 | 0.0845 | 0.124 | -0.140 | 0.041 | -0.041 |
| 8 | Wall and frame with Flexible Spring & friction elements | 0.0548 | 0.0496 | 0.0484 | 0.0845 | 0.161 | -0.162 | 0.210 | -0.210 |
| 9 | Wall with Spring but without columns | 0.0482 | 0.0469 | 0.0446 | 0.0845 | 0.201 | -0.226 | 0.250 | -0.250 |
| 10 | Wall and frame Stiff Beam column joint | 0.0561 | 0.0516 | 0.0485 | 0.0845 | 0.158 | -0.162 | 0.231 | -0.231 |
| 11 | Wall and frame with Flexible Beam column joint | 0.0564 | 0.0485 | 0.0485 | 0.0845 | 0.357 | -0.362 | 0.234 | -0.234 |
| 12 | Wall and frame with beam column joint different plastic properties | 0.0564 | 0.0516 | 0.0485 | 0.0845 | 0.315 | -0.320 | 0.233 | -0.234 |

## Table 3.4: Stresses at various Locations of the Test Models

| Model No. | Description | $\sigma_{xx}$ (N/mm$^2$) | | $\sigma_{yy}$ (N/mm$^2$) | | $\sigma_{zz}$ (N/mm$^2$) | | $\tau_{xy}$ (N/mm$^2$) | | $\tau_{xz}$ (N/mm$^2$) | | $\tau_{yz}$ (N/mm$^2$) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | + ve | - ve | + ve | - ve | + ve | - ve | + ve | - ve | + ve | - ve | + ve | - ve |
| | | | | | | | | | | | | | |
| 1 | Bare Frame | | | | | | | | | | | | |
| 2 | Wall and Frame | 0.068 | -0.053 | 0.033 | -0.027 | 0.032 | -0.147 | -0.021 | -0.021 | 0.038 | -0.041 | 0.027 | -0.027 |
| 3 | Wall and frame with rigid links | 0.068 | -0.053 | 0.046 | -0.049 | 0.034 | -0.161 | 0.025 | -0.025 | 0.036 | -0.040 | 0.025 | -0.025 |
| 4 | Wall and column with Rigid Links and NLlinks | 0.065 | -0.062 | 0.051 | -0.054 | 0.028 | -0.103 | 0.028 | -0.028 | 0.022 | -0.038 | 0.044 | -0.044 |
| 5 | Wall and frame with Rigid Links and Friction elements | 0.062 | -0.066 | 0.080 | -0.076 | 0.046 | -0.106 | 0.034 | -0.034 | 0.023 | -0.040 | 0.046 | -0.046 |
| 6 | Wall and frame with Springs & Friction elements | 0.062 | -0.069 | 0.144 | -0.071 | 0.019 | -0.079 | 0.035 | -0.035 | 0.020 | -0.044 | 0.037 | -0.037 |
| 7 | Wall and frame with Diaphragm | 0.046 | -0.043 | 0.078 | 0.085 | 0.014 | -0.073 | 0.033 | -0.033 | 0.015 | -0.041 | 0.046 | -0.046 |
| 8 | Wall and frame with Flexible Spring & friction elements | 0.061 | -0.066 | 0.070 | -0.067 | 0.015 | -0.075 | 0.034 | -0.035 | 0.021 | -0.043 | 0.024 | -0.024 |
| 9 | Wall with Spring but without columns | 0.067 | -0.050 | 0.042 | 0.029 | 0.034 | -0.149 | 0.020 | -0.020 | 0.037 | -0.039 | 0.023 | -0.023 |
| 10 | Wall and frame Stiff Beam column joint | 0.069 | -0.068 | 0.068 | -0.066 | 0.018 | -0.080 | 0.028 | -0.028 | 0.022 | -0.046 | 0.029 | -0.029 |
| 11 | Wall and frame with Flexible Beam column joint | 0.070 | -0.068 | 0.060 | -0.058 | 0.017 | -0.080 | 0.026 | -0.026 | 0.023 | -0.046 | 0.029 | -0.029 |
| 12 | Wall and frame with beam column joint different plastic properties | 0.070 | -0.068 | 0.062 | -0.060 | 0.017 | -0.080 | 0.027 | -0.027 | 0.023 | -0.046 | 0.029 | -0.029 |

## Table 3.5: Time periods and frequencies of the building for different modes

| Mode | for Rigid Joint Between Beam and Column | | | | Beam and Column joint with 4 Nllinks | | | |
|---|---|---|---|---|---|---|---|---|
| | Period | Frequency | CircFreq | Eigenvalue | Period | Frequency | CircFreq | Eigenvalue |
| Unitless | Sec | Cyc/sec | rad/sec | rad$^2$/sec$^2$ | Sec | Cyc/sec | rad/sec | rad$^2$/sec$^2$ |
| | | | | | | | | |
| 1 | 0.199555 | 5.0112 | 31.486 | 991.37 | 0.194609 | 5.1385 | 32.286 | 1042.4 |
| 2 | 0.130114 | 7.6855 | 48.29 | 2331.9 | 0.127685 | 7.8318 | 49.209 | 2421.5 |
| 3 | 0.102006 | 9.8034 | 61.596 | 3794.1 | 0.093041 | 10.748 | 67.531 | 4560.5 |
| 4 | 0.097036 | 10.305 | 64.751 | 4192.7 | 0.086971 | 11.498 | 72.245 | 5219.3 |
| 5 | 0.096682 | 10.343 | 64.988 | 4223.5 | 0.075605 | 13.227 | 83.105 | 6906.5 |
| 6 | 0.087289 | 11.456 | 71.981 | 5181.3 | 0.064767 | 15.44 | 97.012 | 9411.2 |

**Table 3.6a: Storey Displacement for the building with Rigid Joint between Beam and Column**

| Height (m) | Displacement (mm) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Comb 1 | | | | Comb 2 | | | | Comb 3 | | | | Comb 4 | | | |
| | x | -x | y | -y | x | -x | y | -y | x | -x | y | -y | x | -x | y | -y |
| 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2.300 | 0.854 | -0.866 | 0.056 | -0.061 | 0.038 | -0.041 | 0.251 | -0.288 | 0.849 | -0.874 | 0.058 | -0.062 | 0.038 | -0.045 | 0.254 | -0.296 |
| 4.600 | 2.187 | -2.209 | 0.085 | -0.086 | 0.081 | -0.087 | 0.653 | -0.690 | 2.174 | -2.222 | 0.087 | -0.098 | 0.074 | -0.092 | 0.647 | -0.695 |
| 6.900 | 3.645 | -3.691 | 0.197 | -0.175 | 0.112 | -0.122 | 0.906 | -0.901 | 3.622 | -3.714 | 0.206 | -0.169 | 0.105 | -0.135 | 0.917 | -0.901 |
| 7.800 | 4.339 | -4.398 | 0.308 | -0.274 | 0.110 | -0.122 | 1.076 | -1.164 | 4.312 | -4.426 | 0.315 | -0.267 | 0.104 | -1.320 | 1.100 | -1.198 |
| 9.770 | 6.052 | -6.077 | 0.076 | -0.086 | 0.051 | -0.083 | 1.143 | -1.183 | 6.025 | -6.104 | 0.074 | -0.091 | 0.024 | -0.122 | 1.138 | -1.188 |

**Table 3.6b: Storey Displacement for the building with Joint between Beam and Column modeled by 4 Nllinks**

| Height (m) | Displacement (mm) | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Comb 1 | | | | Comb 2 | | | | Comb 3 | | | | Comb 4 | | | |
| | x | -x | y | -y | x | -x | y | -y | x | -x | y | -y | x | -x | y | -y |
| 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 2.300 | 0.882 | -0.902 | 0.064 | -0.071 | 0.040 | -0.042 | 0.289 | -0.328 | 0.882 | -0.902 | 0.064 | -0.071 | 0.040 | -0.042 | 0.289 | -0.328 |
| 4.600 | 2.509 | -2.531 | 0.113 | -0.139 | 0.171 | -0.164 | 0.886 | -0.906 | 2.509 | -2.531 | 0.113 | -0.139 | 0.171 | -1.640 | 0.885 | -0.906 |
| 6.900 | 4.276 | -4.298 | 0.308 | -0.298 | 0.122 | -0.115 | 0.981 | -0.999 | 4.276 | -4.294 | 0.308 | -0.298 | 0.122 | -0.115 | 0.981 | -0.999 |
| 7.800 | 5.078 | -5.081 | 0.448 | -0.424 | 0.123 | -0.116 | 1.181 | -1.155 | 5.078 | -5.082 | 0.448 | -0.424 | 0.123 | -0.116 | 1.181 | -1.155 |
| 9.770 | 6.949 | -6.833 | 0.065 | -0.085 | 0.176 | -0.193 | 1.225 | -1.260 | 6.949 | -6.833 | 0.065 | -0.085 | 0.176 | -0.193 | 1.225 | -1.260 |

**Table 3.7: Maximum Stresses in the Building**

| S.N. | Stress | | Fixed Beam Column Joint | Beam Column Joint Modeled with 4 NLlinks |
|---|---|---|---|---|
| | | | $N/mm^2$ | $N/mm^2$ |
| 1 | S11 | Tensile | 1.0151 | 1.0330 |
| | | Compressive | -0.9048 | -1.0278 |
| 2 | S22 | Tensile | 0.4459 | 0.5166 |
| | | Compressive | -0.9897 | -0.8959 |
| 3 | S33 | Tensile | 0.6015 | 0.5601 |
| | | Compressive | -0.9044 | -0.7547 |
| 4 | S12 | Positive | 0.3209 | 0.3365 |
| | | Negative | -0.3261 | -0.3365 |
| 5 | S13 | Positive | 0.3727 | 0.3279 |
| | | Negative | -0.3619 | -0.3205 |
| 6 | S23 | Positive | 0.2295 | 0.2479 |
| | | Negative | -0.2149 | -0.2429 |

### Table 3.8: Comparison of Time periods and frequencies of the building for different modes

| Mode | Existing Building | | | Building with Rigid Floor Diaphragm | | | | Modified Building | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Period | Frequency | CircFreq | Period | Frequency | CircFreq | Eigenvalue | Period | Frequency | CircFreq |
| | Sec | Cyc/sec | rad/sec | Sec | Cyc/sec | rad/sec | rad2/sec2 | Sec | Cyc/sec | rad/sec |
| | | | | | | | | | | |
| 1 | 0.19956 | 5.0112 | 31.486 | 1.00000 | 5.6746 | 35.655 | 1271.2 | 0.15234 | 6.5643 | 41.245 |
| 2 | 0.13011 | 7.6855 | 48.29 | 0.08353 | 11.972 | 75.224 | 5658.6 | 0.08403 | 11.901 | 74.776 |
| 3 | 0.10201 | 9.8034 | 61.596 | 0.08318 | 12.022 | 75.534 | 5705.3 | 0.07954 | 12.573 | 78.996 |
| 4 | 0.09704 | 10.305 | 64.751 | 0.08152 | 12.267 | 77.077 | 5940.8 | 0.07540 | 13.263 | 83.333 |
| 5 | 0.09668 | 10.343 | 64.988 | 0.05364 | 18.643 | 117.14 | 13721 | 0.05361 | 18.655 | 117.21 |
| 6 | 0.08729 | 11.456 | 71.981 | 0.04604 | 21.723 | 136.49 | 18629 | 0.04351 | 22.983 | 144.41 |

### Table 3.9: Comparision of Maximum Stresses in the Buildings

| S.N. | Stress | | Existing Building | Building with Rigid Floor Diaphragm | Modified Building |
|---|---|---|---|---|---|
| | | | N/mm$^2$ | N/mm$^2$ | N/mm$^2$ |
| 1 | S11 | Tensile | 1.0151 | 0.4038 | 0.2084 |
| | | Compressive | -0.9048 | -0.4815 | -0.5543 |
| 2 | S22 | Tensile | 0.4459 | 0.2821 | 0.2075 |
| | | Compressive | -0.9897 | -0.2082 | -0.2287 |
| 3 | S33 | Tensile | 0.6015 | 0.5028 | 0.2199 |
| | | Compressive | -0.9044 | -0.8025 | -0.6051 |
| 4 | S12 | Positive | 0.3209 | 0.1245 | 0.1928 |
| | | Negative | -0.3261 | -0.1263 | -0.1941 |
| 5 | S13 | Positive | 0.3727 | 0.3615 | 0.3244 |
| | | Negative | -0.3619 | -0.3746 | -0.3350 |
| 6 | S23 | Positive | 0.2295 | 0.1939 | 0.2360 |
| | | Negative | -0.2149 | -0.2042 | -0.2107 |

### Table 3.10 Relation Between Height of Building and Fundamental Time Period

| Building Having No of Storey | Height of Building, H | Time Period from Free Vibration (Mode 1) | Time Period from IS 1893:2002 $T = 0.09H/\sqrt{d_s}$ (Mode 1) | Time Period from IS 1893:2002 $T = 0.09H/\sqrt{d_L}$ (Mode 2) | Time Period Emperical Relation, T $=0.0299H^{0.8202}$ |
|---|---|---|---|---|---|
| 4 | 9.77 | 0.1946 | 0.3754 | 0.3434 | 0.1939 |
| 3 | 7.47 | 0.1522 | 0.2870 | 0.2626 | 0.1556 |
| 2 | 5.17 | 0.1176 | 0.1986 | 0.1817 | 0.1150 |
| 1 | 2.87 | 0.0704 | 0.1103 | 0.1009 | 0.0710 |

Figure 3.1: Traditional Masonry Dwelling House

Figure 3.2: Traditional timber frame with Column surmounted by Capitals “Meth”

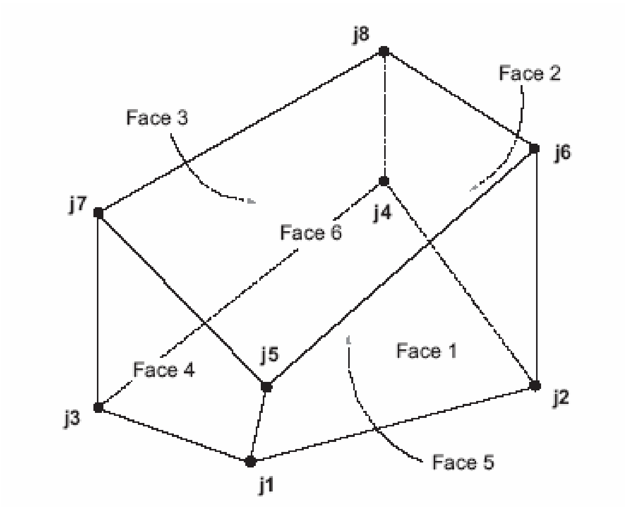


Figure 3.3: Solid Element Joint Connectivity and Face Definition

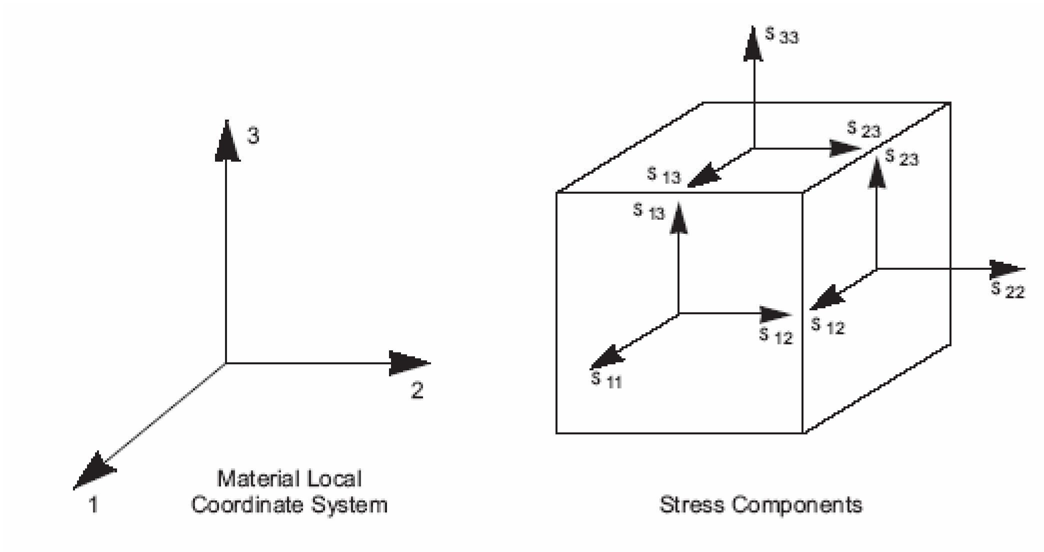


Figure 3.4: Definition of Stress Components in the Material Local Coordinate System

