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SEISMIC PERFORMANCE OF UNREINFORCED MASONRY BUILDING WITH FLEXIBLE FLOOR DIAPHRAGM

A Thesis submitted by

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In partial fulfillment of the requirement for the degree of

MASTER OF SCIENCE

IN

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CERTIFICATE

This is certify that the work contained in this thesis entitled **"Seismic Performance of Un-reinforced Masonry Building with Flexible Floor Diaphragm"** submitted by **Mr. Shyam Sundar Khadka** (062/MSS/r/109), in partial fulfillment of the requirement for the award of degree of **"Master of Science in Structural Engineering"** as a record of bona-fide works carried out by him under my supervision and guidance in the Institute of Engineering, Tribhuvan University, Pulchowk Campus , Lalitpur ,Nepal. The thesis fulfills 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

Un-reinforced brick masonry has been the popular mode of the construction from centuries in Nepal. Past earthquake in Nepal had shown evidence of large damages in URM building. In the mid of the 19th century, during the Rana regime, the big palaces were constructed by un-reinforced brick masonry with timber floor. These building were constructed without seismic consideration and for the residence use only. Today, most of these buildings are used by the different Governmental and non-governmental offices in their daily use. Since, many of theses buildings are over 100 years and possess the heritage value. Preservation of these buildings from future earthquake is very essential for their future use and also preserve to coming generation.

Before analyzing the actual buildings, a fictitious building having a simple plane of 6 m by 3 m is studied in detail, in order to reflect the characteristic of the unreinforced masonry (URM) structure. The finite element method is adopted for a number of parametric analyses to determine the response of the fictitious building in terms of displacement, like:(a) the effect of the wall thickness, (b) the effect of the floor rigidity (c) the effect of opening (d) the effect of number of stories and, (e) the effect of the lateral load distribution on different floor condition.

The preliminary conclusions are used for the analysis of the real building, Shital Niwas, as a case study. Due to the complexity of modeling and analysis of the whole building, only the North wing of the building is taken for the study. The performance of the building is investigated in terms of the displacement response. It is found that the outer wall of the building is collapsed due to the excessive out-of-plane deformation. The loosely connected timber floor with masonry wall, long and unsupported URM wall, which extends throughout the height and length of the building, large sizes of the room which makes the cross wall further apart are main drawback of the existing form of the building. The conclusion obtained by the analysis of the North wing of the building can be generalized with the whole building configuration.

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LIST OF ABBREVIATION

URM	Unreinforced masonry
FEM	Finite element method
1-No Floor	One storey building having No Floor
2-No Floor	Two storey building having No Floor
3-No Floor	Three storey building having No Floor
4-No Floor	Four storey building having No Floor
1-Timber Floor	One storey building having Timber Floor
2-Timber Floor	Two storey building having Timber Floor
3-Timber Floor	Three storey building having Timber Floor
4-Timber Floor	Four storey building having Timber Floor
1-Rigid Floor	One storey building having Rigid Floor
2-Rigid Floor	Two storey building having Rigid Floor
3-Rigid Floor	Three storey building having Rigid Floor
4-Rigid Floor	Four storey building having Rigid Floor

CHAPTER 1

INTRODUCTION

1.1 General

Un-reinforced brick masonry (URM) has been the principal construction materials for buildings in Nepal for a long time. Most of the traditional buildings were constructed in mud mortar, and later some of the prestigious buildings in lime surkhi mortar. Brick masonry is still one of the most popular construction materials with cement sand mortar. Due to the low tensile strength of masonry both the newer and older masonry building are highly vulnerable to the earthquake. In general, the old traditional buildings are low-rise, and most of them have been constructed for residential purposes. These traditional buildings consist of timber floors. The seismic assessment of such buildings is very essential for the conservation and for upgrading their performance for future earthquake. The present study focuses on the buildings constructed during the period of 1850 AD to 1950 AD by the ruling Rana family. These building were constructed mostly for the residential purposes. The buildings were constructed with thick URM walls, in lime mortar and timber floors. Past earthquakes in Nepal show that these building are partially collapsed or damaged due to their structural system. This shows the significant contribution of thick masonry walls against the disastrous damage past earthquake. With an aim to assess the seismic capacity of such building with timber floor diaphragm the present work is undertaken.

1.2 Need for the Study

Nepal lies in a highly seismically vulnerable region by virtue of its proximity to the young Himalayan range and the ongoing neo-tectonic activities in the region. The seismicity of the country is attributed to the location of region in the sub-duction zone of Indian and Eurasian tectonic plate. The Kathmandu Valley is a basin filled with deep soft sediments Not only large magnitude of earthquake but even small magnitudes of earthquake, frequently occur, result into high level of ground motion Kathmandu valley due to the considerable soil amplification of sediment deposit. A number of large earthquakes in the past damaged the Kathmandu valley. The earthquakes of 1833 AD having the magnitude of 7.0-7.5, 1934 AD of magnitude 8.4 and lastly the 1988 AD of magnitude 6.4 had affected the large part of valley.

Both of the earthquakes 1934 AD and 1988 AD had shown evidence of large damages in brick masonry buildings. In 1934 AD earthquake most of the un-reinforced brick and stone masonry houses including many of Rana Palaces were completely destroyed. Most of these structures were built with little or no seismic consideration. Past earthquakes in the country have shown that many such buildings are seismically vulnerable.

The study focuses on the old traditional buildings, which were constructed more than 100 years ago during ruling by Rana families in Nepal. Some of these buildings were partially destroyed in 1934 AD earthquake and some of them survived without substantial damage. These unreinforced brick masonry buildings are vulnerable to seismic hazard especially when they are un-reinforced and constructed without special consideration of seismic design. These building are extensively being used and occupied by different governmental and public offices. For example Sing Durbar, built in 1903 AD, accommodates the secretariat of Government. These buildings were built with best available material during the time of construction with best available technology. However, their seismic resistances are intrinsically limited. Since, at that time no codes were available and no advanced materials of construction like concrete and steel were available, it is assumed that those buildings were made based upon the traditional knowledge and technology. Hence, the need for the present study becomes important for the following reasons:

1. Almost all the palaces accommodate the Governmental and public offices of importance and needs to be safe against the future earthquakes.

2. Since, many of these buildings are over 100 years and possess the heritage value. Archeological conservation of the buildings is essential.

3. Seismic evaluations of such peculiar buildings have yet to be done.

Necessary strengthening or retrofitting of such buildings requires the evaluation of seismic vulnerability of the structural system and components before establishing the mode of strengthening.

Apart from these above mentioned points these buildings should be assessed to the following consideration and poor seismic performance of these buildings may be expected due to the following reasons:

i) Age and consequent degradation of structural materials leading to a decrease of local and global stiffness and strength;

ii) The high number and verity of structural changes that these structures suffered during service time, without considering the effect on the seismic performance;

1.3 Objectives of the study

The general objective is to assess the seismic vulnerability of the traditional brick masonry buildings with timber floor. The specific objectives are to determine as follows:

To determine the seismic capacity of structural wall system with and without floor rigidity

1. To find the effect of thickness of wall.
2. To find the effect of openings in wall.
3. To find the effect of the different floor rigidity.

1.4 Methodology

This study deals with the seismic assessment of the old traditional un-reinforced masonry building. To achieve the above-mentioned objectives the following modes of operation were carried out:

1. Review of the literature related to modeling the un-reinforced masonry wall and timber floor, their seismic performance in the past earthquake, mechanical properties of masonry and timber.
2. Collection of the relevant data and drawing of these historic building as possible as.
3. Modeling and analysis of fictitious building having geometric plan 3m by 6m and storey height of 3m is carrying out considering the floor rigidity, thickness of wall, and opening in the wall.
4. Discussion and concluding the remarks from the above study towards the floor rigidity, thickness of wall, and opening in the wall.

5. Modeling and analysis of the typical palace considering the effect of above mentioned study and made the conclusion towards the upgrading the performance of the building.

6. Modeling and analysis of structure is done by commercial program SAP2000. Eight-noded solid elements are used for the thick wall.

7. The connection between timber joist and thick wall consider as simply supported. Frame element is used for timber joist.

8. Analysis of the structure shall be carried out for various loading condition such as dead load, imposed load, earthquake using Seismic Coefficient method and Response Spectrum method as per IS 1983:2002(part I).

1.5 Organization of chapters

The study is divided into six chapters. In chapter 2, the various literatures related to modeling of the un-reinforced masonry wall and timber floor, methods and types of modeling, mechanical properties of masonry and timber, failures of URM building in different earthquake are discussed.Chapter 3 describes the description of masonry buildings in Nepal, in general. It mainly focuses on the structural system with thick walls different floor diaphragm of the building, connection between the wall and floor system. It also describes the current status of the building.

In chapter 4, the finite element modeling of the fictitious building having plan dimension of 3m by 6m in plan and storey height of 3m is present. The modeling regarding the thickness of wall, rigidity of the floor, opening in wall are studied and results are discussed in detail. Analysis is carried out by using the seismic coefficient method.

Chapter 5 presents the modeling and analysis of the typical palace considering the study of fictitious building in chapter 4 as a case study. And discussion and conclusion about the study also presented in this chapter.

Chapter 6 presents the conclusion of the study. Assessment of old traditional building is very essential for the retrofitting and strengthening for future earthquake. The floor rigidity is more essential for the un-reinforced masonry building. This chapter also suggest for the further study in order to assess the old traditional building.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Extensive researches have been carried out to study the seismic behavior of the un-reinforced masonry (URM) buildings. The researches on URM consist of study on materials, modeling and analysis including full-scale and reduced scale model testing structures. Entire masonry building with wood diaphragms has also been conducted and computer simulations to the old traditional URM buildings are also conducted. This chapter presents the summary of the available literature on modeling the structure, the performance of the URM building on earthquake, failure mechanism, and material properties of the URM structure having wood diaphragms.

2.2 Literature related with modeling of URM building

This section describes the method of modeling of components of URM buildings, namely, timber floors.

Page (1978) developed finite element analysis for masonry subjected to in-plane loading. In his model, the in-plane behavior of masonry is modeled using a continuum of plane stress elements with superimposed linkage element simulating the mortar joints. Bricks are modeled using conventional eight- parameter rectangular plane stress elements with four internal degree of freedom. Joint elements are modeled as linkage elements with limited tensile strength, high compressive strength (with non-linear deformation characteristics) and variable shear- strength depending upon the degree of compression present. The non-linear response of masonry in the model is, therefore, produced by both the inelastic mortar properties and progressive joint failure.

Lotfi and Shing (1994) studied that the behavior of un-reinforced masonry and compared it with the behavior of concrete. Although intact concrete may be assumed homogenous and isotropic, the presence of mortar joints makes un-reinforced masonry composite, both heterogeneous and orthotropic. In the finite-element analysis of un-reinforced masonry structures, the effect of mortar joints as the major source of weakness and material non-linearity has been accounted for with different level of refinement. The least-refined approach is to consider a homogenous material law for a masonry composite, which takes into account the effect of mortar joints in an average sense. Although the approach with this level of refinement is the most suitable for the analysis of large masonry structures, it is not adequate for detailed stress analysis and for capturing the various failure mechanisms of masonry assemblages.In a less-refine approach, masonry units are modeled with continuum elements, while mortar joints are modeled by means of interface elements. Obviously, the approach with this level of refinement is computationally intensive for the analysis of large masonry structures.