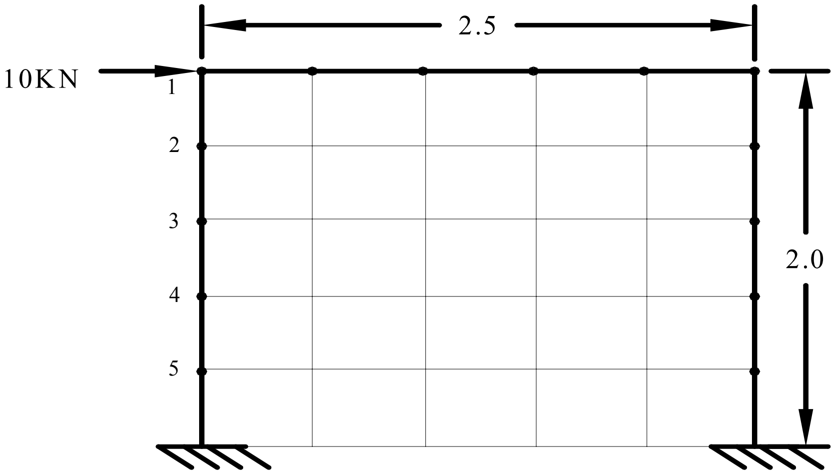


Figure 3.5: Test Model for the selection of contact elements

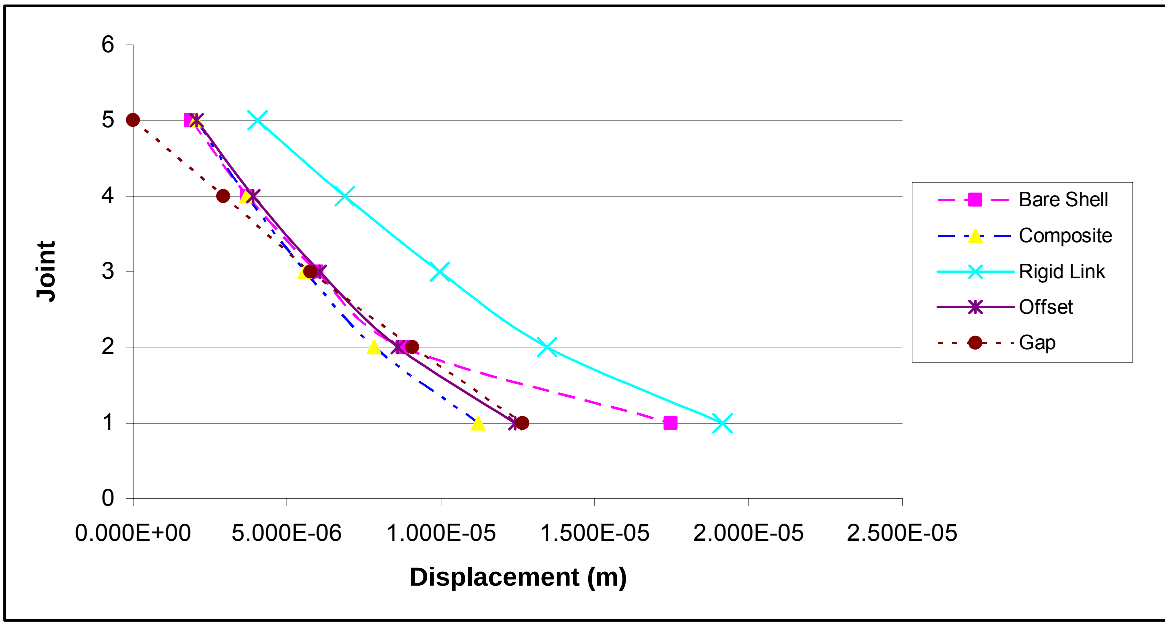
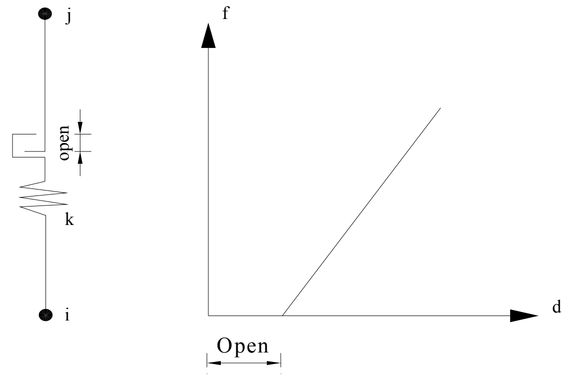


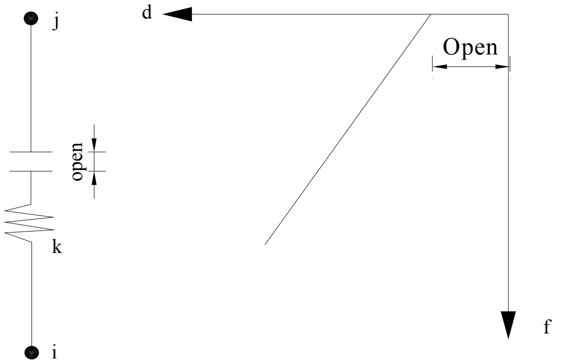
Figure 3.6: Displacement at different levels of test model for the selection of contact elements



Hook Element

Hook Element
force-displacement relationship

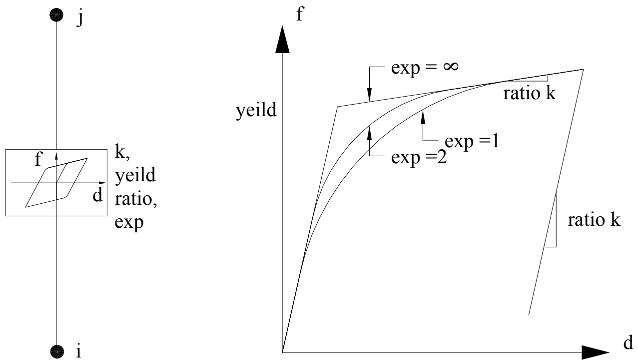
Figure 3.7: Hook Element and Hook element force-displacement relationship



Gap Element

Gap Element
force-displacement relationship

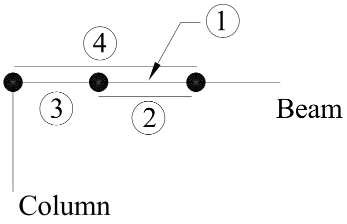
Figure 3.8: Gap Element and Gap element force-displacement relationship



Plastic1 Element

Plastic1 Element
force-displacement relationship

Figure 3.9: Plastic1 element and Plastic1 element force-displacement relationship



1 - Gap Element
2 - Hook Element
3 - Plastic1 Element
4 - Plastic1 Element

Figure 3.10: Typical joint of a timber frame modeled with four Nlink Elements

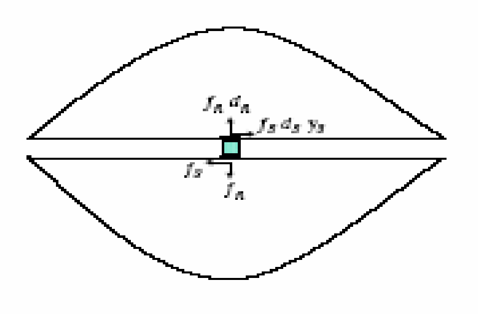


Figure 3.11: Three dimensional Nonlinear Friction-Gap Element

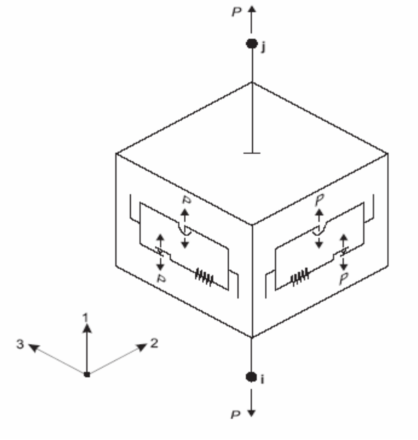


Figure 3.12: Friction-Pendulum Isolator Property for Biaxial Shear Behavior

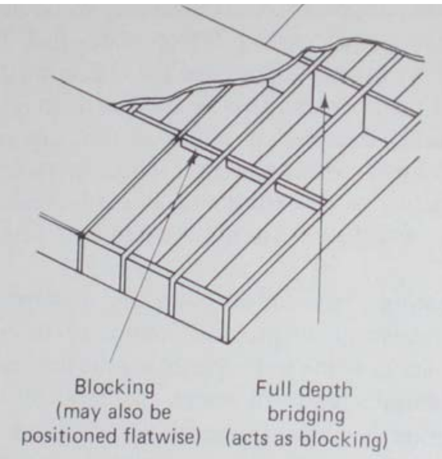
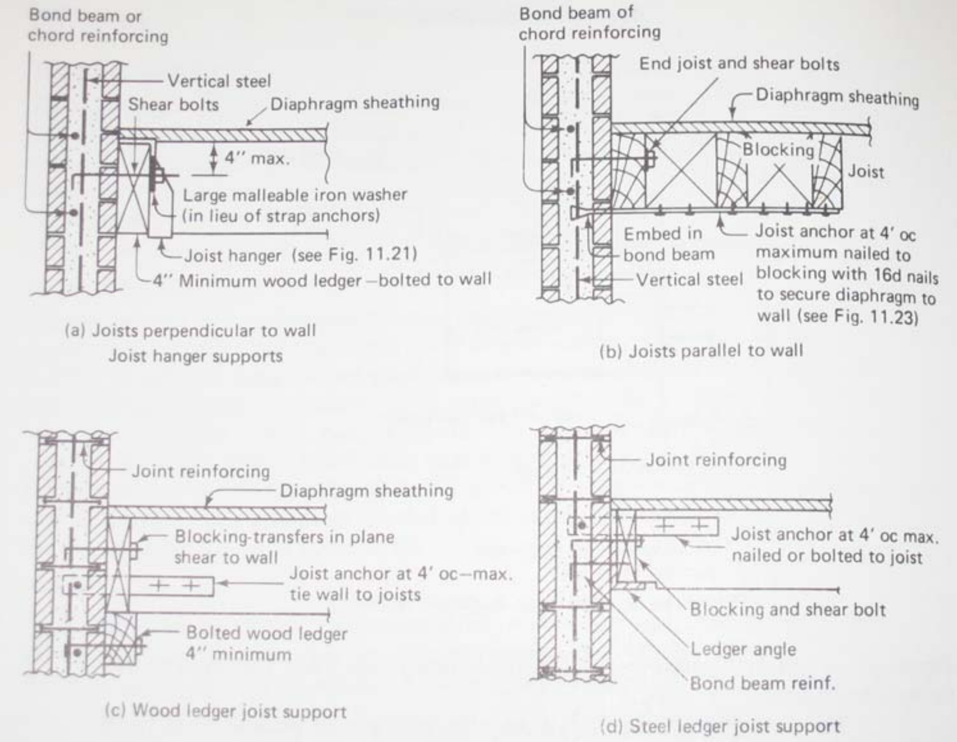


Figure 3.13: Blocking in plywood diaphragm construction



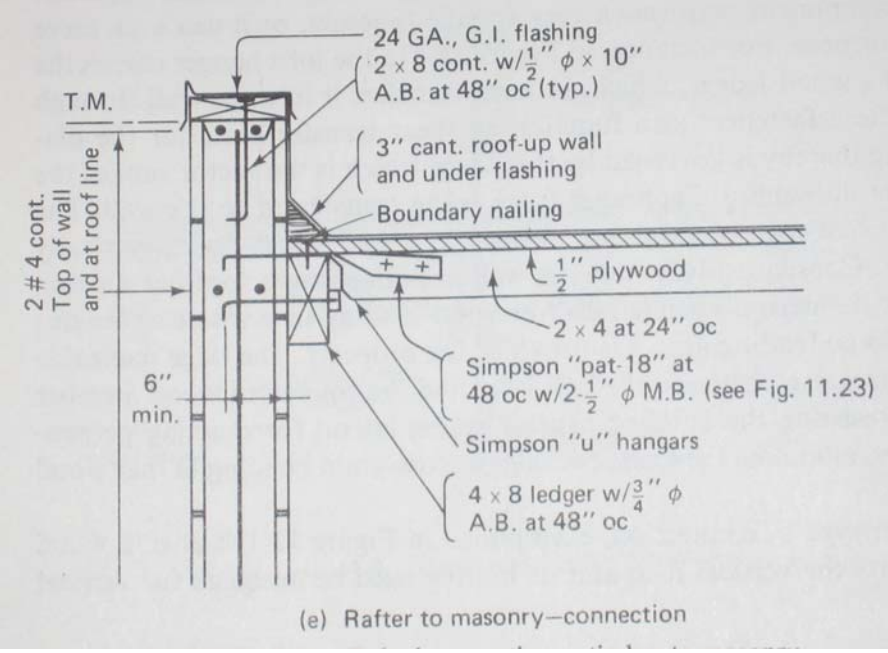


Figure: 3.14: Bolted connections – timber to masonry (Courtesy: ref 24)