Wang et al. (1997) discussed about the microscopic and macroscopic masonry model and the choice of the best model in terms of computational effort and for the best result. The authors found that analytical results based on the microscopic model were not satisfactory. In the author's opinion, this model was unable to simulate the crack propagation between elements after debounding. The discontinuity of deformation between a deboned mortar element and a unit element caused numerical solution problems. For various reasons, a macroscopic model was chosen in which the wall was descretized into finite elements with no particular attention given to the position of mortar layers. Debounding would most probably occur within an element that could be modeled as a crack and allowed to propagate as a crack. Properties of the masonry assemblage as a whole, that is, the properties of masonry prism, could thus be used instead of different properties of each individual material component.

Mehrabi and Shing (1997) introduced a dilatent interface constitutive model to simulate the behavior of mortar joints between masonry units and a smeared crack finite element formulation had been used to model masonry units. However, there are still some aspects of the physical behavior of interface, which have not been considered properly in most existing models. These include the compressive hardening behavior of interfaces, the reversal of shear dilatancy in the case of cyclic loading, and the normal contraction of an interface under shear sliding.

Amadei et al. (1989) have shown in their experiments conducted on the clay brick masonry that relative shear displacement in a mortar joint can result in a normal expansion *called* shear dilatancy. They have also shown that this dilatancy irreversible; i.e. changing in the direction of the relative shear displacement reverses the normal dilatation.Their test results have further indicated that relative shear displacement under a high compressive stress can result in the crushing and wearing of joint materials. This can lead to a normal contraction of the joint, the level of which depends on the quality of the material and the level of the compressive stress.

Gambarotta and Lagomarsino (1997) developed a finite element approach for masonry where brick units and mortar joints are considered separately. A model had been defined in which the brick units are modeled through four or eight isoparametric elements with four nodes while the mortar joints are modeled by interface element with four nodes. This model is too burdensome in analyzing full-scale masonry walls.

Papa (2001) proposed two models, based on the damage mechanics and suitability for the analysis of masonry structures subjected to plane stress condition: the heterogeneous models and the homogenous models. The heterogeneous models, analyze the masonry walls descretising bricks and mortar separately through finite element and/or interface elements, masonry is considered as a homogenous, orthotropic material. A limited domain, endowed with a suitable hardening/softening law, is defined according to piecewise linear function; material damage is taken into account as a function of inelastic deformation. In homogenous model, masonry is assumed as a continuum material: its properties are either obtained by introducing suitable constitutive laws able to represent the directional behavior of masonry, or by starting from the characteristics of the masonry components, that is mortar and bricks, and using a suitable homogenization procedure. In this way, real masonry buildings can be studied with reasonable computational effort. Moreover, considering the masonry as a composite material, it is possible to detect the occurrence of failure either in the bricks or in the mortar joints, although these components are not descretized separately. In this model the overall mechanical proprieties of masonry are determined based on the properties of the component, for which a unilateral damage model is assumed. The numerical results given by these two models appear to be good in good agreement with the experimental data related to masonry panels and buildings.

2.3 Literature related with the timber floor

There are few literature towards the modeling the wood diaphragm and less experimental investigation. However, there are many experimental investigations on wood diaphragms having wooden shear wall.

Tenga-Colunga and Abrams (1992) suggested that the discrete MDOF dynamic model can be understood as an equivalent system of condensed beams (representing the cantilever walls) linked by elastic springs (representing the flexible diaphragms). Response is calculated at the translation degree of freedom of these elements. The elastic discrete modeling has the capability to include the flexibility of the diaphragms, rotation of the walls and soil-structure, interaction effects through generalized translation and rotational springs at the base. However, the modeling does not consider the dynamic constraints imposed by the walls running in the perpendicular direction. Recorded dynamic responses of the subject building were represented reasonably well with the discrete model (wave forms, natural period estimates, frequency contents and peak response). Variation between measured and computed response were obtained within ranges of variation of material properties and the soil. In this work a case study is presented describing seismic response of an instrumented grouted brick wall structure subjected to the Loma Prieta Earthquake. The subject structure is a two- story office building located at Palo Alto, California. Recorded peak ground acceleration was as high as 0.21g and peak roof acceleration as high as 0.53g. Considerable amplification of the peak acceleration between the ground and the roof were observed. The building withstood the Loma Prieta earthquake with little damage. This research work has prompted out the necessity to improve the understanding of the behavior to structural system with flexible diaphragms (plywood, flexible reinforced concrete etc) to define: damping characteristics, the contribution of joists and sheathing to the lateral stiffness, their non-linear behavior. The knowledge gained through this research has been possible to the availability of instrumented data from a grouted brick wall building during Loma Prieta Earthquake. The discrete model is used in this study proved again to be a helpful analytical tool for the evaluation of low-rise masonry structures with flexible diaphragms.

Tenga-Colunga and Abrams (1996) found that the **s**tructure with flexible floor diaphragms system behaved differently under dynamic lateral loading than the structures with rigid diaphragms. Torsional effect can be reduced considerably as the flexibility of the diaphragms is increased. The fundamental period of building system with flexible diaphragms is consistently longer than values estimated with simplified methods given by current seismic codes.

2.4 Literature related with material Properties

Makarios and Demosthenous (2006) studied the seismic response of the traditional building of Lefkas Island, Greece. The mechanical properties of the masonry and timber they used in their are presented in Table 2.1.

2.5 Literature related with failure mode of the URM building

Manifas (1973) describes the catastrophic out-of-plane failures of adobe and brick masonry buildings during the earthquake of Ghir in Iran of 1972.

Bruneau (1994) describes the possible various failure models of URM buildings. Commonly failure modes, based largely in part on damage observed after the earthquake. The failure can be regrouped in the following categories: lack of anchorage, anchor failure, in-plane failure, out-of –plane failure, combined in-plane and out-of-plane failure and diaphragm related failures.

Due to the absence of anchorage of the floor and roof to the URM walls the exterior wall behaves as cantilevers over the total height of the building. The risk of the wall out-of-plane failure due to excessive flexural stresses at the base of the wall obviously increases with its height, but, more importantly, global structural failure can occur by the slippage of the joists and beams from their supports. (Figure 2.1)In- plane shear failures are expressed by doubly –diagonal (X) shear cracking (Figure 2.2). Due to the presence of numerous openings in masonry facades, spandrels and the short pier between those spandrels may also fail in shear. Flexural failure of those structural elements is also possible, particularly for slender URM element transforms it into rigid body no further lateral-load resisting capacity, unless gravity forces can provide a stabilized effect.Un-reinforced masonry buildings are most vulnerable to flexural out-of-plane failures (Figure 2.3). In-plane failure does not endanger the gravity-load –carrying capabilities of a wall, the unstable and explosive out-of-plane failure will. Parapet failures fall in this category (Figure 2. 4). These nonstructural URM elements behave, if unrestrained, as cantilevers of a walls extending beyond the roof line;

located at the top of the buildings, they are subjected to the greatest amplification of the ground motions, and consequently prone to flexural failures.

Among these, he emphasis that URM buildings are most vulnerable to flexural out-of-plane failure. As in-plane failure may not right away lead to collapse since the load carrying capacity of the wall is not completely lost by diagonal cracking. However, out-of-plane failure leads to unstable and explosive collapse. He also points out that masonry construction of poor quality often shows total failure while monumental / Institutional masonry buildings of high quality often perform quite well.

Magenes and Calvi (1997) describe the principal failure mechanism of masonry piers subjected to seismic action. These are:

Rocking failure: As horizontal load and displacement demand increase, bed joint crack in tension and shear is carried by the compressed masonry; final failure is obtained by overturning of the wall and simultaneous crushing of the compressed corner as shown figure 2.7 (a).

Shear failure: Peak resistance is governed by the formation and development of inclined diagonal cracks, which may follow the path of bed-and head-joint or may go through the bricks, depending on the relative strength of mortar joints, brick-mortar interface, and bricks as shown figure 2.7 (b).

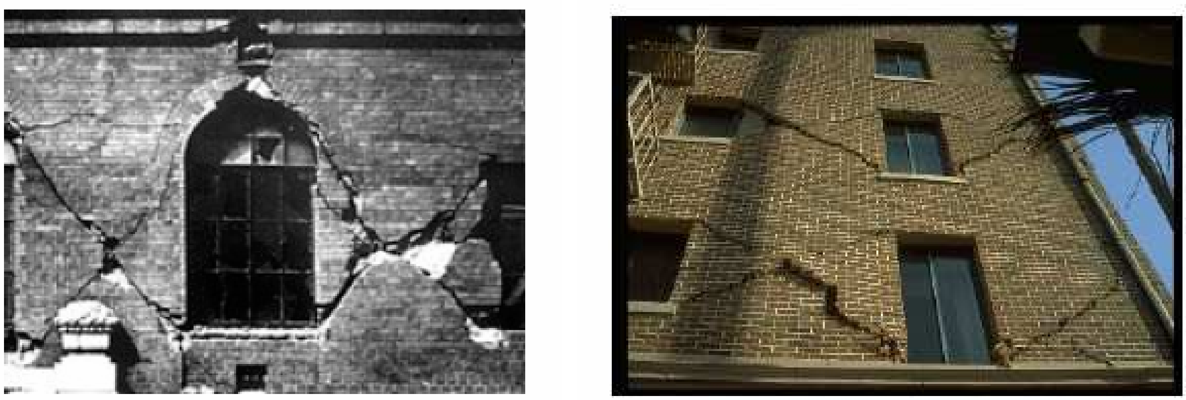
Sliding failure: Due to the formation of tensile horizontal cracks in the bed joints, subjected to reversed seismic action, potential sliding planes can form along the cracked bed-joints; this failure mode is possible for low levels of vertical load and /or low friction coefficients as shown figure 2.7 (c).

Table 2.1: Mechanical properties of the material (mean values)

Material	Youngs modulus(kN/m2)	Poisson Ratio	Mass density (t/m3)	Compressive strength (MPa)	Tensile strength (MPa)
Wood	9000 000	0.30	0.5	24	46.5
Brick masonry	1708 000	0.15	2.1	4	0.4

Compton Junior High School, Long Beach Earthquake, 1933

Figure2.1**:** Totally collapsed one story building made of brick walls and wooden roof *(Courtesy of EERC Library)*



Long Beach Earthquake, 1933 Santa Monica, Northridge Earthquake, 1994

Figure 2.2: In-plane shear failure*(Courtesy of EERC Library)*

North of Coalinga, Coalinga Earthquake, 1983 Downtown Coalinga, Coalinga Earthquake, 1983

Figure2.3*:* Out-of- plane failure

Watsonville, Loma Prieta Earthquake, 1989

Hollister Area, Loma Prieta Earthquake, 1989

Figure 2.4: Parapet failure *(Courtesy of EERC Library)*

Figure 2.5 Combine out-of- plane failure

Figure 2.6 Separation of wall corner

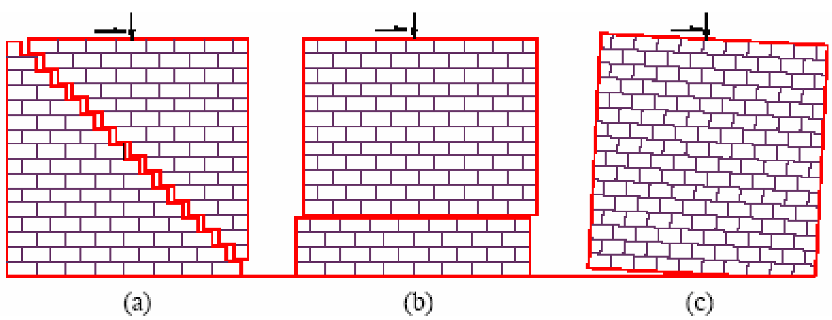


Figure2.7 In- plane failure modes of a laterally loaded URM wall:

a) Shear failure b) Sliding failure c) Rocking failure

(Source: Seismic In-Plane Behavior of URM Walls with Upgraded with composites- PhD Thesis 2004 EPFL by Mohamed ELGAWADY)

CHAPTER 3

MASONRY BUILDING IN NEPAL

3.1 General

Brick masonry is one of the oldest types of construction materials for buildings. In the past buildings in Nepal used to be built in sun-dried clay brick, mud mortar with timber floor. Later brunt clay brick, mud mortar with timber floor became popular in construction. Even after the advent of new construction materials like concrete and steel in construction brick masonry still continue to be the main type of construction materials for the building, either in the form of load bearing wall or in-fill wall. Construction of buildings in brick masonry has been carried out for a long time. Ancient buildings used to be in adobe, burnt clay bricks in mud mortar and in latter stage with enhanced mortar like lime surkhi mortar or cement sand mortar. A large building stock existing in Nepal at present comprises of un-reinforced brick masonry based system. It has been well recognized that the brick masonry buildings are highly vulnerable to earthquake action.

3.2 Masonry building of Rana Period (1850-1950 AD)

Nepal experienced a new scenario in construction of buildings during the middle of the Nineteenth century. After returning from the Europe tour the first Rana Prime Minister Janga Bhaduar Rana built the palaces for him with inspiration of the European style. Construction of such neo-classical building introduced 1850 AD continued to take place during over 100 years of ruling by Rana family. These palaces of neo-classical type are scattered all over the Kathmandu valley. The best quality of material and new technology at that time of construction were used in the construction of the palaces. These were built in best quality of bricks available and in lime surkhi mortar. The present study is confined to this type of buildings in Nepal. It is to be noted that these places were used for residential purpose in general, and the construction was different from that of other civil residential buildings in terms of quality and grandiose. At present, almost all of these places are being used by governmental and public enterprises. The current usages of some of the palaces in the

Kathmandu valley are presented in table 3.1. These palaces are usually three storied and cover large area with huge compounds. In general, the palaces are built in courtyard system, sometimes with a number of courtyards. The large wings of the buildings are of gallery type with long corridors with spacious rooms of different sizes. Singha durbar one of the largest palaces, for instance, covers a large area with many courtyards and a large compound. All important government offices include Ministries of Government and National Planning Commission are accommodated in the buildings. In 1970 AD it was badly affected by fire and renovated using traditional technology. Another palace, Sital Niwas, which at present accommodates the Ministry of Foreign and Affairs, was re-built after the damage in 1934 AD earthquake. The original old Sital Niwas was built in 1923 AD and during the devasting earthquake in 1934 AD it was extensively damaged and again reconstructed on the same ground immediately after earthquake. The present condition of Sital Niwas palace is shown in figure 3.1. Hasty re-construction of palaces after the devasting earthquake with no seismic consideration these structures more vulnerable, indicating further need of seismic risk assessment. The building plan of the north wing of Sital Niwas palace presented in figure 3.2 to illustrate the typical structural plan of the palaces.

3.3 Structural System

The information on the structural system and components of the buildings are essential for evaluating their seismic resistance. The buildings of discussion are court-yard type, and contain one or more than one court-yards. Usually they are three to four stories with large number of opening in their façade and have slope roofs. The buildings constitutes of long walls with rectangular or square shaped court-yards. The main structural system for lateral load resisting is thick un-reinforced masonry wall with about one meter thickness in the ground floor, reduced thickness in the upper floors but with 0.6 meter as a minimum thickness. The walls are made of high class burnt clay brick with good quality of lime surkhi mortar. Thick timber floors are used with timber joists provided normal to the front façade wall, and with planks. Depending upon the spans of the room, sometimes rolled steel I- sections are employed to support the joists of the flooring with decorative steel plates. There are clusters of openings in the wall for adequate lightning and also for architectural aesthetic. Sizes of opening are same in all floors and placed in symmetrical position. Strip brick masonry foundations of the buildings are rigid. The plinth level is considerably high above the ground level with a provision for ventilation in the suspended timber ground floor. All the palaces have sloped roof normally covered with GI sheets or clay tiles with timber truss and rafters.

3.4 Structural components of the building

This section describes the main structural components of the buildings are briefly described in the following paragraphs.

3.4.1 Brick masonry wall

The main structural system for lateral load resisting system is thick brick masonry wall. Due to massive wall the inertia forces is increase, which is adverse for earthquake in the meantime it increases the in-plane stiffness of the building. These walls are constructed in Wythe system for the purpose of insulation. The overall thickness of the wall is 0.75 m. There is a long wall in corridor for example 47.25 m in length as shown in figure 3.2 without any cross wall and have a large number of openings. These walls are very prone to out-of –plane failure. The clay brunt brick with Lime surkhi mortar, with powder of burnt clay brick, typically named as “Bajra”, is used for masonry. The “Bajra” is used for plastering the wall both inside and outside of the wall.

3.4.2 Timber Floor and Roof

The floor system consists of timber joist and planks and considered as flexible diaphragms. Despite the heavy dead load the timber floor does not provide any kind of seismic-resistant function. Usually the connection between the timber elements of the floor and masonry walls is supposed to done with iron or wood element. But there is no such evidenced to see that there is connection. So, this type of floor does not provide floor rigidity at their level and lateral load capacity is limited. Most of the palaces have the slope roof. Roofs are made with timber truss and clay tile or GI sheet. There is an attic floor in top of the building, which is covered by the roof, and as the same case of floor the roof also does not provide the rigidity at their level.

3.4.3 Openings

There are large numbers of opening having same sizes for doors and windows. The openings are in the same horizontal and vertical alignment throughout the building. Most of the opening includes doors and windows are arch type. And, a number of arch opening are in corridor of the building. Due to the presence of opening the seismic resistance capacity of the masonry wall is reduced to in plane loading despite the thick masonry wall. When subjected to the seismic loading, stress concentration takes places in the opening zones, which may result in unexpected cracking of masonry wall.

3.4.5. Foundation

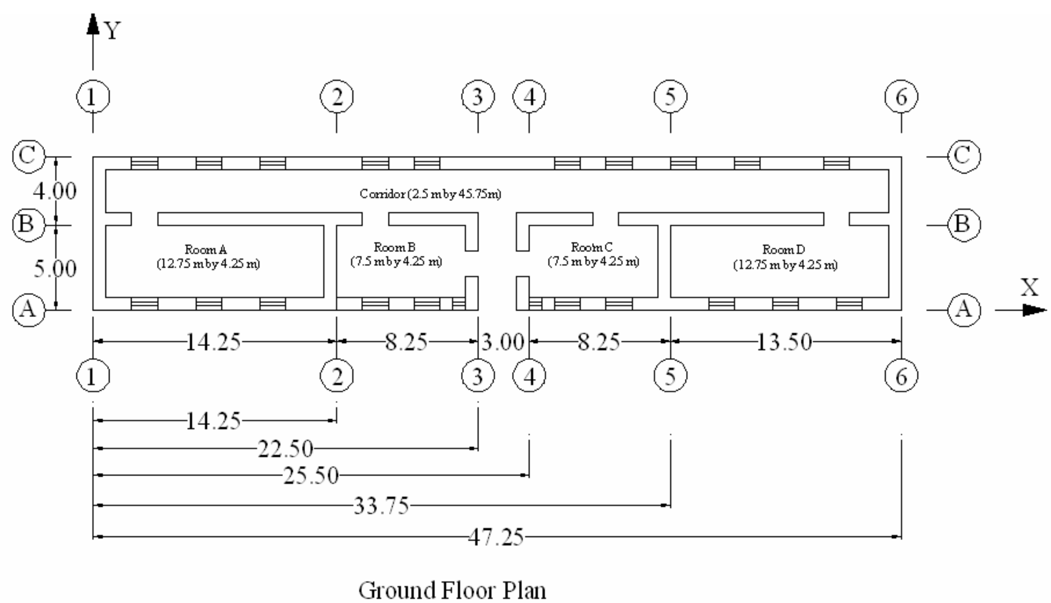
Generally, for old traditional building the spread foundation is commonly adopted. Foundation may seprate to each other. The depth of the foundation is varied from 1 to 4 meters. The base of foundation is steeped up to the plinth level. Since, the soil of the Kathmandu valley is alluvial, prone to liquefaction, and tends to amplify earthquake forces. These considerations were no considered during the design of the foundation of the traditional buildings. The past earthquake shows the evidence of liquefaction in the Kathmandu valley and it might be cause the foundation failure. For the analysis, the foundation of the buildings assumed as a rigid foundation.

Table 3.1: Some palaces buildings and their current usage

SN	Palace	Construction Date (AD)	Current Usage
1	Bagh Durbar	1805	Kathmandu metropolitan city
2	Narayanhiti Durbar	1847	Royal Palace
3	Singha Mahal	1847	Nepal Rastra Bank
4	Lal Durbar	1890	Hotel Yak and Yeti
5	Agni Bhawan	1894	Hotel Shankar
6	Bahadur Bhawan	1889	Election committee
7	Kaisher Mahal	1895	Ministry of Education
8	SinghaDurbar	1903	Secretariats
9	Shree Mahal	1920	Ministry of local development
10	Sital Niwas	1934	Ministry of Foreign Affairs
11	Ananda Niketan	1890	Institute of Engineering (TU)

Figure 3.1: The Shital Niwas

(Source:http://www.nepalmountainnews.com/upimages/subfolders/others/ministryforeign.jpg)



(All dimensions are in meter)

Figure 3.2: Floor Plan, North Wing of Shital Niwas

CHAPTER 4

MODELING OF UN-REINFORCED MASONRY BUILDING

4.1 General

The study of un-reinforced masonry (URM) building for seismic resistance largely depends upon the modeling and analysis of the structure. This chapter describes the modeling techniques of URM buildings with a special reference with thick walls. With an aim to study the behavior of URM building in earthquake action along with gravity load, a hypothetical simple one-story building is modeled and analyzed. The building is investigated for its performance in-terms of displacement for different wall thickness, different floor rigidities, location and amount of openings. In order to evaluate the effect of height of the building, the hypothetical building is extended to two, three and four stories. The URM brick walls are modeled as eight nodded solid elements considering the brick masonry as homogenous and isotropic material.

4.2 Fictitious simple URM building

At the out-set of the evaluation of seismic performance of the old traditional building, a simple fictitious URM building having simple plan is considered. The building is of dimension 3m by 6m in plan having a story height of 3m. The building is assumed to be situated in Kathmandu. The fictitious building is analyzed for gravity and earthquake loads in orthogonal directions considering different floor rigidities, wall thickness location and areas of openings. The same building is further investigated with extension of one more storey with variation of the parameters. The typical plan of the fictitious building analyzed is presented in figure 4.1.

4.3 Modeling of brick masonry

The selection of appropriate and practicable “Numerical Model” and feasible “Finite Element” depend on the computational effort required for handling and processing the model and verification of the desired results. It is well recognized that homogenous models of masonry with solid element provide sufficient accuracy with reasonable computational effort. On the other hand, heterogeneous models with shell or solid elements render the better results than that of the former models but at the cost of high computational effort, especially when a real wall or building must be analyzed.