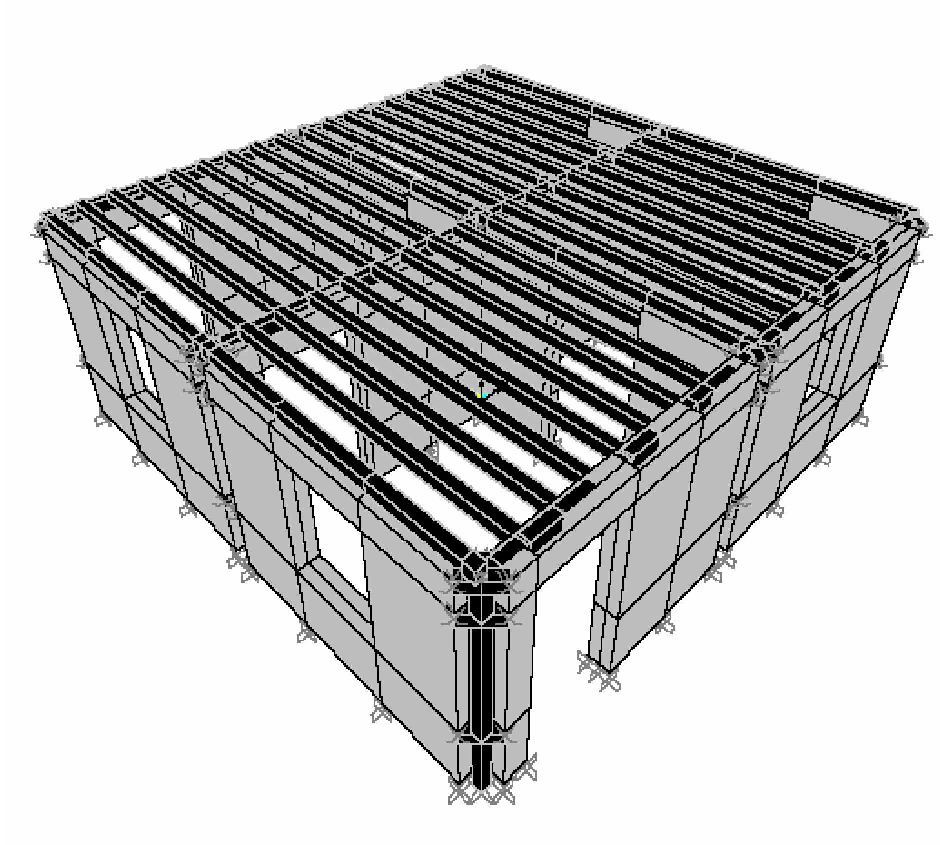


Figure 3.15: An ideal one storey building for Analytical Test

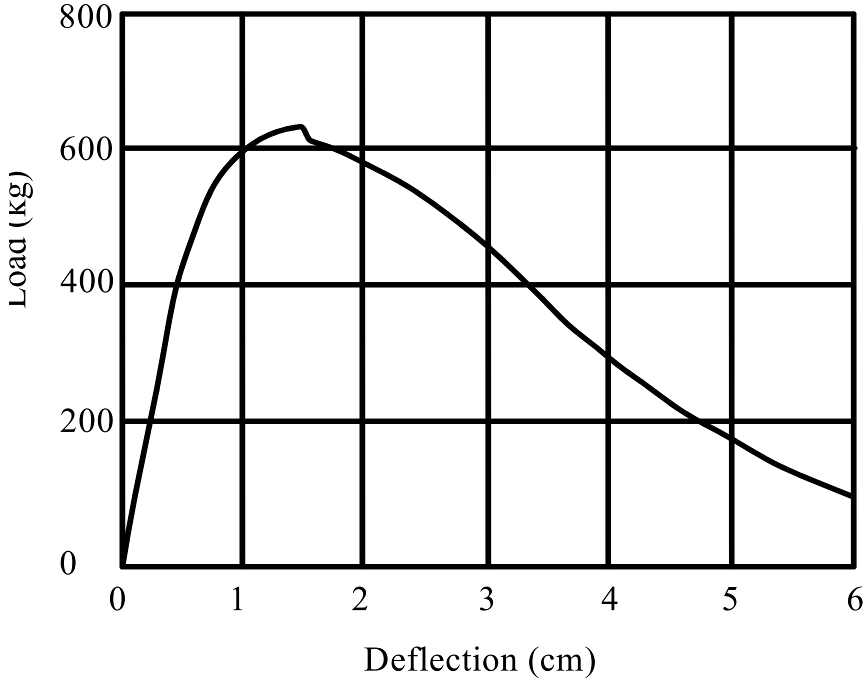


Figure 3.16: Modulus of Elasticity of Sal Wood

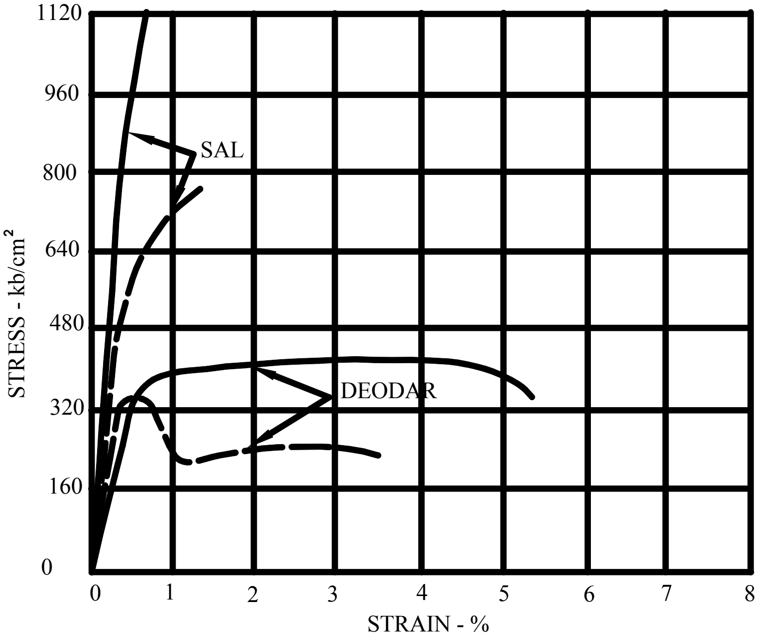


Figure 3.17: Typical Stress-Strain Curves of Timber

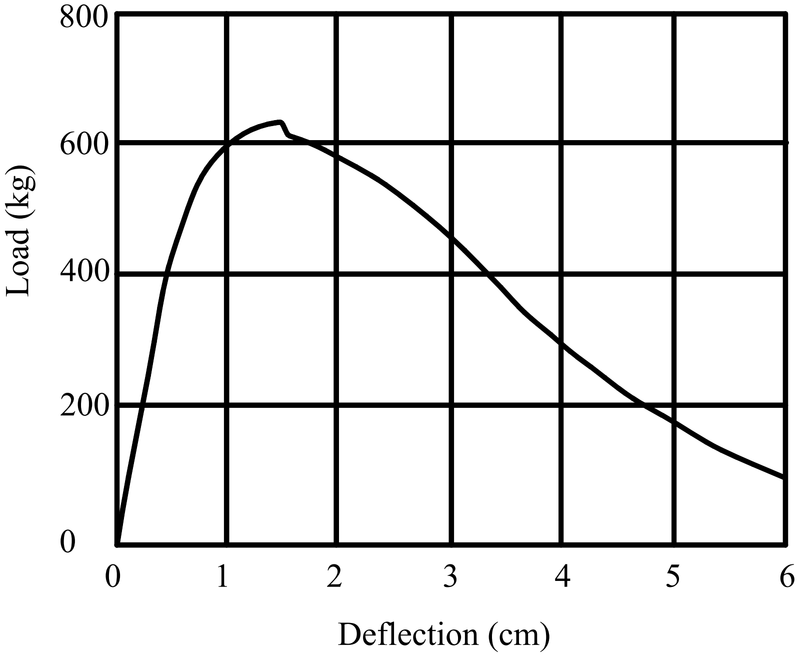


Figure 3.18: Typical Beam Test of Timber

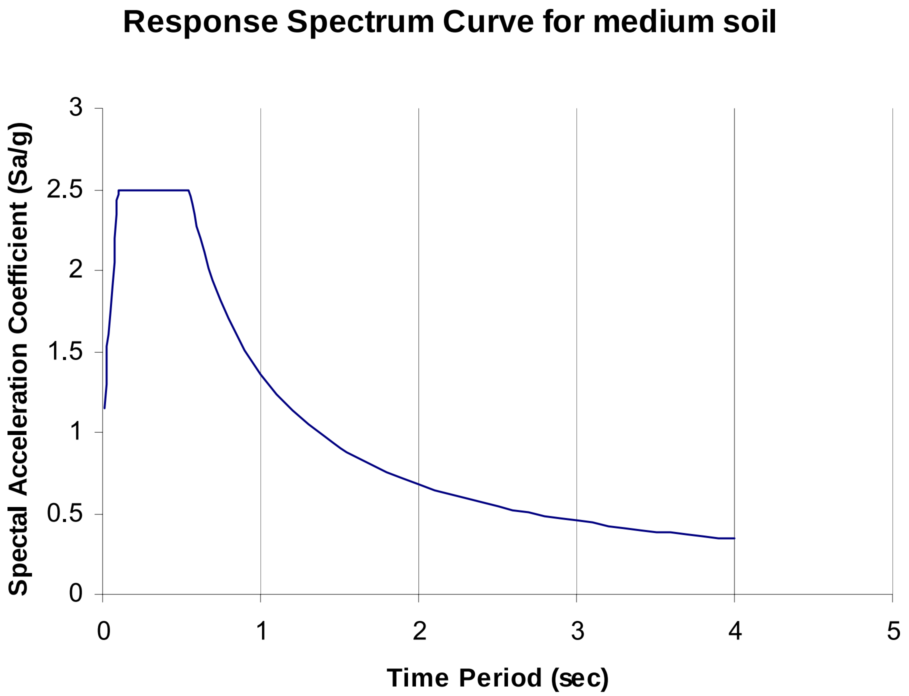


Figure 3.19 Response Spectra for medium soil site for 5% Damping

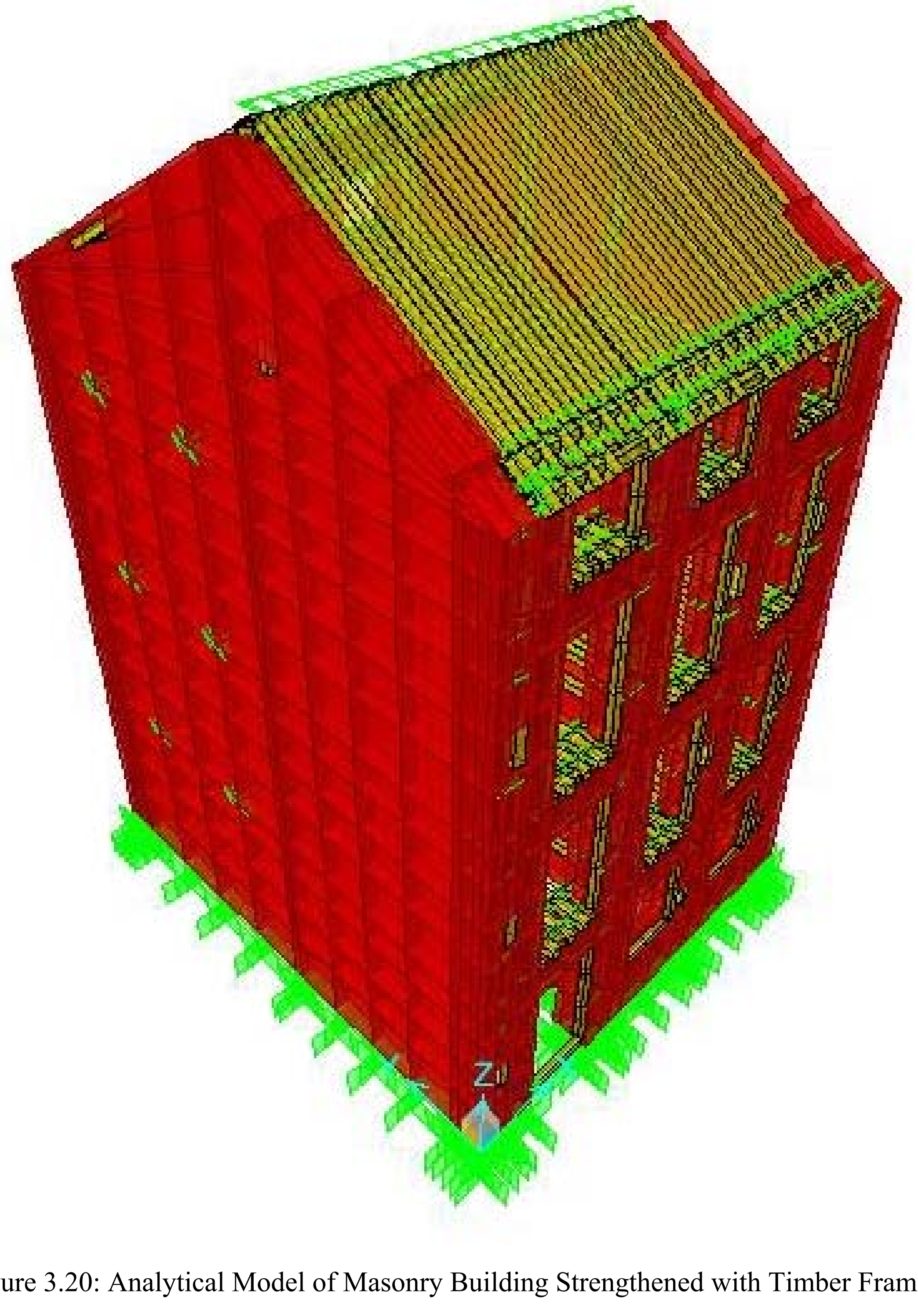


Figure 3.20: Analytical Model of Masonry Building Strengthened with Timber Frames

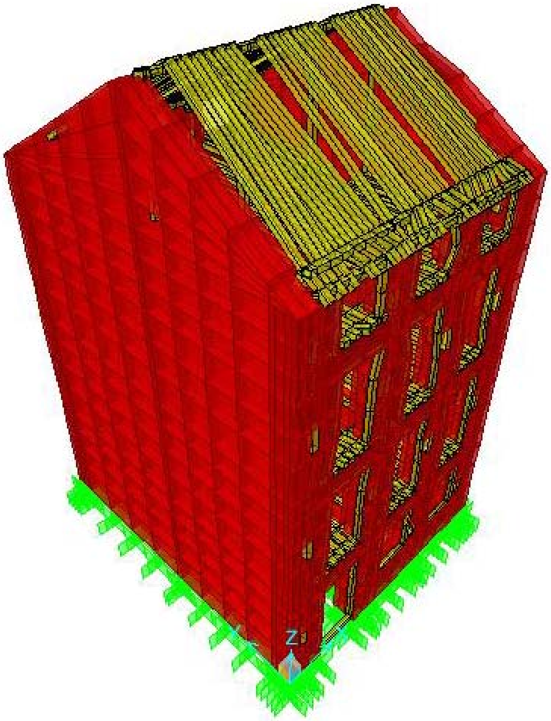


Figure 3.21: Deformed Shape of Building under Comb1 (Earthquake in x-direction)

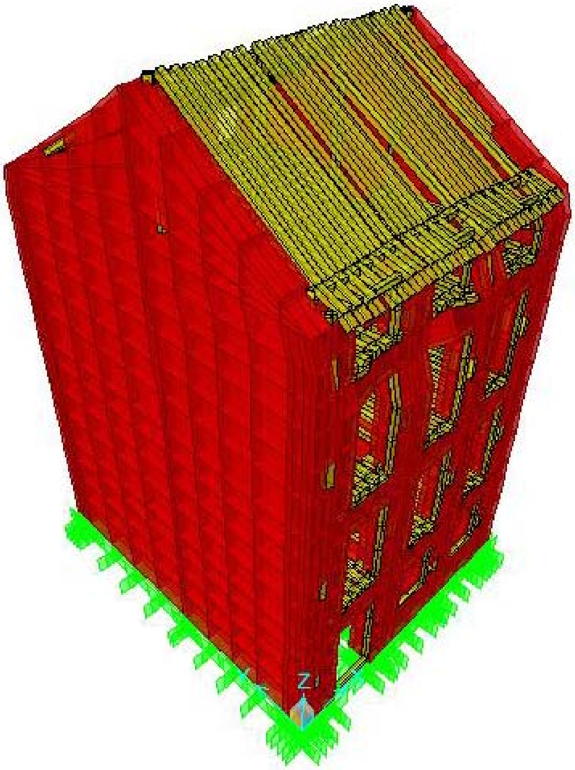


Figure 3.22: Deformed Shape of Building under Comb2 (Earthquake in y-direction)