Giordano, et al. (2002) had suggested that the un-reinforced masonry structures are made of blocks/bricks connected by the mortar joint. Due to this intrinsic geometrical complexity, which is obviously reflected in the computational effort needed, it is necessary to assume a properly homogenized material and perform the analysis through the finite element method (FEM), when the global behavior of an entire structure is investigated. On the contrary, when a single structure element is being studied, the actual distribution of blocks and joints can be accounted for.

Based on the conclusions of many researchers like that of Giordano, et al. (2002), a homogenous solid element model is employed for modeling brick masonry wall. The solid element of SAP2000 is an eight-nodded element for modeling three dimensional structures and solids. The solid element local axes, coordinate system and stress output sign convention are shown in fig.4.2. The aspect ratio for a solid model should be one for better results and not greater than ten. So, the results depend on the refinement of meshes.

Regarding the sizes of elements, Kappos,. et al. (2002) had studied both 2-D and 3-D modeling for a building and also the fine and coarse meshes. It was found that in 2-D analysis reasonable estimates of displacements could be obtained with rather coarse meshes, the case is more in the 3-D case, since the computational cost involved in refining the mesh is not justified by the slightly improve accuracy in the result. They also suggest that the uncertainties associated with the seismic input and the properties of masonry which are layered, nonlinear, heterogeneous material are expressed in terms of a constant young’s modulus and a shear modulus, any attempt to over refine the finite element model appears as a futile exercise.

4.4 Modeling of floors

Horizontal stiffness of building largely depends on the floor rigidity. In general, the modeling of timber floor depends upon its in-plane rigidity and its connection with walls. In older buildings the timber joist simply rest on the wall. So, timber floor cannot be considered as a rigid floor diaphragm for modeling. In the case of reinforced concrete floor, it acts as a rigid diaphragm due to its high in-plane stiffness compared to that of wall. For modeling the reinforced concrete floor diaphragm constraint is applied at their level. For the considered hypothetical building, three different cases were considered. These are:

i) Considering no in-plane stiffness of floor

ii) Considering finite in-plane stiffness of the timber floor with simply supported connection with walls, and

iii) Considering infinite in-plane stiffness, that is, rigid diaphragm of the floor.

In the first case only the brick walls, in-plane and out of plane resistance contributed in undertaking the horizontal loads. In the second case the timber joists of floors are modeled as the frame elements with free rotation at the connections with the walls, simulating the flexible diaphragms. In the third case, rigid diaphragm of the floor is considered, as is exhibited by concrete floors. These three cases were considered in the analysis to evaluate the effect of the floor system in the structure.

4.5 Openings in the wall

The seismic performance of the un-reinforced masonry building greatly depends on the opening in the wall. In this research work, the effect of the opening is limited only for one story building with reinforced concrete floor and timber floor diaphragm for all thicknesses of wall. The effect of opening is studied separately in both longitudinal (along x-axis) and transverse (along y-axis) wall of the Fictitious building as shown in the figure 4.1. Two different cases are studied in the fictitious building for opening. These are:

1. Centrally located opening with varying the horizontal width in the transverse wall of the fictitious building, shown in figure 4.1.

2. Centrally located opening with varying the vertical height of in the longitudinal wall of the fictitious building, shown in figure 4.1.

Deep Beam Theory

Manual method for calculation of deflection

For no opening in wall

In order to verify the results obtained from finite element analysis with the manual method, here is popular derivation, for the total deflection of prismatic cantilever shear wall, as shown in figure 4.7 ,that is, without opening and with constant cross section is given by the sum of the deformation of both flexural and shear.

Therefore, the total deflection is given by;

$$\Delta_{total} = \Delta_{flexural} + \Delta_{shear}$$

$$= PH^3/3EI + K *\{PH/GA\}$$

$$= \{P/\{E*b\}\}* [4*(H/L)^3+3*(H/L)] \qquad (4.5.1)$$

Where,P= Lateral force;

H= Height of Wall;
E= Modulus of Elasticity;
K= Shape factor =1.2 for rectangular section
A= Cross section area,
I= Moment of inertia = bL^3 /12 for a rectangular wall,
μ = Poisson's ratio =0.15 for masonry

The flexure term considers the wall as a vertical cantilever with moment of inertia *I*. The shear term contains the shape factor which accounts for the distribution of shear stresses across the section and the shear area *A*. The material properties *E* and *G*, respectively, which are related by

$$G= E/2(1+ \mu) \qquad (4.5.2)$$

For opening in wall

If the shear wall has openings with dimensions that are significant compared to the dimensions of the wall the shear wall no longer behaves as a deep beam.

Brandow et al. (1997); Lindeburg and Baradar (2001) suggested a simplified procedure to determine the lateral stiffness of shear walls with opening as presented in Figure 4.9. At the first, the deflection Δ_{total} of the solid cantilever wall is calculated by ignoring openings illustrated all openings. Next a strip is considered whose length equals that of the wall and whose height is that of the tallest opening. The strip displacement $\Delta_{solid\ strip}$ is calculated and subtracted from that of the solid wall. As far as the support conditions for the strip are concerned, there is conflicting information in the literature.

Brandow et al. (1997) suggested considering the strip fixed–fixed while Lindeburg and Baradar (2001) recommend cantilever action. But the stiffness of all piers contained in the openings is summed up assuming fixed–fixed conditions. The final displacement of the wall with openings is

$$\Delta_{wall} = \Delta_{solid\ wall} - \Delta_{solid\ strip} + \Delta_{piers} \qquad (4.5.3)$$

Such that the stiffness becomes

$K_{\text{wall}} = 1/ \Delta_{\text{wall}}$ (4.5.4)

Note that for fixed–fixed conditions, the shear deformation is identical to that in Equation 4.5.1 and the flexure deformation is four times smaller than that for a cantilever such that

Δ total = Δ flex + Δ shear = $\{P/ \{E*b\}\}* [(H/L)^3+3*(H/L)]$ (4.5.5)

4.6 Analysis of structure:

The building structure, firstly of single storey and later of two, three and four stories were analyzed for gravity load and earthquake loads. The earthquake load analysis is carried out by seismic coefficient method as recommended in IS 1893 (Part I)-2002.

Seismic Coefficient Method

In this method the dynamic seismic force is transmitted into an equivalent static force on the building and is distributed throughout the height of the building to each of the resisting elements. The static seismic force is assumed to be an external base shear force, V, which is applied to the structure.

In this research the seismic analysis has been done by seismic coefficient method as per IS 1893(part I): 2002

The steps to be followed in the seismic coefficient method are as follows:-

Design Horizontal Seismic Coefficient A_h

Design Horizontal Seismic Coefficient A_h can be determined by the following expression:

$$A_h = \frac{ZISa}{2Rg} \qquad (4.6.1)$$

Provided that for any structure with T<0.1 seconds, the value of A_h will not be taken less than Z/2 whatever be the value of I/R.

Where,

Z = zone factor given in Table 2 of IS Code for the maximum considered earthquake and service of the structure in a zone

I = Importance factor, depending upon the functional use of the structures

R= response reduction factor

S_a/g= Average response acceleration coefficient for rocks or soil sites

The Design Seismic Base shear:

The total design lateral force or design seismic base shear (V_B) along any principal; direction shall be designed by following expression:

$$V_B = A_h * W \qquad (4.6.2)$$

Where,

W = Seismic weight of all floors

A_h = seismic coefficient

Fundamental natural Period

The approximate fundamental natural period of vibration (T_a), in seconds for masonry structure, may be estimated by the formula

$$T_a = \frac{0.09}{\sqrt{d}} * h \qquad (4.6.3)$$

Where,

h= height of building in m

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

Distribution of design base shear

The design base shear (V_B) computed above shall be distributed along the height of the building as per the following expression:

$$\mathbf{Q_i} = \frac{Wihi2}{\sum Wihi2} * V_B \qquad (4.6.4)$$

Where,

Q_i = Design lateral force at i^{th} floor

V_B = base shear

W i = Seismic weight of floor i

h_i= height of floor i^{th} from the base

4.7 Results and discussion

Using the above method the fictitious building is analyzed for gravity loads as well as the horizontal seismic loads using the SAP2000. The results are presented in figure 4.3 to 4.6 and 4.10 to 4.16 .In this study the results of three parametric studies namely, (1) effect of different wall thickness, (2) effect of different floor system on the same structure and (3) the effect of openings sizes of one story building having 3m by 6m in plan and 3m storey height are presented. The plan of the fictitious building is shown in Figure 4.1. For each of the three cases the results are presented in terms of the displacement response.

4.7.1 Single storey building

In the first study, the effect of different thickness of wall, that is, 230 mm, 350 mm, 450 mm, 600 mm, 750 mm and 900 mm of one storey building having different floor rigidities are investigated. The study is further extended with increasing in number of stories up to four storey building. The nomenclatures of the different model are presented in table 4.1. From the analysis, it is found that, the building response during the earthquake excitation largely depends on the types of floor rather than the thickness of the wall.

Comparing the response of building with timber floor and no floor

The top displacement due to earthquake force applied along y-direction, of the one storey building, plan of which is shown in figure 4.1, presented in figure 4.3. It is clear from the figure that the displacement is drastically changes for different thickness of wall when there is no floor assumption. But, it is gradually decreased in its value in the case of 450 mm to 900 mm thick wall. The top displacement, if is 7.57 mm in case of 230 mm thick wall, reduces sharply as the wall thickness increased and drops to 0.755 mm in case of thicker wall of 750 mm or so.

When the same model is analyzed with the timber floor, the top displacement of the building has a similar character as in the timber floor assumption. The top displacement of the building having 230 mm and 350 mm thick wall is reduced in some extent due to the presence of the timber floor, as seen in figure 4.3. The top displacement of the building is 7.57 mm for no floor case, 230 mm thick wall, is nearly equals as for the timber floor for same thick wall, which is 6.3 mm. But, there is no significant difference in the displacement value when the thickness of wall exceeds from 450 mm to 900 mm thick wall, for both timber floor and no floor condition as evident from figure 4.3.

It is clear that, the loosely connected timber floor has some finite in-plane stiffness. Up to 450 mm thick wall, the timber floor exhibits some in-plane stiffness along its direction. But, when the thickness of the wall is increased from 450 mm, the in-plane stiffness of the floor is not significant as compare to the high in-plane stiffness of the thick masonry wall.

Comparison of building responses with respect to rigid floor

The same model is again analyzed for the rigid floor diaphragm. As the in-plane stiffness of floor is increased to infinite, introducing the heavier, stiffer reinforced concrete floor, the thickness of wall seems to have a little significance to top displacement of the building. For the building having 230 mm thick wall, the top displacement is 0.25 mm and for the 900 mm thick wall it is 0.17 mm. It seems that there is no significance difference in the displacement value as in the case of the no floor and timber floor assumption. As is clear from figure 4.3, the top displacement pattern of the buildings having rigid floor has almost constant displacement value.

It may not be necessary to strengthen or uneconomical to construct the 900 mm thick wall with reinforced concrete floor for seismic resistant point of view as compared to the 230 mm thick wall with reinforced concrete floor. Nonetheless, low rise buildings having thicker walls with sufficient connection of timber floor, may be efficient from seismic consideration.

4.7.2 Two storey building

The same model is studied by adding one more storey. As in the previous case the earthquake force is applied along the Y-direction of the building, as shown in figure 4.1.

Comparing the response of building with timber floor and no floor

In no floor case, there is drastic change in the top displacement of the building having 230 mm and 350 mm thick wall as evident from figure 4.4. The top displacement of the building, for the 230 mm thick wall is 19.96 mm but it is sharply reduced to 7.36

mm for the 350 mm thick wall. As, the storey level is increased there is gradual variation of the top displacement of the wall, as compare to the one storey building. So, the top displacement of the building wall is then gradually decreased from 350 mm to 900 mm thick wall.

In case of the timber floor, the top displacement of the building having 230 mm thick wall is significantly reduced in compare to no floor condition. But, in the case of wall thickness from 350 mm to 900 mm, there is no significant difference in the displacement value for both timber floor and no floor condition as seen in figure 4.4.

Comparison of building responses with respect to rigid floor

The top displacement of the building having different thickness has almost constant as same as in one storey building. The displacement pattern is shown in figure 4.4. The top displacement of the building is drastically reduced by improving the floor rigidity. As the thickness of the wall is increased, the top displacement of the building is also increased in case of the high thicker wall. The top displacement of the building having 900 mm thick wall is 0.97 mm and it is 0.95 mm for the 600 mm thick wall. This is due to high thickness wall which increases the inertia force compared to the low thickness of wall.

4.7.3 Three storey building

The displacement pattern of the three storey building for the timber floor and no floor condition are same as in the one and two storey building. It is clear that, as the thickness of the wall increases the in-plane stiffness of the timber floor is less effective. Though, it exhibits some finite stiffness for 230 mm thick wall. Beyond the wall thickness of 350 mm to 900 mm, the displacement of the building depends on the in-plane stiffness of the masonry wall. The displacement pattern is shown in figure 4.5

By adding the rigid floor in the building, the top displacement of the building is decreased as compare to both no floor and timber floor condition. But the top displacement of the building is increased for the high thick wall, say 750 mm to 900 mm, despite of the rigid floor diaphragm as evident in figure 4.5.The top displacement of the building having 750 mm to 900 mm is increased due to the heavier concrete floor and high thickness of wall, which increased the inertial force acting on the building.

Up to the 600 mm thick wall, the top displacement of the building is decreased gradually. As already discussed above, the top displacement value of the building is increased for the 750 mm and 900 mm thick wall due to high seismic force as compare to other. The displacement pattern is shown in figure 4.5.

4.7.4. Four storey building

The displacement pattern is same for the timber floor and the no floor condition as in three storey building. In case of the rigid floor, the top displacement of the buildings having different thicknesses of wall is not like a constant value as in previous case of one and two storey building. The top displacement value is gradually changed from 230 mm to 600 mm thick wall as shown in the figure 4.6. But the top displacement value of the wall having thickness 750 mm and 900 mm is more than 600 mm thick wall. This is due to the high seismic force as compare to the 600 mm thick wall.

The top displacement of the building reduced from 13.85 mm to 8.84 mm for the 230 mm to 600 mm thick wall respectively. But, it is increased to 9.16 to 9.55 mm for the 750 mm to 900 mm thick wall respectively, as seen in figure 4.6.

It is clear that, for higher storey unreinforced masonry building having high thick wall, providing the heavier reinforced concrete floor is not always good. In order to provide the heavier floor, the floor rigidity of the existing timber floor is improved by placing the steel tie rod with the timber joist and other techniques for increasing the floor rigidity. Therefore, the heavy reinforced concrete floor is not always good for improving the seismic behavior of the unreinforced masonry building.

4.7.5 Vertical distribution of lateral forces

In the URM building, the distribution of inertia forces over the height of a building varies or depends on the floor system of the building. In previous study, the horizontal seismic force is distributed uniformly throughout the height of the structure without consideration the types of the floor. However, due to the nature of the floor configuration, the distribution of the seismic force is different. In case of reinforced floor diaphragms, acts as rigid floor, the seismic force should be applied at the mass center of the floor, instead of applying uniformly throughout the height of the structure. But, in the timber floor or flexible floor, the distribution of seismic force may uneven distribution due to insufficient of the wall to floor connection. Therefore, the seismic force is distributed uniformly, instead applying at the mass center of the flexible floor. So, the floor rigidity plays a vital role for distribution of seismic force in the building.

Kappos, A.J. et al (2002) suggested that it becomes crucial to apply the concentrated seismic force at the mass center of the timber floor diaphragm due to flexibility of the floor. So, the seismic forces have to be distributed to each element as an inertial load (product of the element mass and the seismic acceleration), a method which produces an essentially uniform load pattern instead of the "triangular" one specified by the code procedures. In their study, they analyzed an actual two-story stone masonry building situated in Kalamata, Greece, which was damaged by the 1986 earthquake that hit the city. The story height of the building is 4.5 m. In their analysis, they found that the top displacement of the cantilever un-reinforced masonry wall subjected to a triangular pattern of concentrated (at floor levels) forces is 1.9 times the corresponding displacement calculated using the a uniformly distributed loading that gives the same base shear.

The typical codal provision for distribution of the seismic force is inverse triangular. But due to the uniform distribution mass of masonry structure over the height of the structure, for loosely connected timber floor, the seismic force should be applied to the each element of the masonry wall as suggested by the Kappos, A.J. et al. This consideration leads to the uniform distribution of the seismic forces along the height of the structure rather than the inverse triangular profile. Simple inertia force distributions, inverse triangular and uniform are shown in Figure 4.17 (a) and 4.17 (b) respectively.

Therefore, in this section two different load cases are studied for the URM building having rigid floor diaphragm.

1. Inverse triangular load: Due to the presence of the rigid floor on the building, the seismic force should be applied at the mass center of the floor instead of the wall, which produces the inverse triangular loading. The load profile is shown in figure 4.17 (a).

2. Uniformly distributed load: Due to uniformly distributed mass of the building, the seismic force is distributed to each element as an inertial load, which produces the uniformly distributed load. The load profile is shown in figure 4.17 (b).

In order to identify the effect of the vertical force distribution on the URM building having rigid floor diaphragm, the study is again repeated to the different thickness of wall and the number of the story.

The top displacements of the building having different thickness of wall due to the inverse triangular profile and uniform profile are present in Table 4.3.It is found that, for one-story building the top displacement of the building given by the inverse triangular profile is more than 2 times that of uniform profile, as presented in Table 4.3. For two -story, it is 1.9 times; for three- story, it is 1.8 times and that for the four-story building, it is 1.75 times. The base shear for the both distribution of the inertia force is same. From this study it shows that the as the level of the structure is increased, the effect of the inverse triangular loading is gradually decreased.

4.7.6 Effect of opening sizes

Parametric finite-element analysis is carried out in order to investigate the effect of the opening of the same building having different floor system. In this section opening sizes are varies horizontally and vertically in transverse and longitudinal wall of the fictitious building respectively, as shown in figure 4.1. The same model is also analyzed with manually and compares the results. Finite-Element analysis is carried out by using the SAP2000® and manual method is carried out by deep beam theory, as discussed in section 4.5. In this case, the earthquake forces are applied along the opening of the wall. Two cases are studied in order to find the effect of the opening in the shear wall of the URM building having different floor system as mentioned earlier.

These are:

1. Centrally located opening with varying the horizontal width in the transverse wall of the fictitious building, shown in figure 4.1.

2. Centrally located opening with varying the vertical height of in the longitudinal wall of the fictitious building, shown in figure 4.1.

Centrally located opening with varying the horizontal width in the transverse wall

In the transverse wall of the fictitious building, as shown in figure 4.1, the opening is increased from 12 % to 44 %. The vertical height of the opening is kept constant and the horizontal width is varying. The analysis is carried out for both timber and the rigid floor diaphragm. In this case, the effect of the floor rigidity and the thickness of wall are investigated for the different amount of opening. The earthquake force is applied along the y-direction of the building.

As the percentage of the opening is changed greatly, the top displacement of the wall is abruptly changed. The top displacement of building is 6.28 mm for 44 % of opening, while it is only 1.1 mm for 12 % of opening, for 230 mm thick wall. For the high thick wall, it is 0.53 mm for 10 % of opening, and 1.26 mm for 46 % of opening, as shown in figure 4.10. It is clear, as the amount of opening is increased largely; the effect of the floor rigidity is reduced.

Up to 20 % of opening, the floor rigidity has considerable effect as compare to the thickness of the wall, as seen from figure 4.10. But, when the opening is increased more than 20 %, the floor rigidity has less effective. The top displacement of the building is controlled by the thickness of the wall for larger opening.

The results come from the finite element analysis is compared with the manual method. The manual method for determining the top displacement is building is described in section 4.5 of this chapter. It is found that, up to 10 % of the opening in the wall, the manual method works well for all different thickness of the wall. As the amount of the opening is increased more than 20 %, for 230 mm thick wall, the manual method doesn’t works well, as shown in figure 4.12 (a). for the medium thick wall, say 350 mm and 450 mm, manual method is quite similar to the finite element method. There is no significance difference in the top displacement of the building for the same amount of opening, as seen in figure 4.12 (b) and 4.12 (c).

For the high thick wall, in range between 600 mm to 900 mm, the manual method is quite satisfactory for 10 % of the opening in the wall, as compare to the finite element method. But, as the amount of opening is increased more than 10 %, the manual method overestimates the top displacement of the building as compare to the finite element method. The results are shown in figure 4.12 (d) to 4.12 (e).

There is no significant difference in the top displacement of the building for the timber floor as the amount of opening is increased greatly. The top displacement of the building is decreased as the thickness of wall increased as shown in figure 4.11.

Centrally located opening with varying the vertical height in the longitudinal wall

In this study, the vertical height of the opening is increased from 0% to 20 % at the longitudinal wall of the building. In order to find the effect of opening in the wall, the earthquake force is applied along longitudinal wall. Both manual and finite element analysis is carried out as above. By performing the finite element analysis, it is found that there is no significant difference in the top displacement of the building for different thicknesses of wall as seen from figure 4.13 . For 230 mm thick wall with rigid floor diaphragm, as the opening reaches 20 %, the top displacement of building is double by the manual method as compare to the finite element analysis. Similarly, for the 900 mm thick wall ,as the opening is 20 %, the top displacement of the building is also double by the manual method as compare to the finite element method. Figure 4.15 (a) to 4.15 (f) shows the top displacement of the building for the different amount of opening for different thickness of wall by both manual and finite element method.

In longitudinal wall when there is no opening there is no significant difference in the top displacement of wall by both the finite element analysis and hand method as indicated by the figure 4.15 (a) to 4.15 (f). It shows that as the amount of the opening is increased, the manual method overestimates the displacement responses as compare to the finite element analysis. But for the timber floor, the top displacement of the wall has a constant value for the different amount of opening. The displacements value is decreased as the thickness of wall increased as seen the figure 4.14.

4.7.7 Time period of the building

The fundamental time period of the building system with flexible diaphragm, having thick wall, is consistently longer than values estimated with simplified methods given by the current seismic codes. The fundamental time period of flexible and rigid floor is tabulated in Table 4.3.The fundamental time period of the structure changes dramatically if the diaphragms are rigid. It is due to that rigid diaphragm lead to uniform distribution of acceleration and deformation in all connecting elements. In contrast, flexible diaphragms lead to uneven deformation of the connecting elements according to their relative stiffness. Figure 4.16 shows variation of the time period of the building with different wall thickness and floor rigidity. As shown the figure 4.16, the time period of building decreased with increased the wall thickness for rigid floor diaphragm. But, the rate of decrease is not drastically as the thickness of the wall changed. Therefore, floor rigidity plays a vital role in the time period of the building rather than the thickness of the wall.