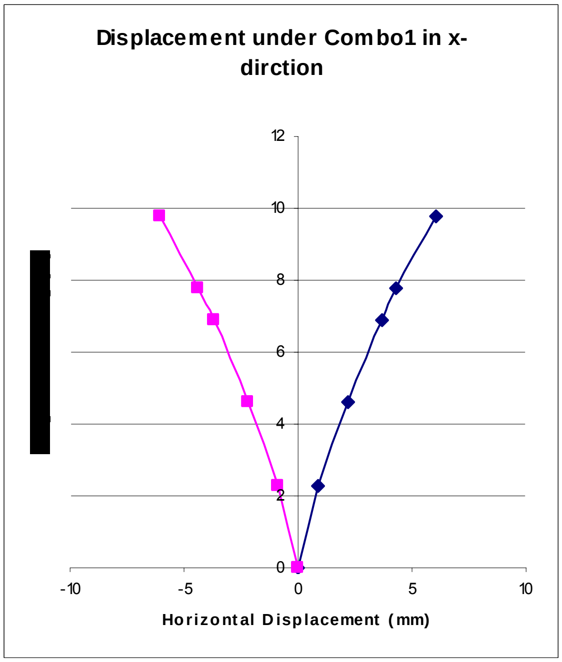


Figure: a

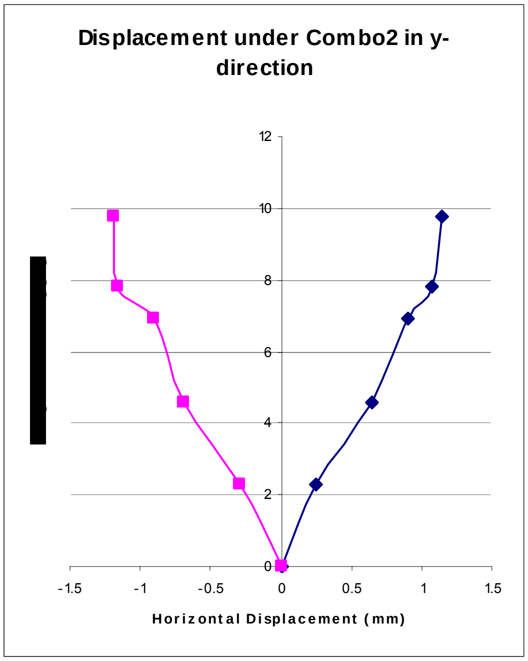


Figure: b

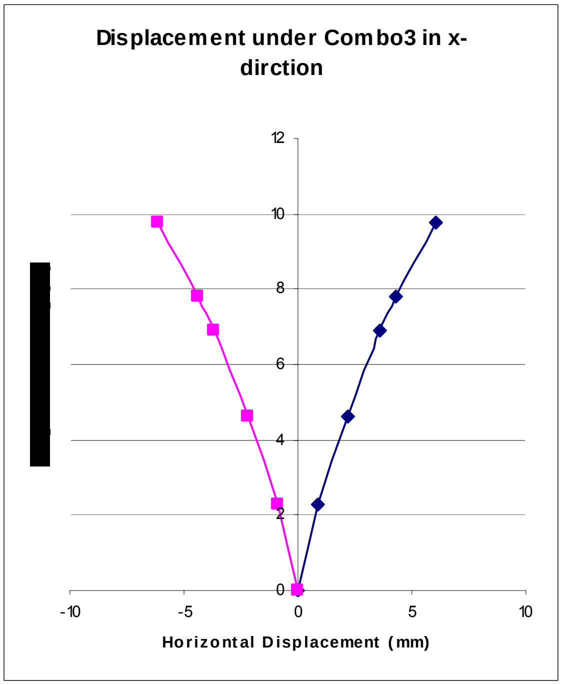


Figure: c

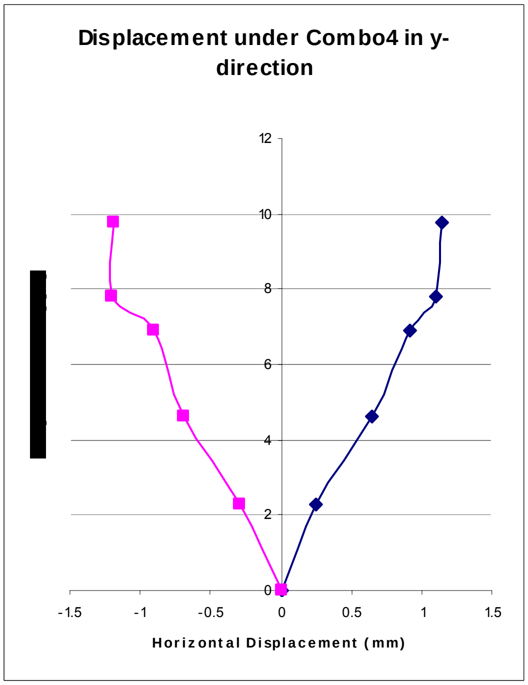


Figure: d

Figure 3.23: Storey Displacement of existing building

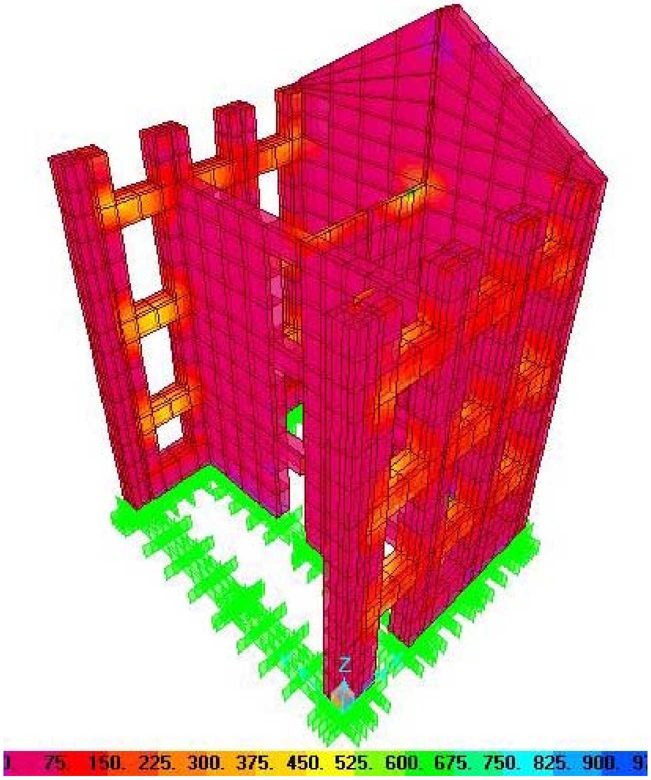


Figure 3.24a: Tensile Bending Stress (S11) Contour under Comb 3

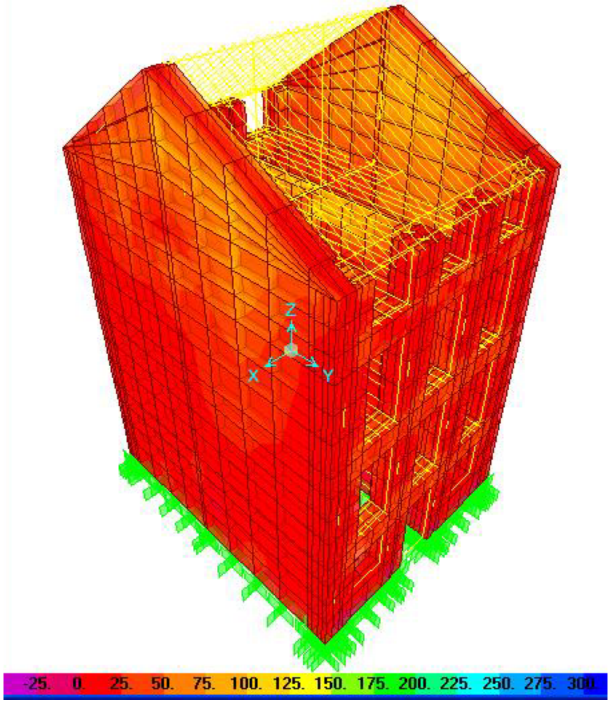


Figure 3.24b: Tensile Bending Stress (S22) Contour under Comb 3

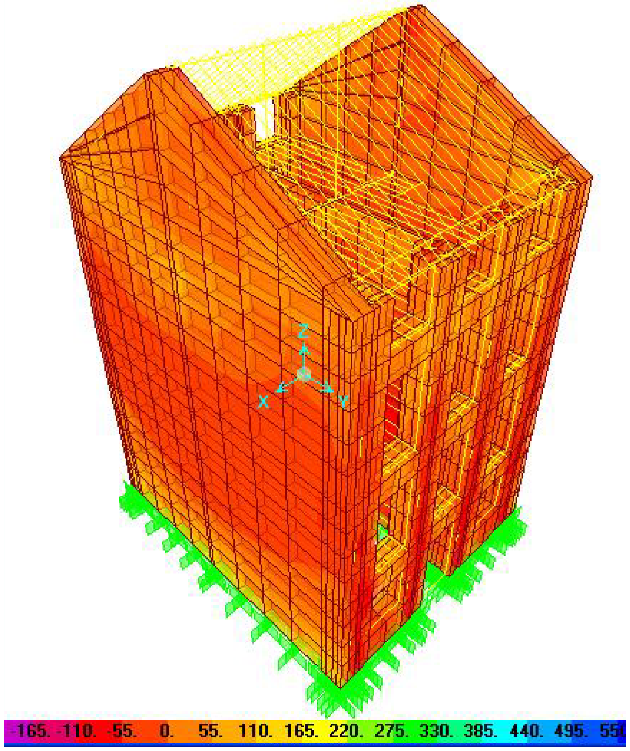


Figure 3.24c: Tensile Bending Stress (S33) Contour under Comb 3

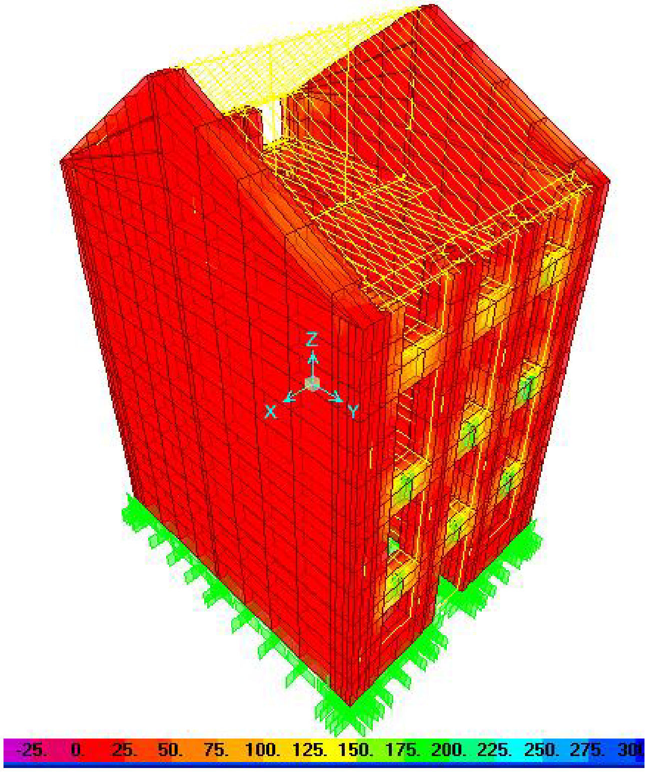


Figure 3.24d: Shear Stress (S12) Contour under Comb 3

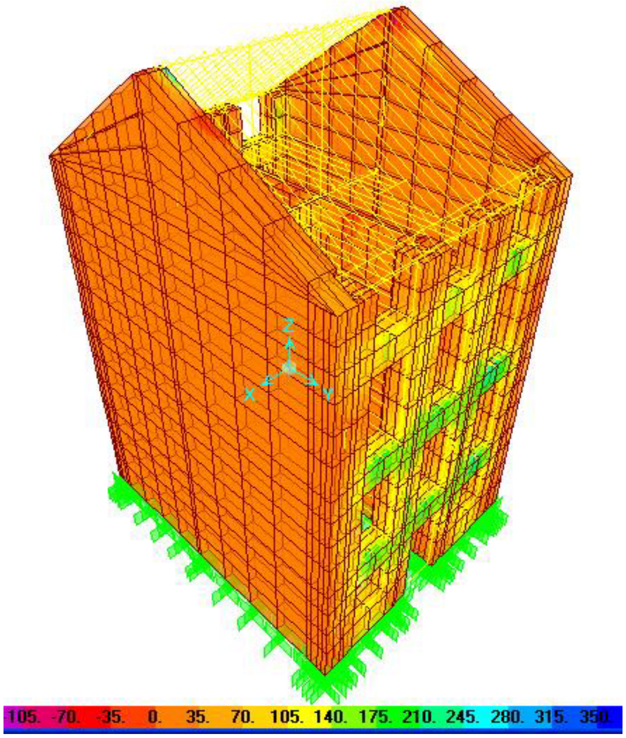


Figure 3.24e: Shear Stress (S13) Contour under Comb 3

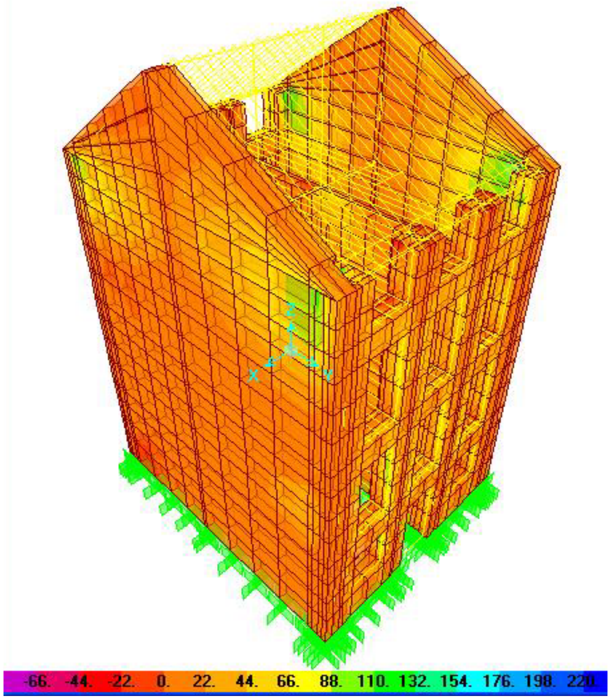


Figure 3.21f: Shear Stress (S23) Contour under Comb 3

Figure 3.24: Stress contour in building due to Load Combination 3 (x-direction)

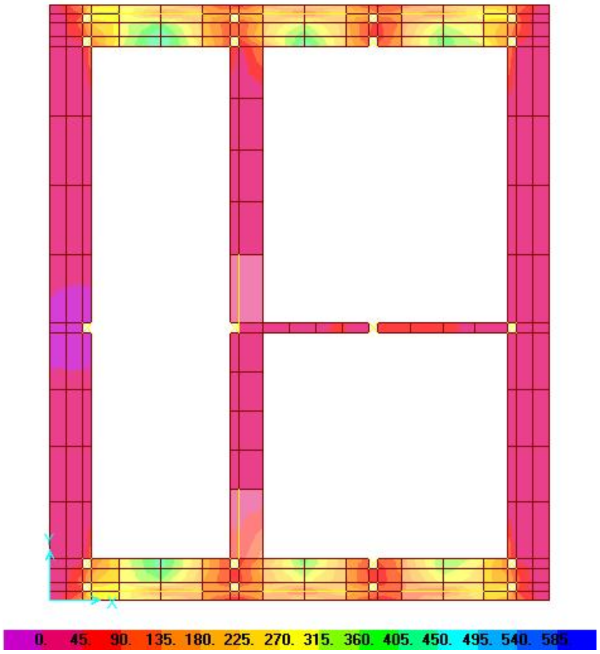
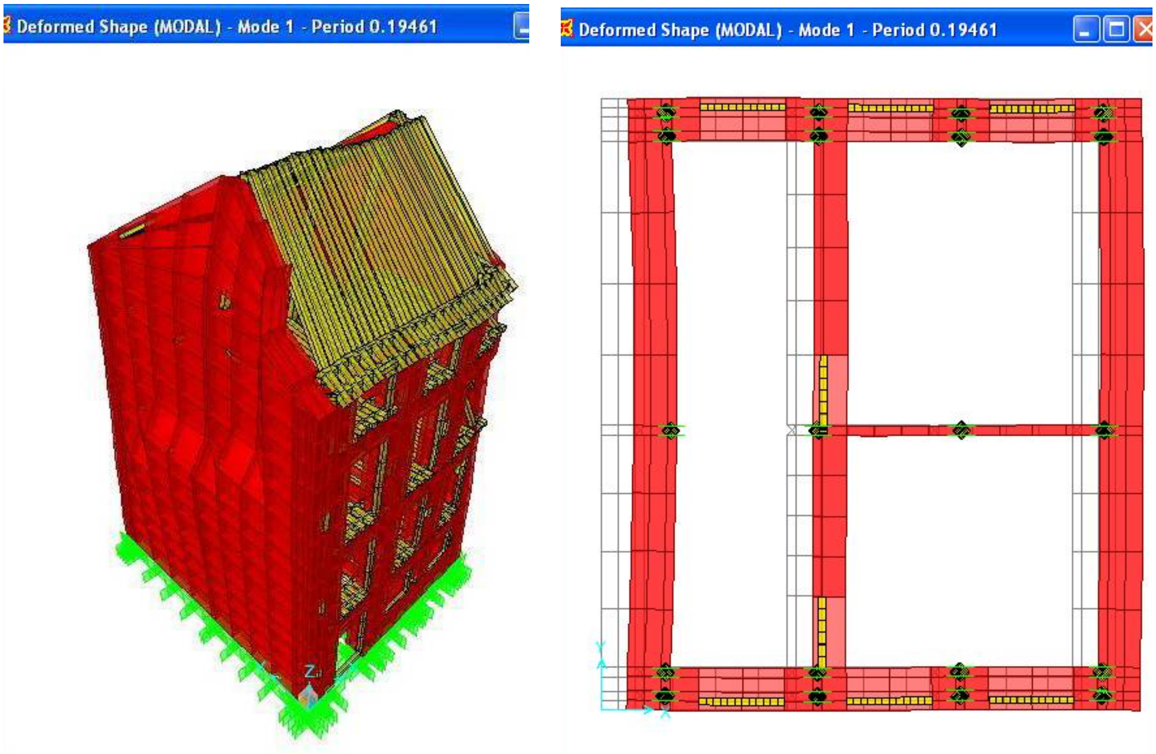
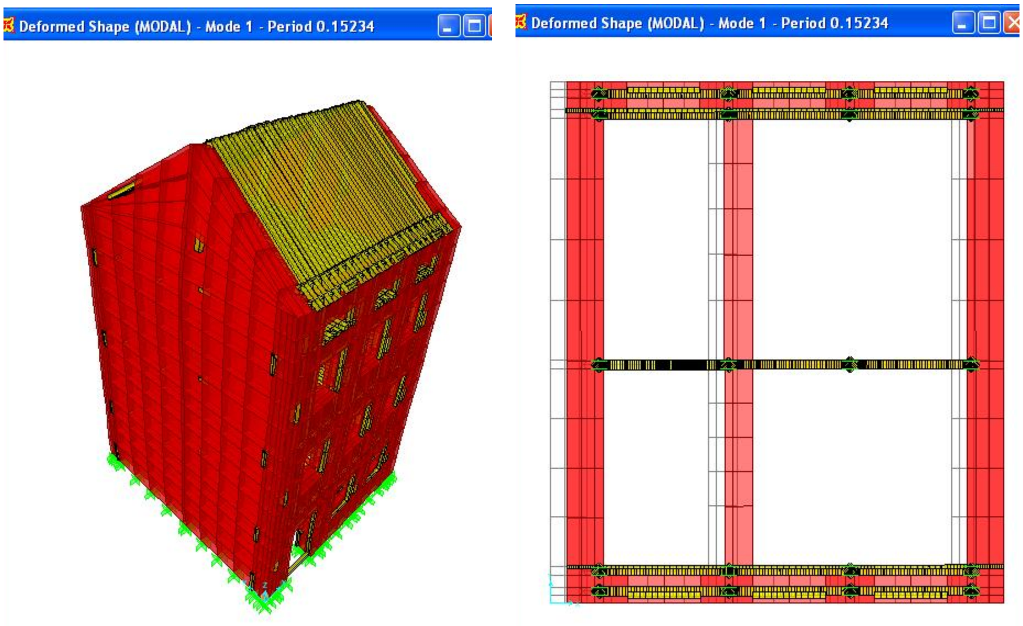


Figure 3.25: Variation in Tensile Bending Stress in Wall Section



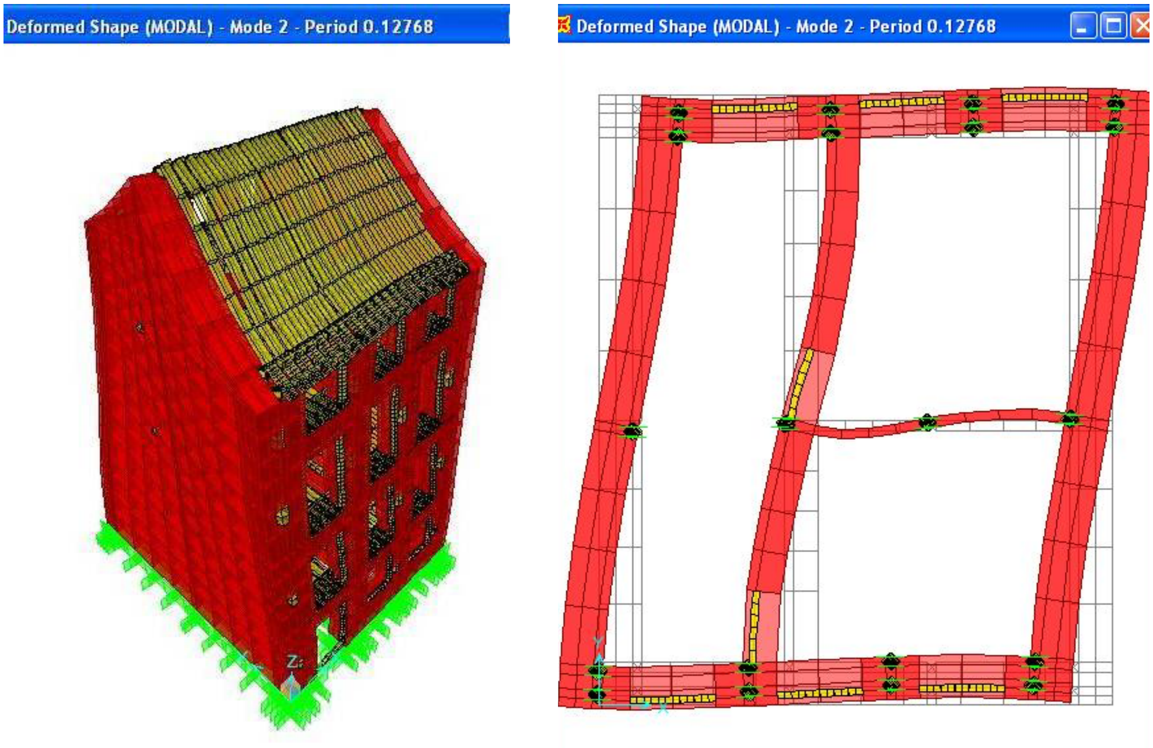
a) 3D view b) Plan view

Figure 3.26a: Mode Shape of Existing Building (Mode 1)



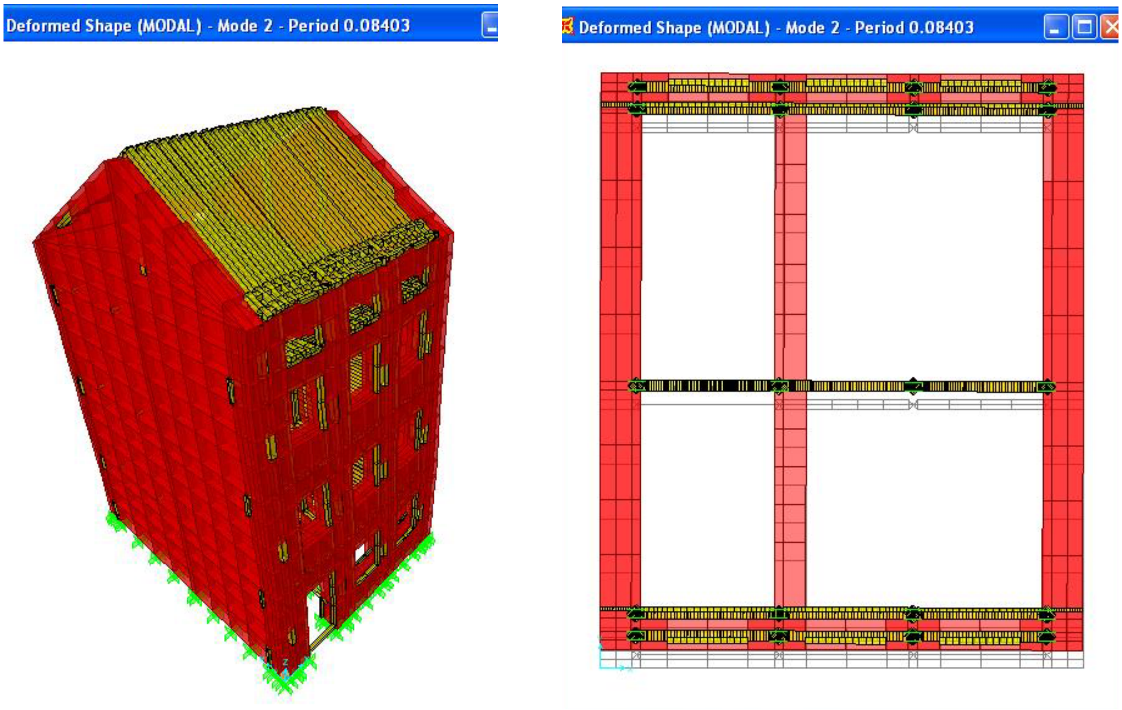
a) 3D view b) Plan view

Figure 3.26b: Mode Shape of Modified Building (Mode 1)



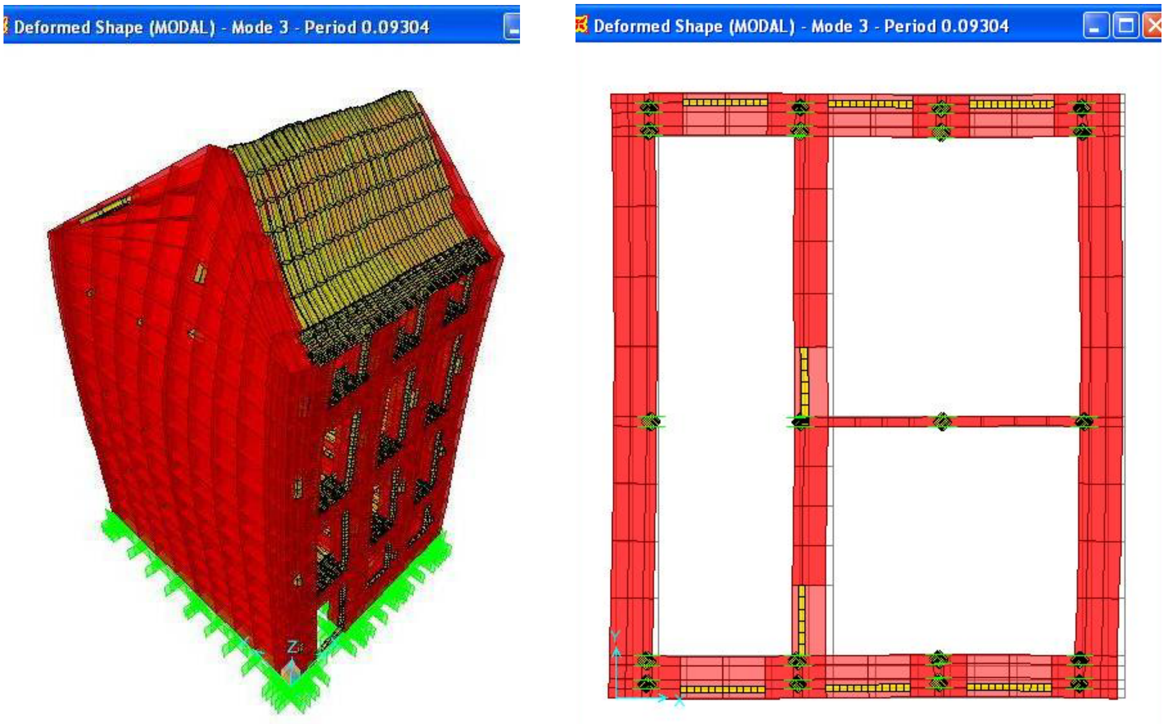
a) 3D view b) Plan view

Figure 3.26c: Mode Shape of Existing Building (Mode 2)



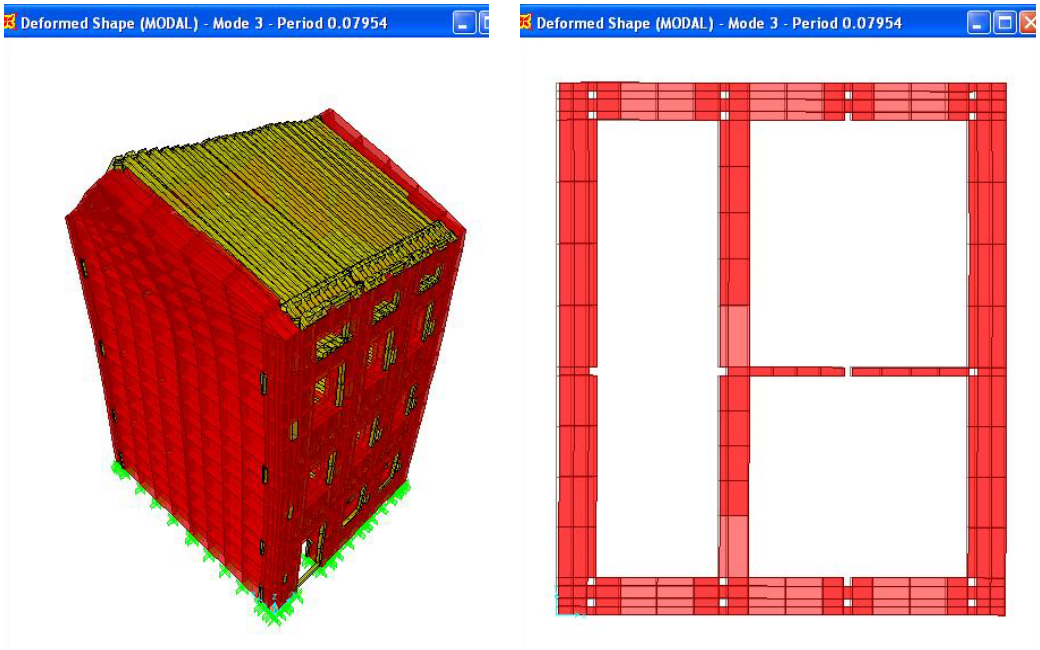
a) 3D view b) Plan view

Figure 3.26d: Mode Shape of Modified Building (Mode 2)



a) 3D view b) Plan view

Figure 3.26e: Mode Shape of Existing Building (Mode 3)



a) 3D view b) Elevation view

Figure 3.26f: Mode Shape of Modified Building (Mode 3)