It is found that as the floor level of the building is increased, the fundamental time period of the building is also increased for both the rigid floor and timber floor diaphragm. Figure 4.16 shows the variation of the time period as the floor level of the building is increased. It is due to as the floor level is increased; the mass of the structure is increased greatly as compare to the stiffness of the building. The fundamental time period of the structure depends on the mass of the structure. As the mass of the building is increased, it decreases the frequency of the structure and which increases the fundamental time period of the building.

The fundamental time period of 2^{nd}, 3^{rd}, 4^{th} and 5^{th} modes of the model is presented on the Table 4.4.

4.8 Conclusion

This chapter focuses on the modeling and analysis of a fictitious building having a simple plan, as shown in figure 4.1. Before analyzing the actual structure a number of parametric analysis is carried out, which reflects the characteristics the URM structure. Before presenting the outcomes of the preliminarily analysis of the fictitious building, it is useful to present various aspects of investigation. The study focuses on the following important parameters:

1. The effect of the thickness of the wall on the URM building response.
2. The effect of the floor rigidity and number of floors on the URM building response.
3. The effect of the opening on the URM building response.
4. The vertical distribution of the seismic load on the URM building response.

From this study the following points are concluded:

i) The top displacement of the building is sharply reduced as the thickness of the wall increased from 230 mm to 450 mm thick wall, in case of the timber floor. For the high thickness of wall, that is, more than 450 mm, the top displacement is gradually decreased.

ii) For high thickness of wall, more than 450 mm, the loosely connected timber floor has no significant effect on the building response. But, the in-plane stiffness of the timber floor has a considerable effect on the 230 mm and 350 mm thick wall. So, consideration of the no floor case for loosely connected timber joist having thick wall gives the same results during the analysis.

iii) As the in-plane stiffness of floor is increased to infinite, by introducing the heavier and stiffer reinforced concrete floor, the top displacement of the building is drastically reduced as compare to the timber floor.

iv) For low rise URM building, the floor rigidity play a vital role to control the top displacement of the building rather than the thickness of the wall.

v) For reducing the out-of-plane failure of URM building the timber floor must be replaced by the heavy reinforced concrete floor, which acts as rigid floor diaphragm, for lower storey building.

vi) For higher storey URM building, having the high thick wall, the replacement of the timber floor with heavy reinforced concrete floor is not a good option for strengthening the building.

vii) From the analysis it is found that, for the four storey's URM building, for 750 mm thick wall, the top displacement of the building with timber floor and rigid floor is not different. Therefore, it is not necessary to replace the existing floor with concrete floor for strengthening of the traditional building. Hence, for the high rise with thicker wall of URM building, the in-plane rigidity of the existing floor is increased by proper tying the floor with the walls, placing the cross beam across the timber joist and nailing the plank with the timber joist.

viii) The in-plane displacement of the building is depends on the opening of the building rather than the floor rigidity. As the amount of the opening is increased, the floor rigidity decreased and the displacement of the building is control by the thickness of the wall. If the percentage of opening is increased the in-plane displacement is increased greatly.

ix) For the larger opening, the manual methods overestimate the response of the building as compare to the finite element method.

x) The time period of the building is also dependent on the flexibility of the floor system. Diaphragm flexibility increases the fundamental time period of the building as compare to the rigid floor diaphragm.

xi) In the URM building, the seismic loading is dependent on the floor rigidity. As the mass of the masonry is distributed uniformly, the seismic load should be applied uniformly throughout the height of the building rather concentrated on the mass center of the floor. This produces the uniformly distributed loading pattern.

xii) For the rigid floor diaphragm two types of loading are applied; inverse triangular, from the typical code provision, and the uniformly distributed loading. The inverse triangular profile gives more top displacement than that of uniform profile for the same base shear force. But the effect of triangular force is reduced as the height of the structure is increased.

Table 4.1: Nomenclatures of models used for the analysis

S.N	Description of the model	Label			
		One story	Two story	Three story	Four story
1	Building having No Floor	1-No Floor	2-No Floor	3-No Floor	4-No Floor
2	Building with Timber floor (Flexible)	1-Timber Floor	2-Timber Floor	3-Timber Floor	4-Timber Floor
3	Building having Rigid (Reinforced concrete) Floor	1-Rigid Floor	2-Rigid Floor	3-Rigid Floor	4-Rigid Floor

Table 4.2: Comparisons of the displacement due to the inverse triangular and uniform profile along the height of the building

Wall Thickness	For one story			For two story			For three story			For four story		
	Top displacement		Ratio	Top displacement		Ratio	Top displacement		Ratio	Top displacement		Ratio
	Triangular	Uniform		Triangular	Uniform		Triangular	Uniform		Triangular	Uniform	
(mm)	(mm)	(mm)		(mm)	(mm)		(mm)	(mm)		(mm)	(mm)	
230	0.60	0.252	2.38	2.7	1.431	1.89	9.33	5.2	1.79	24.4	13.85	1.76
350	0.530	0.216	2.45	2.3	1.22	1.89	7.8	4.39	1.78	20.16	11.51	1.75
450	0.486	0.2	2.43	2.0	1.09	1.88	6.96	3.89	1.79	17.9	10.17	1.76
600	0.44	0.183	2.4	1.92	0.95	2.02	6.47	3.4	1.9	16.61	8.84	1.88
750	0.39	0.163	2.39	1.8	0.95	1.89	6.18	3.47	1.78	16.06	9.16	1.75
900	0.4	0.166	2.41	1.85	0.97	1.89	6.42	3.6	1.78	16.76	9.55	1.75

Table 4.3: Comparisons of the fundamental time period of the building having flexible and rigid floor for first modes of vibration

	For one story			For two story			For three story			For four story		
Wall Thickness	Free vibration analysis		IS Code	Free vibration analysis		IS Code	Free vibration analysis		IS Code	Free vibration analysis		IS Code
	Flexible	Rigid		Flexible	Rigid		Flexible	Rigid		Flexible	Rigid	
(mm)	(Sec)	(Sec)	(Sec)	(Sec)	(Sec)	(Sec)	(Sec)	(Sec)	(Sec)	(Sec)	(Sec)	(Sec)
230	0.1387	0.05770	0.1064	0.20669	0.10073	0.2128	0.24680	0.16732	0.3192	0.3102	0.25936	0.4256
350	0.10409	0.04351	0.1021	0.1735	0.09169	0.2041	0.2022	0.1570	0.3062	0.27206	0.2433	0.4082
450	0.08327	0.03858	0.1006	0.1389	0.0880	0.2012	0.180	0.15267	0.3019	0.25496	0.23694	0.4025
600	0.06301	0.03518	0.0986	0.10633	0.08463	0.1972	0.1622	0.14924	0.2958	0.24227	0.23314	0.3944
750	0.05312	0.03325	0.0986	0.09579	0.08113	0.1972	0.15284	0.14361	0.2958	0.23092	0.22430	0.3944
900	0.04794	0.03294	0.0949	0.09392	0.08170	0.1897	0.15284	0.14586	0.2846	0.23464	0.22862	0.3795

Table 4.4: Comparisons of the fundamental time period of the building having flexible and rigid floor for 1^{st},2^{nd}, 3^{rd}, 4^{th} and 5^{th} modes

Wall Thickness	Storey Level	Fundamental time period for rigid floor diaphragm (Seconds)					Fundamental time period for timber floor diaphragm (Seconds)				
(mm)		Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5
230	First Storey	0.05770	0.05464	0.04294	0.04030	0.03822	0.14678	0.13782	0.07822	0.06601	0.0604
350		0.4351	0.03771	0.03191	0.0300	0.02790	0.10409	0.09882	0.0582	0.0505	0.0460
450		0.11968	0.09452	0.08795	0.08138	0.07569	0.08327	0.07954	0.04779	0.04224	0.03814
600		0.0318	0.02656	0.02417	0.02415	0.02065	0.06301	0.06066	0.03719	.03531	0.02986
750		0.03325	0.02581	0.02349	0.02079	0.01837	0.05311	0.05167	0.03276	0.0325	0.0266
900		0.0294	0.02489	0.02359	0.01880	0.01698	0.04794	0.04746	0.03177	0.02943	0.02544
230	Second Storey	0.1007	0.06430	0.05306	0.04397	0.02390	0.020699	0.1007	0.07685	0.06457	0.03449
350		0.09169	0.06105	0.0507	0.04694	0.04155	0.17354	0.15783	0.09164	0.07910	0.07589
450		0.0880	0.06002	0.04966	0.03765	0.03411	0.13894	0.13148	0.08563	0.06880	0.06084
600		0.08463	0.05952	0.04857	0.02937	0.02922	0.10633	0.10193	0.07864	0.06250	0.05025
750		0.08113	0.05858	0.04788	0.02806	0.02664	0.09570	0.08574	0.07279	0.0605	0.04533
900		0.08170	0.05657	0.04850	0.02789	0.02663	0.09639	0.07201	0.05868	0.04361	0.04360

Table 4.4: Continue

Wall Thickness (mm)	Storey Level	Fundamental time period for rigid floor diaphragm (Seconds)					Fundamental time period for timber floor diaphragm (Seconds)				
		Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5
230	Three Storey	0.16732	0.10752	0.05706	0.04283	0.02933	0.24680	0.12034	0.11398	0.08776	0.03933
350		0.15708	0.1034	0.07541	0.0556	0.04960	0.20227	0.19611	0.15542	0.11776	0.11122
450		0.15269	0.1022	0.07445	0.0508	0.03998	0.18053	0.15738	0.14206	0.10542	0.0942
600		0.14924	0.1023	0.07325	0.0476	0.03996	0.16255	0.12277	0.11457	0.10392	0.07033
750		0.1436	0.1006	0.07238	0.04568	0.03994	0.152844	0.1099	0.10186	0.0965	0.06466
900		0.1458	0.0970	0.0736	0.04551	0.03992	0.15713	0.10538	0.09954	0.0633	0.05727
230	Four Storey	0.2593	0.1650	0.1023	0.08067	0.07340	0.31021	0.17046	0.14922	0.0900	0.04902
350		0.2433	0.15794	0.1006	0.07238	0.05332	0.27206	0.21463	0.20469	0.16111	0.14721
450		0.2369	0.15603	0.09954	0.06954	0.05530	0.25496	0.1883	0.1644	0.1582	0.11751
600		0.2331	0.15691	0.09810	0.06701	0.05328	0.25111	0.16338	0.15680	0.09144	0.08787
750		0.2243	0.15407	0.09702	0.06460	0.05324	0.23092	0.15518	0.13427	0.1007	0.08361
900		0.2286	0.1480	0.0989	0.06489	0.05322	0.23892	0.15143	0.12833	0.07993	0.07630