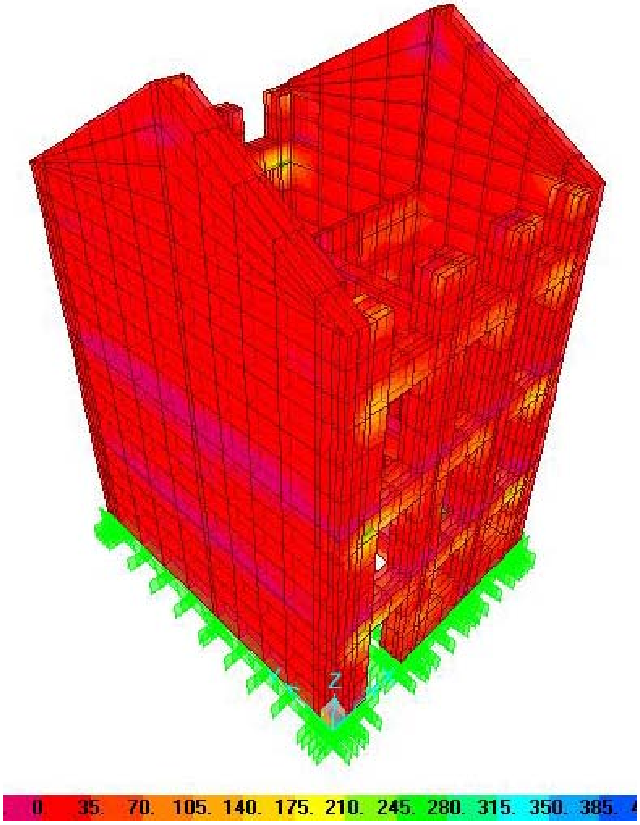


Figure 3.27: Tensile Bending Stress (S11) Contour under Comb 3 in Building with Rigid Floor Diaphragm

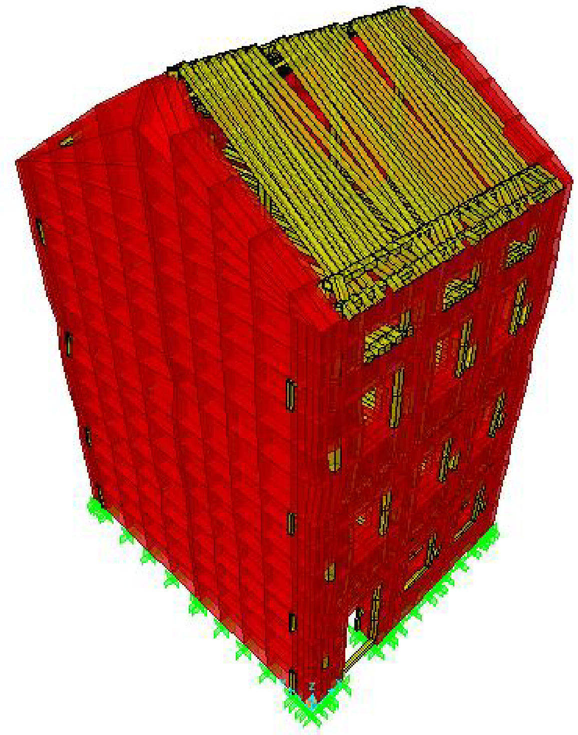


Figure 3.28: Deformed Shape of Building under Comb3 (Earthquake in x-direction)

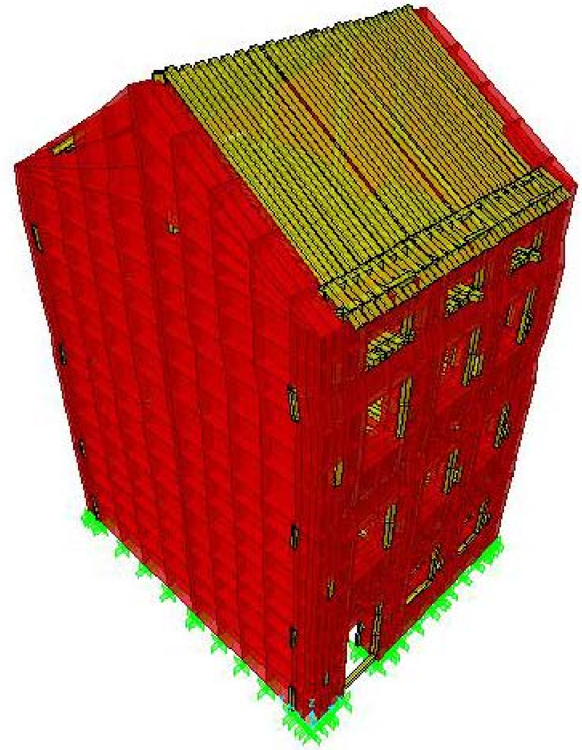


Figure 3.29: Deformed Shape of Building under Comb2 (Earthquake in y-direction)

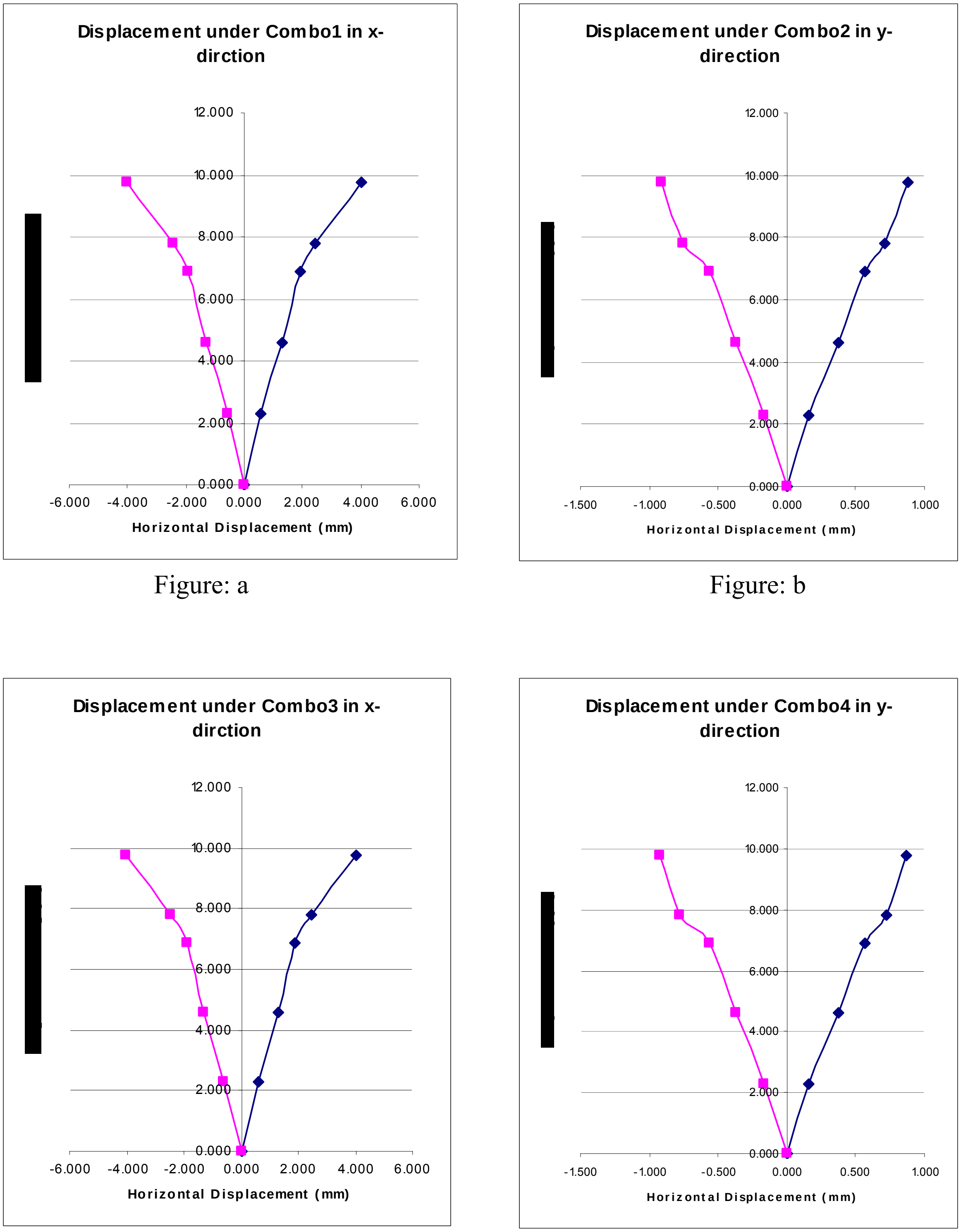


Figure: a

Figure: b

Figure: c

Figure: d

Figure 3.30: Storey Displacement of modified building

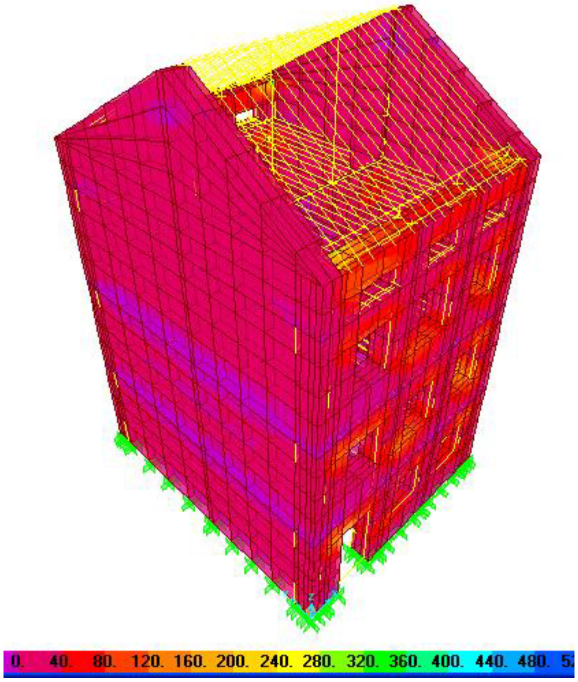


Figure 3.31a: Tensile Bending Stress (S11) Contour under Comb 3 in Modified Building

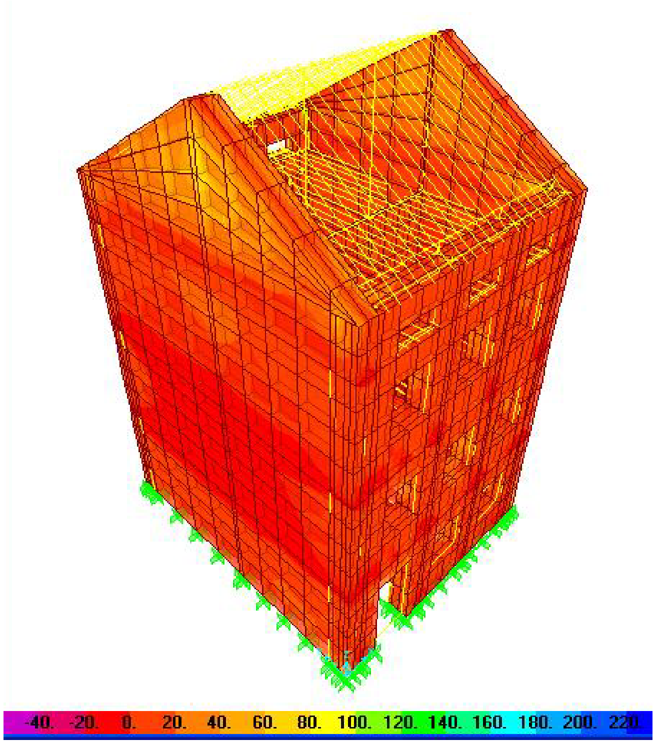


Figure 3.31b: Tensile Bending Stress (S22) Contour under Comb 3 in Modified Building

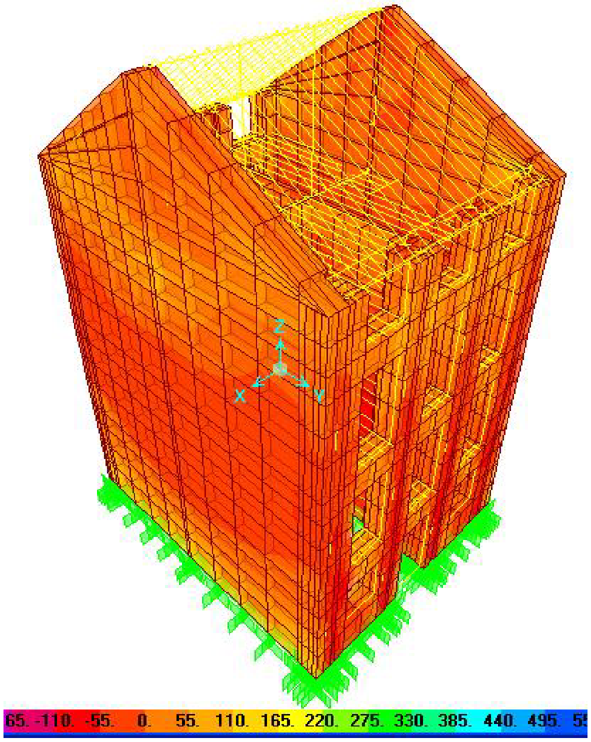


Figure 3.31c: Tensile Bending Stress (S33) Contour under Comb 3 in Modified Building

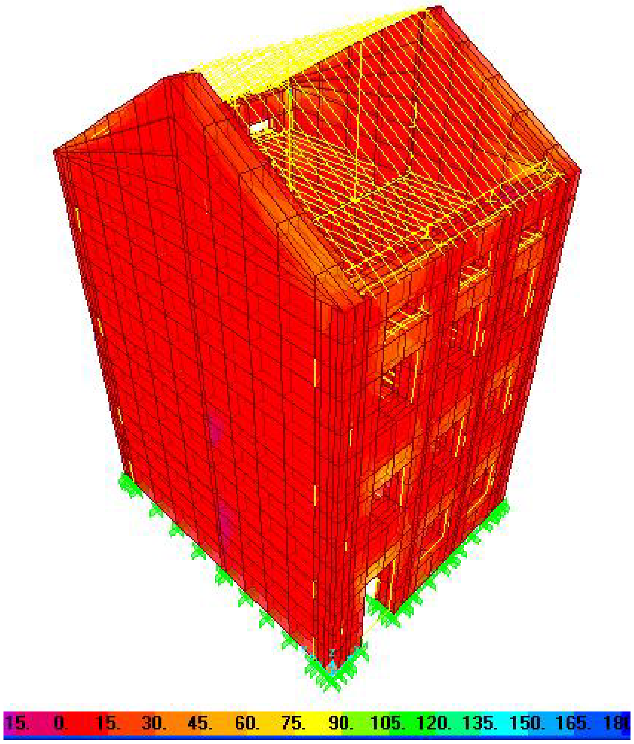


Figure 3.31d: Shear Stress (S12) Contour under Comb 3 in Modified Building

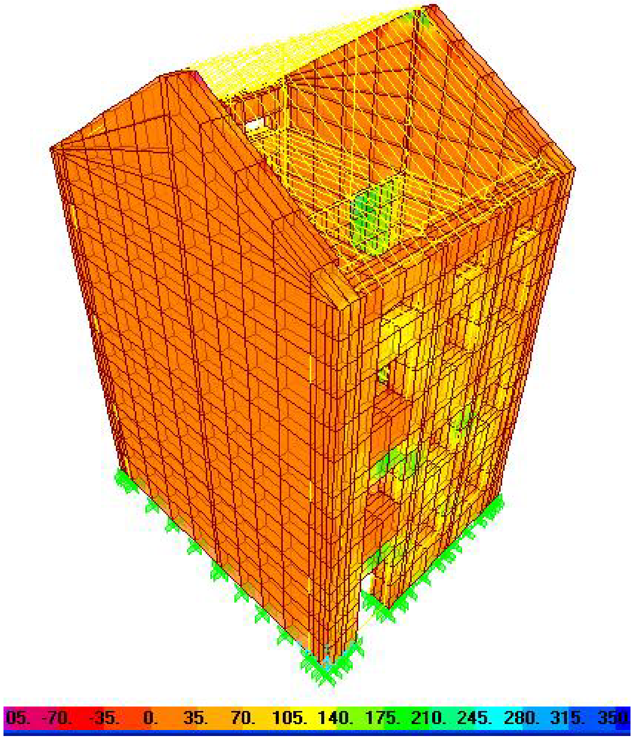


Figure 3.31e: Shear Stress (S13) Contour under Comb 3 in Modified Building

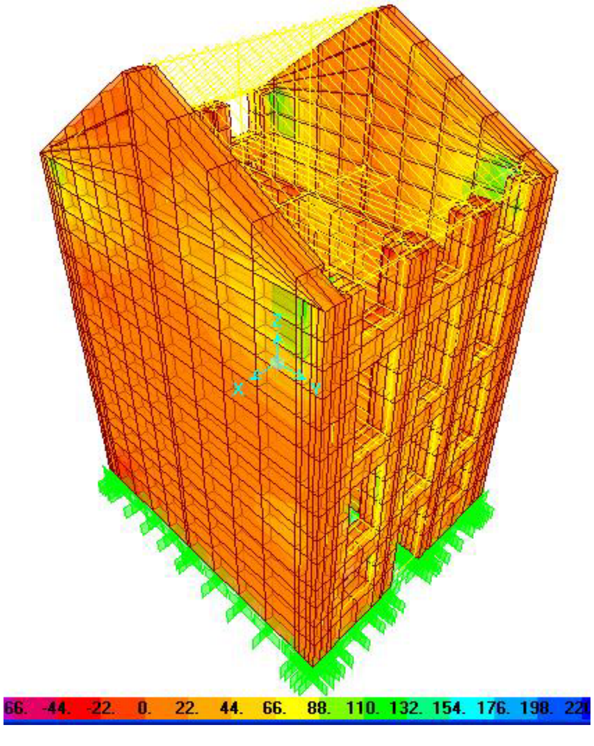


Figure 3.31f: Shear Stress (S23) Contour under Comb 3 in Modified Building

Figure 3.28: Stress contour in modified building due to Load Combination 3 (x-direction)

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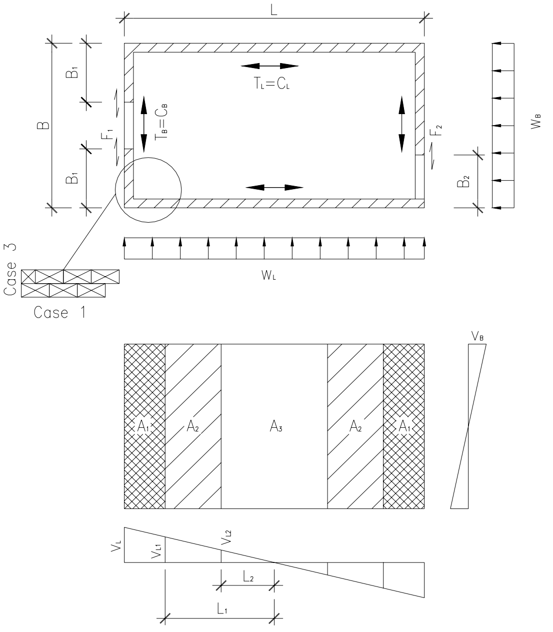
# **ANNEX1** (ref: www.Engineering-International.com)

## Diaphragm Design

### INPUT DATA



LATERAL FORCE ALONG L SIDE: $w_{L,\ WIND}$ = 256 plf,for wind
$w_{L,\ SEISMIC}$ = 224 plf,for seismic
LATERAL FORCE ALONG B SIDE: $w_{B,\ WIND}$ = 256 plf,for wind
$w_{B,\ SEISMIC}$ = 343 plf,for seismic
DIMENSIONS: L = 21.5 ft , B = 18 ft
$B_1$ = 9 ft , $B_2$ = 4.17 ft
PANEL GRADE ( 0 or 1) = 1 <= Sheathing and Single-Floor
MINIMUM NOMINAL FRAMING WITH ( 2 or 3) = 2 in
MINIMUM NOMINAL PANEL THICKNESS = 1/2 in
COMMON NAIL SIZE ( 0=6d, 1=8d, 2=10d ) 1 8d
SPECIFIC GRAVITY OF FRAMING MEMBERS 0.43

### DESIGN SUMMARY

**A1:** (2) - 0 ft x 18 ft
BLOCKED 15/32 SHEATHING WITH 8d COMMON NAILS
@ 6 in O.C. BOUNDARY / 6 in O.C. EDGES / 12"O.C. FIELD.

**A2:** (2) - 0 ft x 18 ft
BLOCKED 15/32 SHEATHING WITH 8d COMMON NAILS
@ 6" O.C. BOUNDARY & EDGES / 12"O.CFIELD.

**A3:** (1) - 21.50 ft x 18 ft
UNBLOCKED 15/32 SHEATHING WITH 8d COMMON NAILS
@ 6" O.C. ALL EDGES / 12"O.CFIELD.

THE CHORD FORCES: $T_L = C_L$ = 0.82 k , $T_B = C_B$ = 0.65 k
THE DRAG STRUT FORCES: $F_1$ = 0.00 k , $F_2$ = 1.56 k
THE MAXIMUM DIAPHRAGM DEFLECTION: $\Delta$ = 0.10 in

### ANALYSIS

THE DIAPHRAGM IS CONSIDED FLEXIBLE IF ITS MAXIMUM LATERAL DEFORMATION IS MORE THAN TWO TIMES THE AVERAGE SHEAR WALL DEFLECTION OF THE ASSOCIATED STORY. WITHOUT FURTHER CALCULATIONS, ASSUME A FLEXIBLE DIAPHRAGM HERE.

FROM THE TABLE 3.1 IN ASD MANUAL SUPP, PAGE SP-12, THE PANEL BENDING STRENGTH CAPACITY IS 390 in-lbs/ft, THAT IS THE DIAPHRAGM CAN RESISTS 65 psf GRAVITY LOADS (DL+LL) AT 2'-0" o.c. SPACING SUPPORTS.

THE MAX DIAPHTAGM DIMENSION RATIO L / B = 1.2 < 3, [satisfactory]
THE MAX SHEAR FORCE ALONG B SIDE $v_L$ = 153 plf, ( Boundary Spacing = 6 in, Edges ReqD = 6 in )
THE MAX SHEAR FORCE ALONG L SIDE $v_B$ = 144 plf, ( Required Boundary/Edges Nail Spacing for Case 3 = 6 in )
THE ALLOWABLE SHEAR FORCE FOR CASE 1 @ 6 in NAIL SPACING v1 = 221 plf, L1 = 10.8 ft
THE MAX ALLOWABLE UNBLOCKED SHEAR FORCE FOR CASE 1 v1 = 197 plf, L2 = 10.8 ft

THE SHEAR CAPACITIES PER UBC Table 23-II-H :

| Panel Grade | Common Nail | Min. Penetration (in) | Min. Thickness (in) | Member Width (in) | Blocked Nail Spacing Boundary / Other Edges | | | | Unblocked | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 6 / 6 | 4 / 6 | 2.5 / 4 | 2 / 3 | Case 1 | Others |
| Sheathing and Single-Floor | 8d | 1 1/2 | 15/32 | 2 | 221 | 295 | 435 | 492 | 197 | 148 |

Note: The indicated shear numbers have reduced by specific gravity factor per note 1 of the table.

THE CHORD FORCES: $T_L = C_L = ( w_L L^2 ) / ( 8B )$ = 0.82 k $T_B = C_B = ( w_B B^2 ) / ( 8L )$ = 0.65 k
THE DRAG STRUT FORCES: $F_1 = 0.5\ (B-2B_1)\ \mathrm{MAX}( v_{1,\ WIND}, \Omega_0 v_{1,\ SEISMIC} )$ = 0.00 k
$F_2 = B_2\ \mathrm{MAX}( v_{1,\ WIND}, \Omega_0 v_{1,\ SEISMIC} )$ = 1.56 k
$\Omega_0$ = 2.8 (UBC 1633.2.6, or IBC 1620.1.6)

THE MAXIMUM DIAPHRAGM DEFLECTION: ( Section 3.3, ASD MANUAL SUUP, Page SW-12)

$$\Delta = \Delta_{Bending} + \Delta_{Shear} + \Delta_{Nail\ \ slip} + \Delta_{Chord\ \ splice\ \ slip} = \frac{5 v_L L^3}{8EAB} + \frac{v_L L}{4Gt} + 0.188 L e_n + \frac{\Sigma(D_C x)}{2B} = 0.104 \text{ in}$$

2.634121 mm

Where:
$v_L$ = 153 plf
A = 1.50 $in^2$
t = 0.298 in,(UBC97 Page3-420)
L = 22 ft
B = 18 ft
$e_n$ = 0.018 in,(UBC97 Page3-422)
E = 2.9E+06 psi
G = 1.2E+06 psi,(UBC97 Page3-421)
$\Sigma(D_c x)$ = 0.59 in

Note: The deflection, $\Delta$, above is based on completely blocked. For unblocked diaphragm, 2.4$\Delta$ should be used.