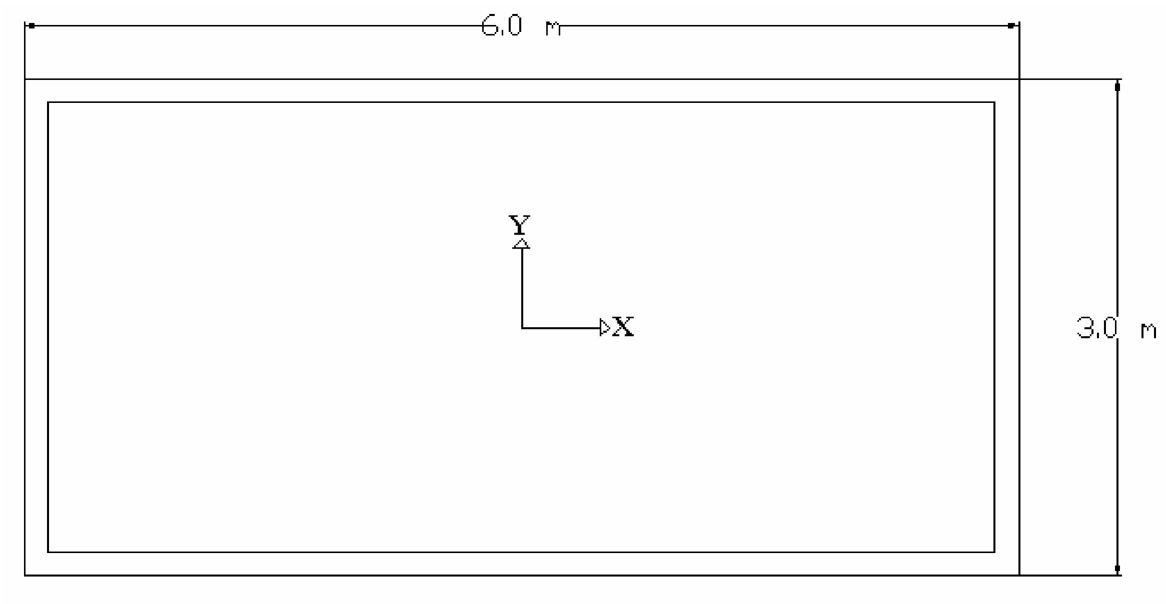


Figure 4.1: Plan of the fictitious building

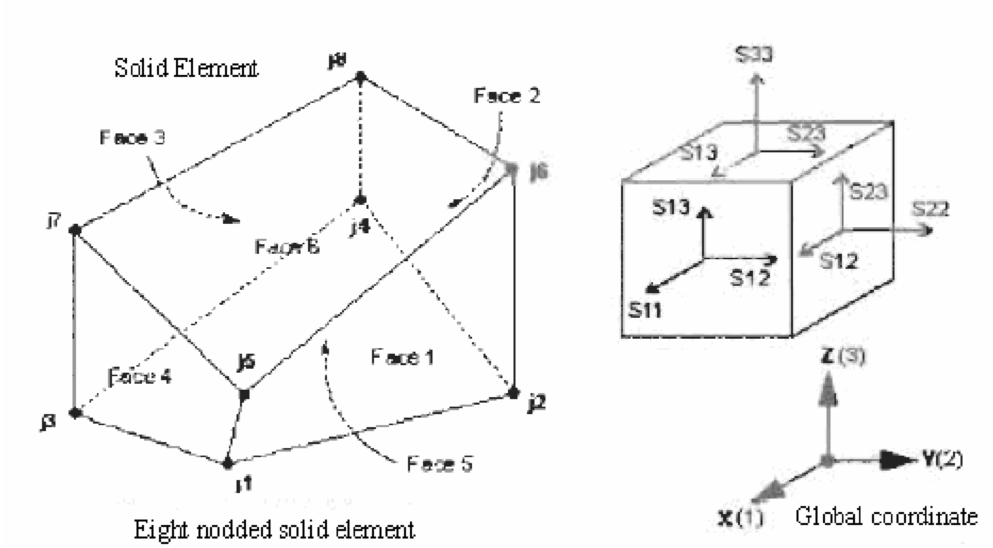


Figure 4.2 Solid element stresses and sign convention in SAP2000

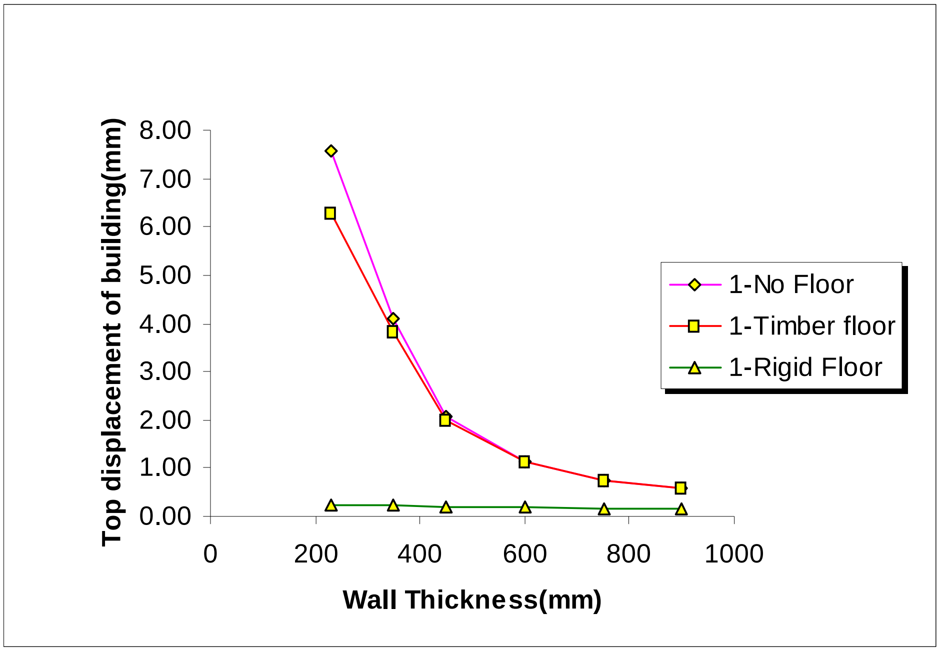


Figure 4.3: Top displacement of one storey building with different thicknesses of wall and different floor system (Earthquake force along y-direction)

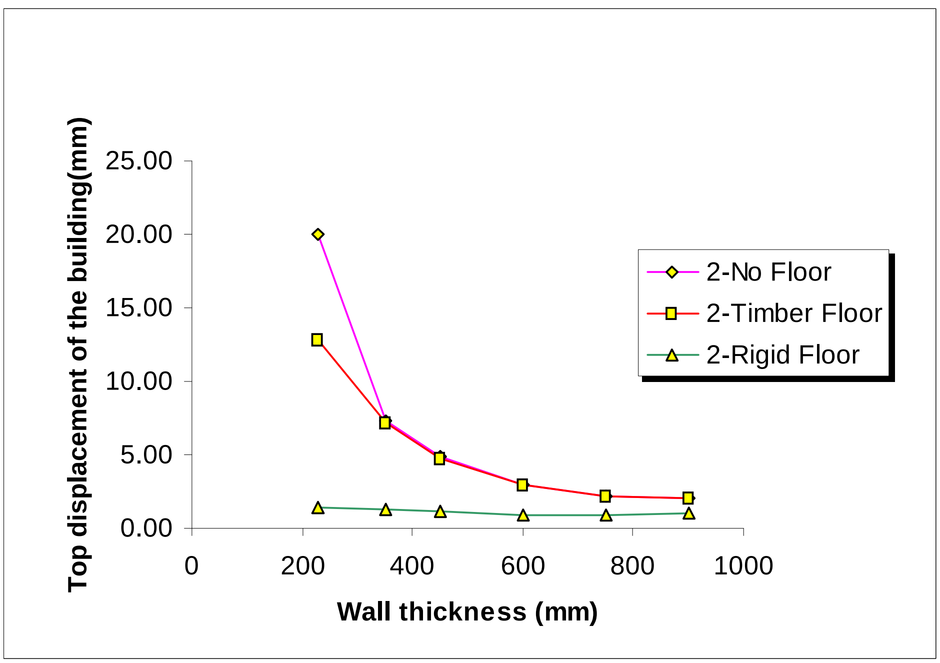


Figure 4.4: Top displacement of the two-story building having different floor systems (Earthquake force along y-direction)

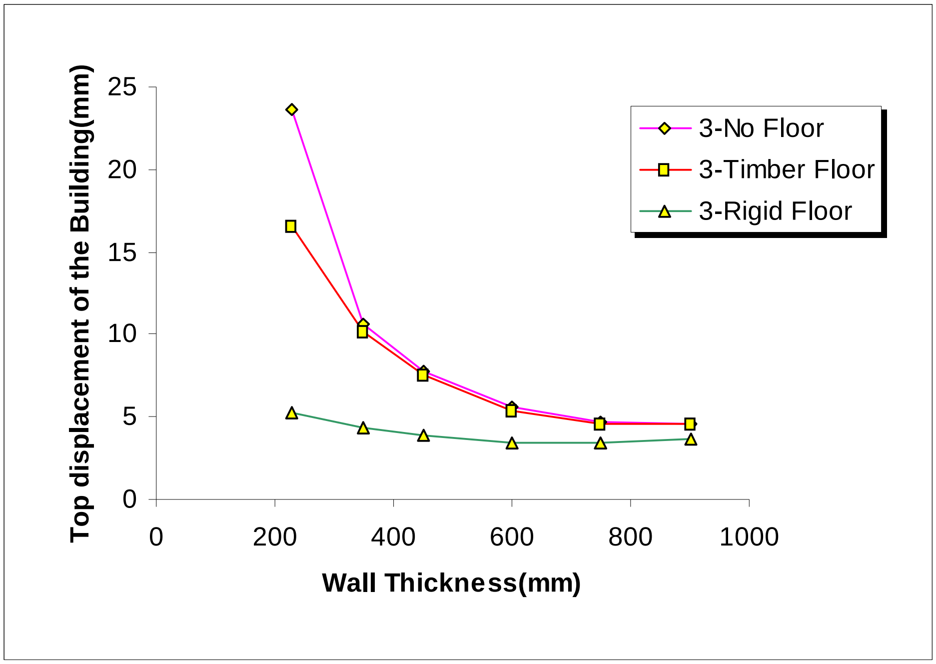


Figure 4.5: Top displacement of the three-story building having different floor rigidity

(Earthquake force along y-direction)

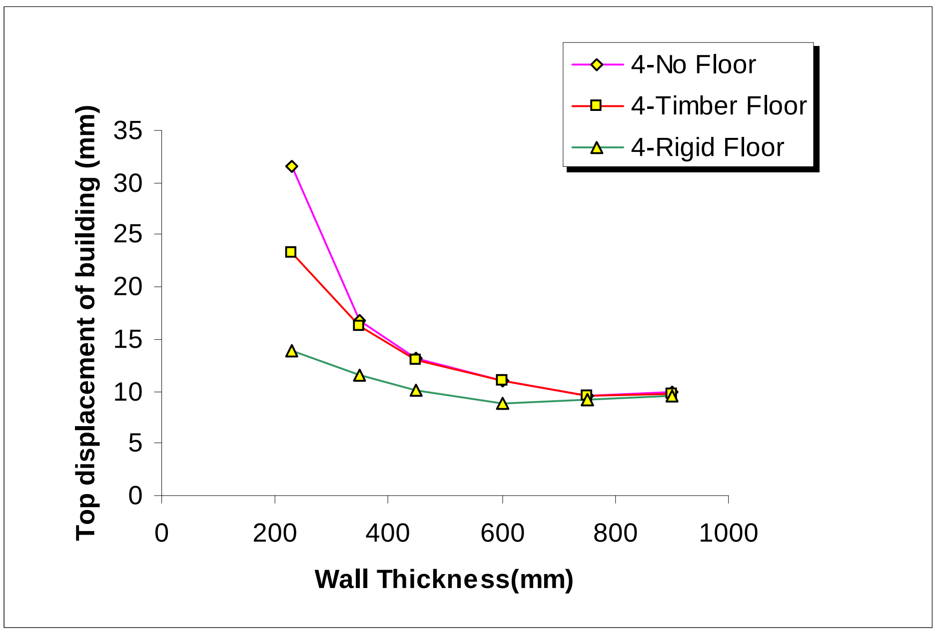


Figure 4.6: Top displacement of the Four-story building having different floor rigidity (Earthquake force along y-direction)