# ANNEX 2:

Architectural Drawings of Building

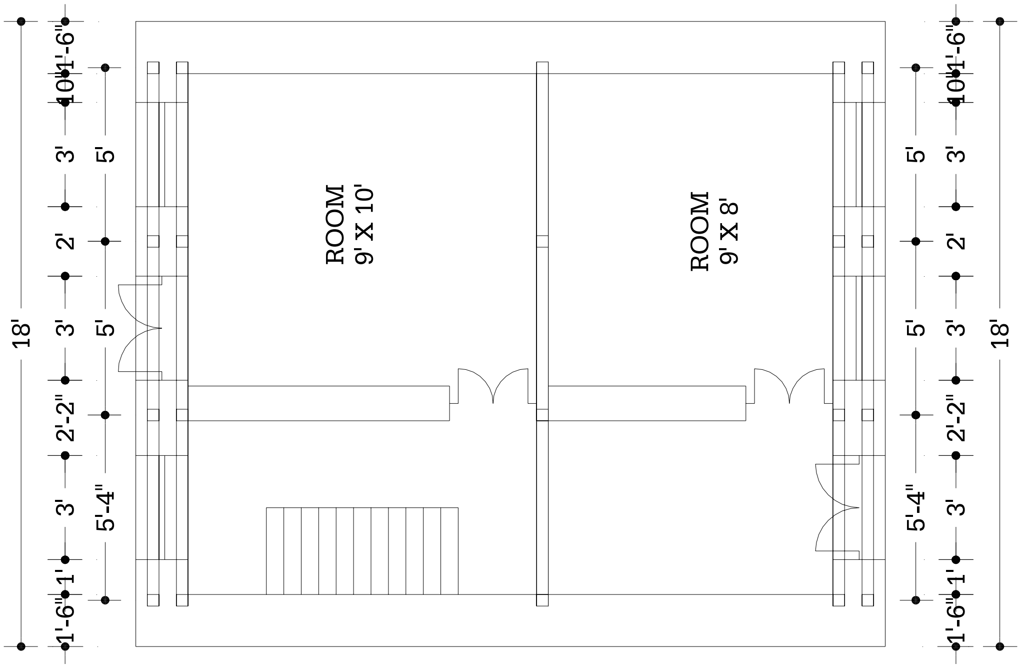


Figure A1: GROUND FLOOR PLAN

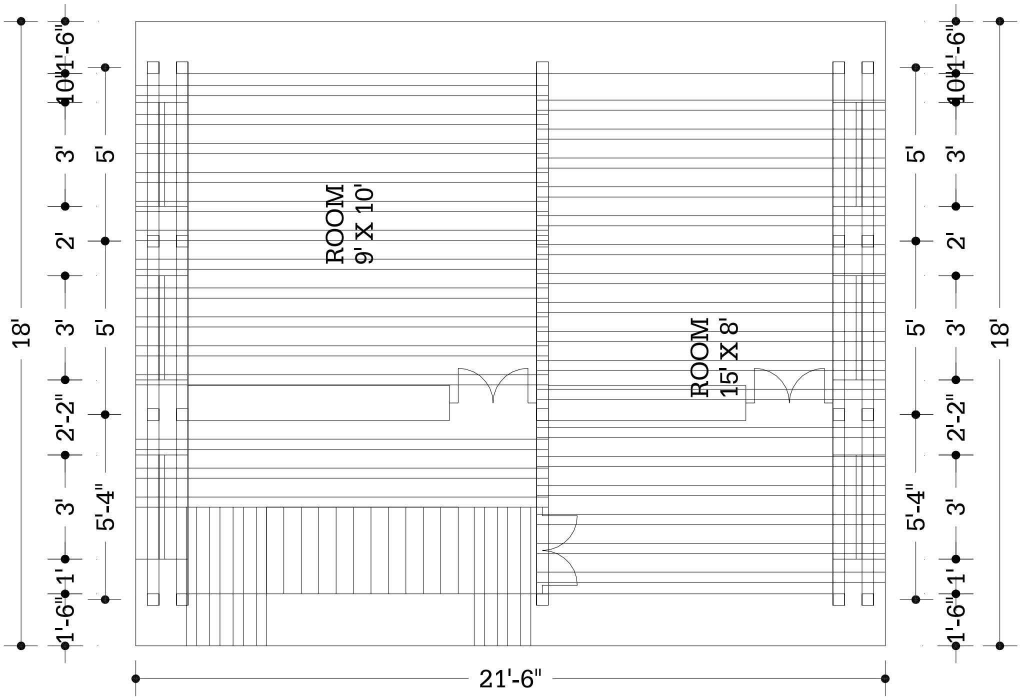


Figure A2: FIRST FLOOR PLAN

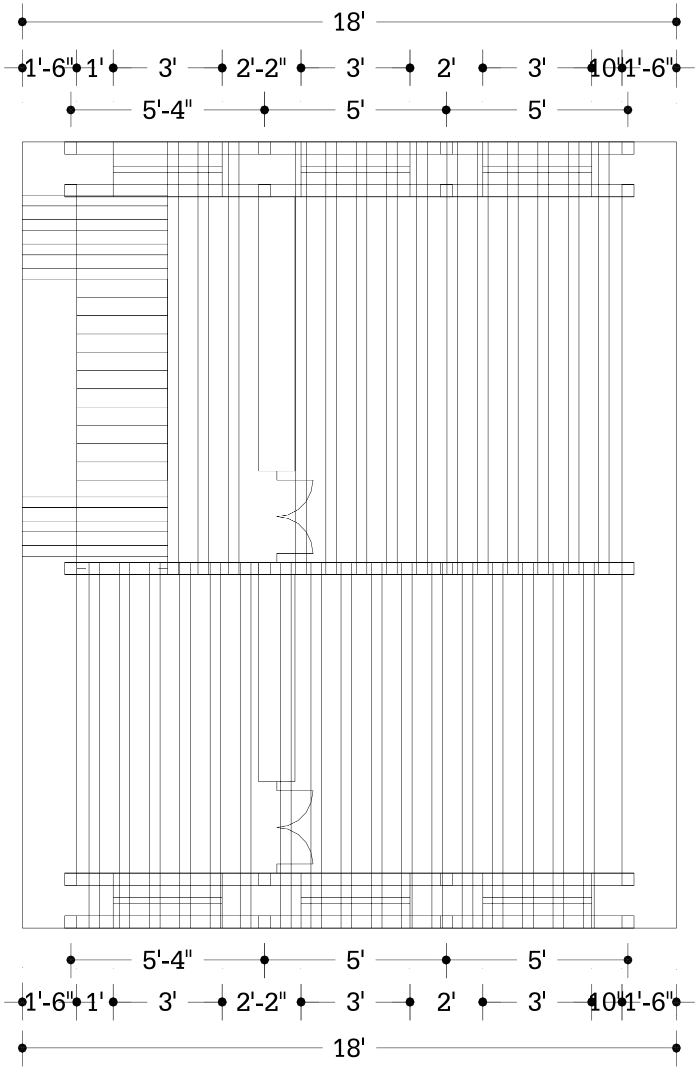


Figure A4: ATTIC FLOOR PLAN

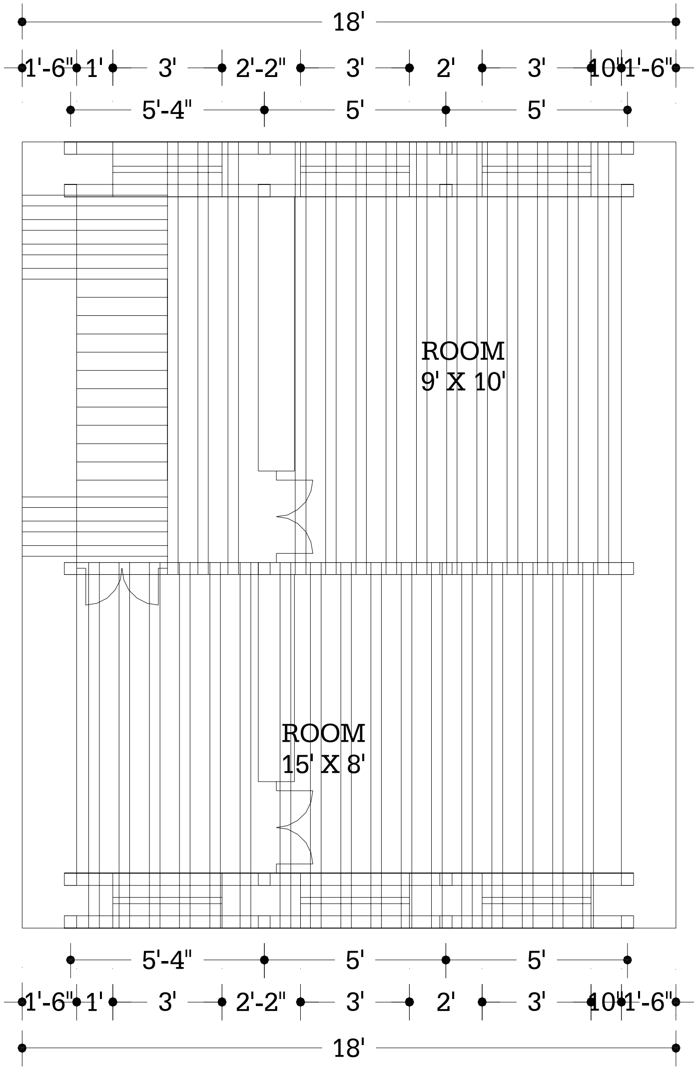


Figure A3: SECOND FLOOR PLAN

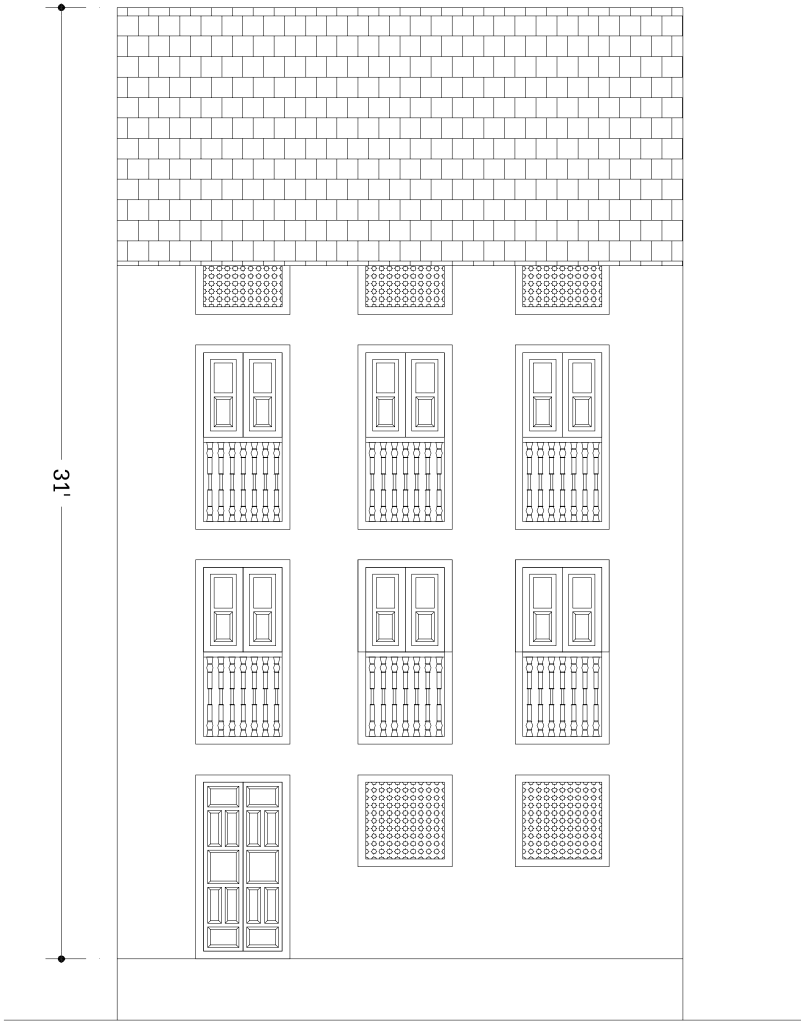


Figure A5: FRONT ELEVATION

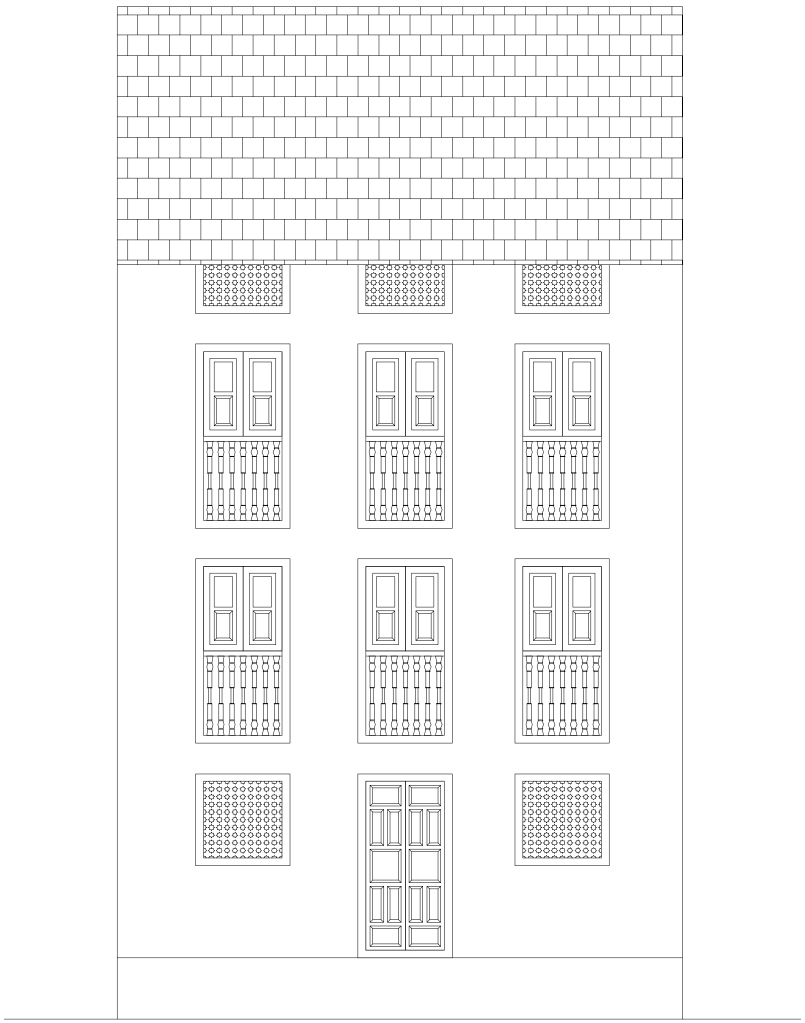


Figure A6: BACK ELEVATION

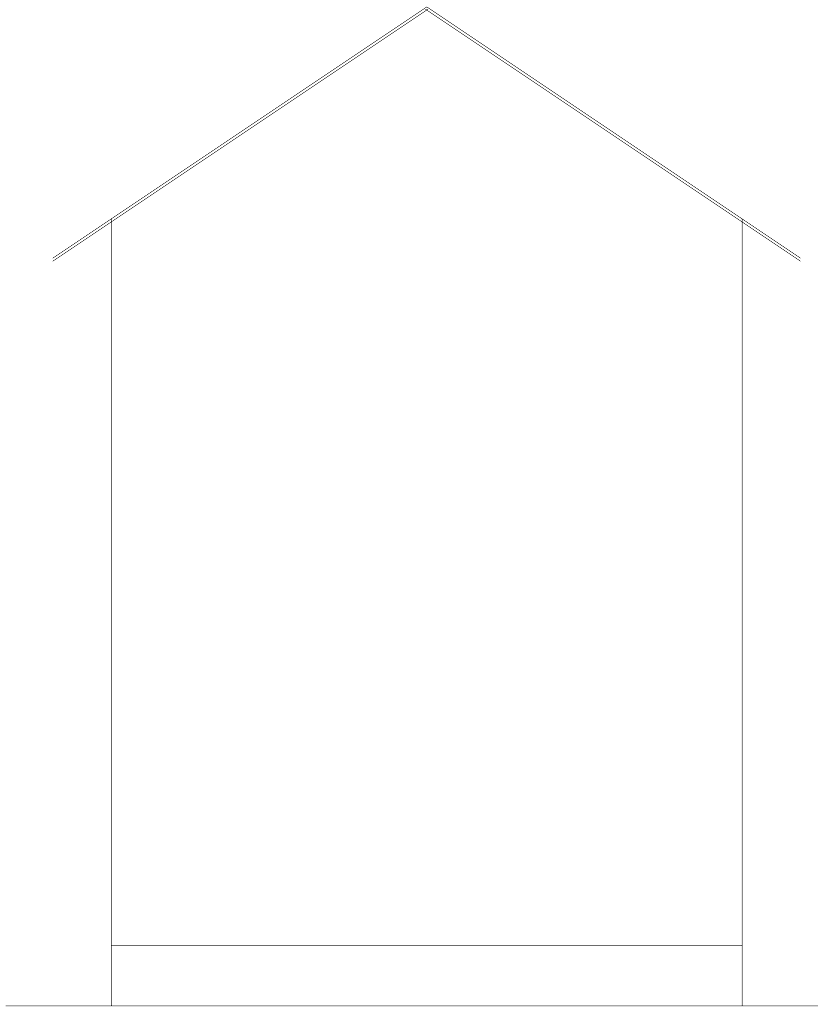


Figure A7: SIDE ELEVATION

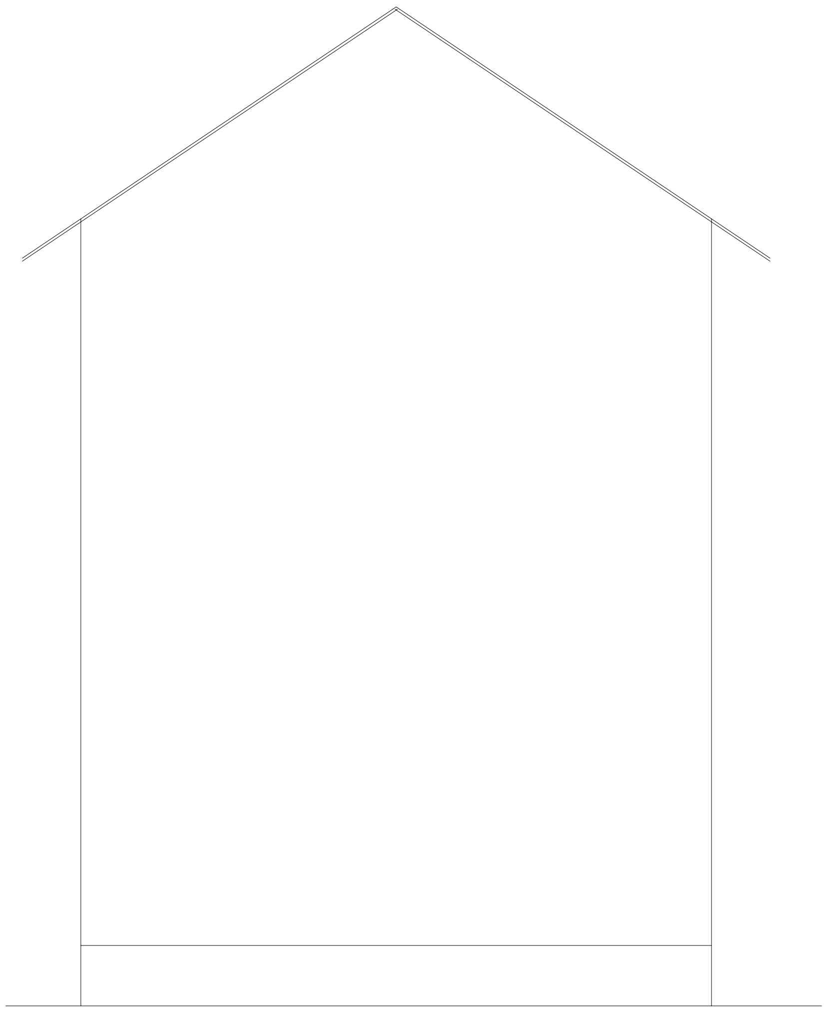


Figure A8: SIDE ELEVATION