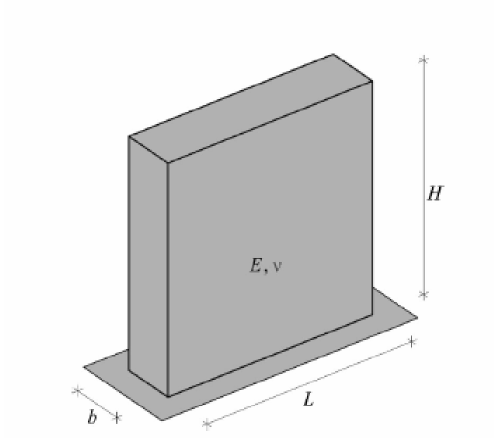
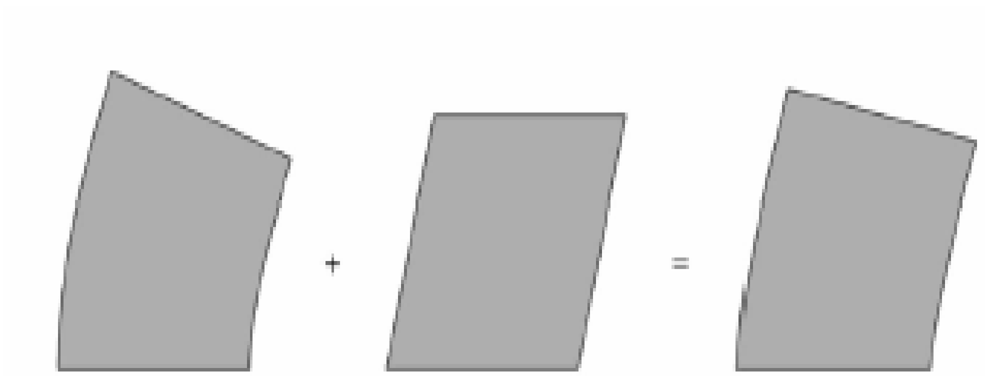


Figure 4.7: Prismatic Shear wall



Flexural Deformation   Shear Deformation   Total Deformation

Figure 4.8: Shear Wall Deformation

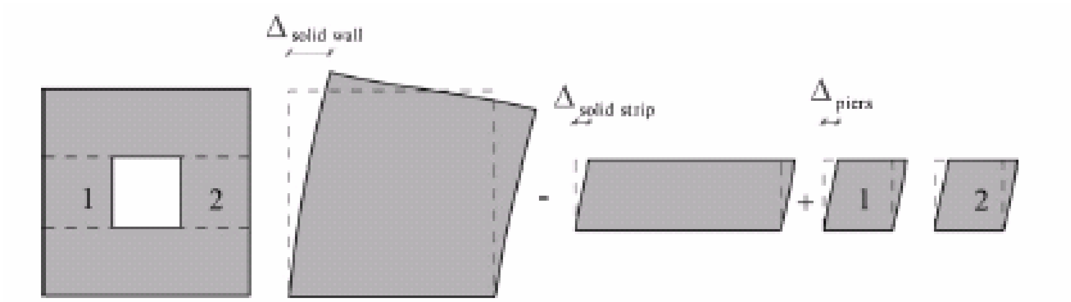


Figure 4.9: Manual Hand method for finding displacement of shear wall with openings

(Source: A. Neuenhofer, Journal of Structural Engineering, ASCE/November 2006)

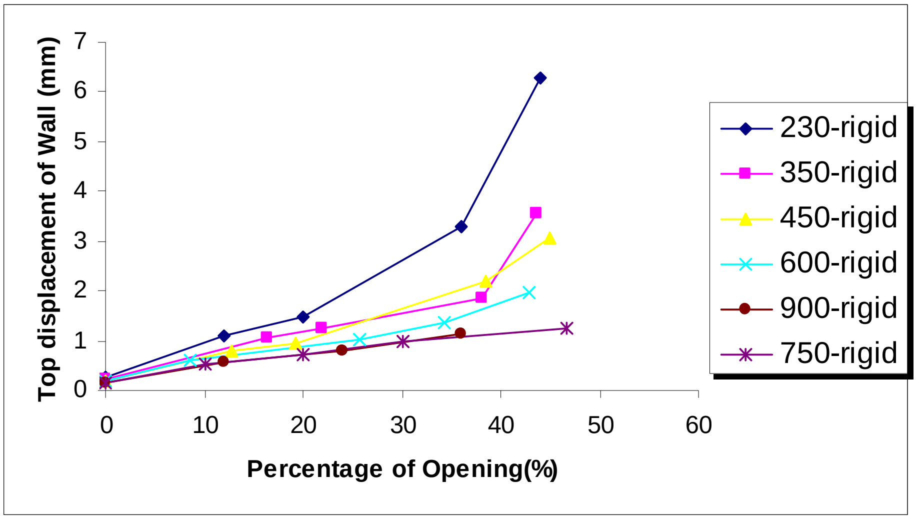


Figure 4.10: Top displacement of the one storey building having rigid floor diaphragm with different amount of opening in transverse wall (Earthquake force along Y-direction)

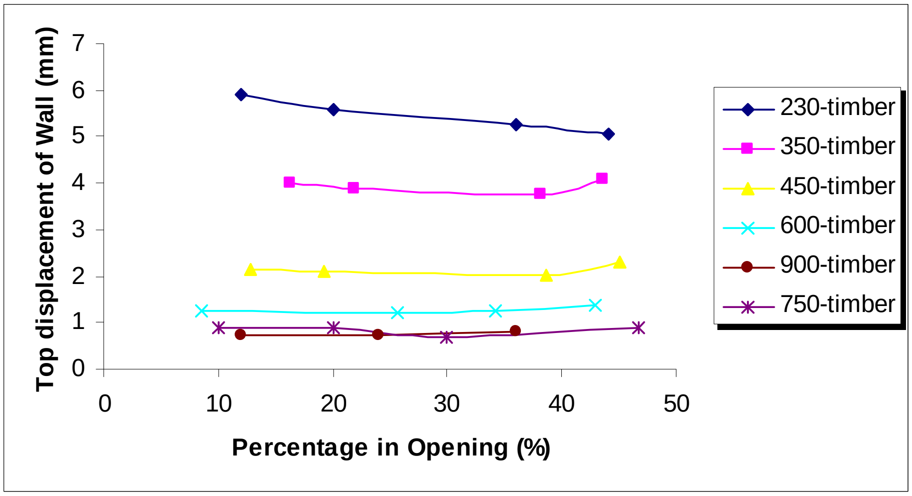
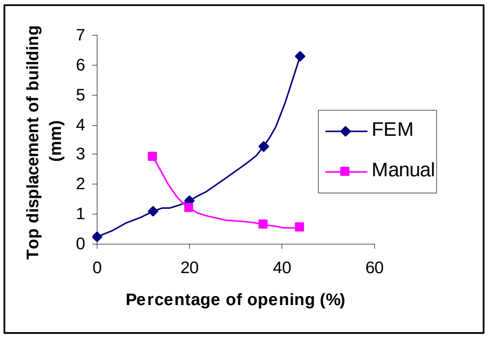
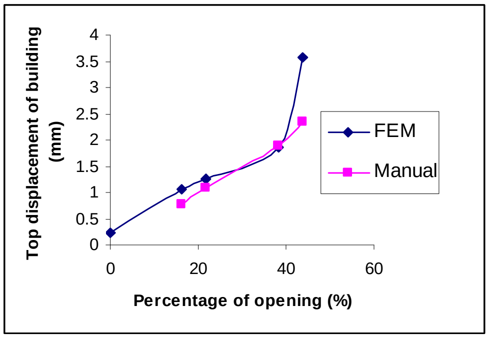


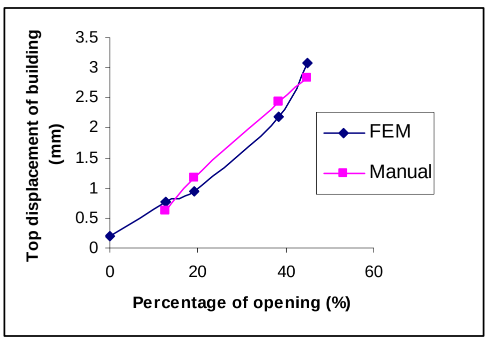
Figure 4.11: Top displacement of the one storey building having timber floor diaphragm with different amount of opening in transverse wall (Earthquake force along Y-direction)

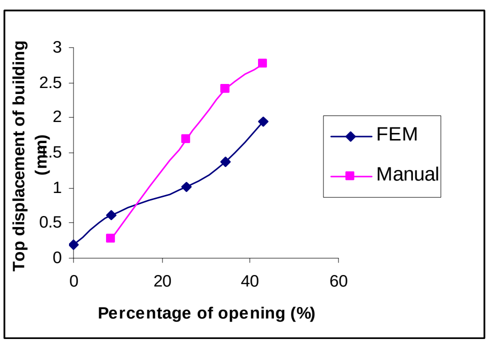




(a) For 230 mm thick wall

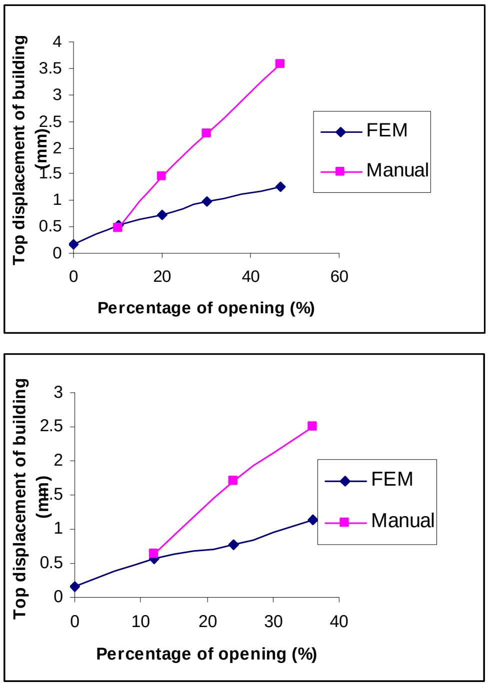
(b) For 350 mm thick wall





(c) For 450 mm thick wall

(d) For 600 mm thick wall



(e) For 750 mm thick wall (f) For 900 mm thick wall

Figure 4.12: Comparison between the top displacements of the one storey building, with the manual method for different thicknesses of wall having the rigid floor diaphragm, opening in the transverse wall of the fictitious building. (Earthquake force along y-direction)

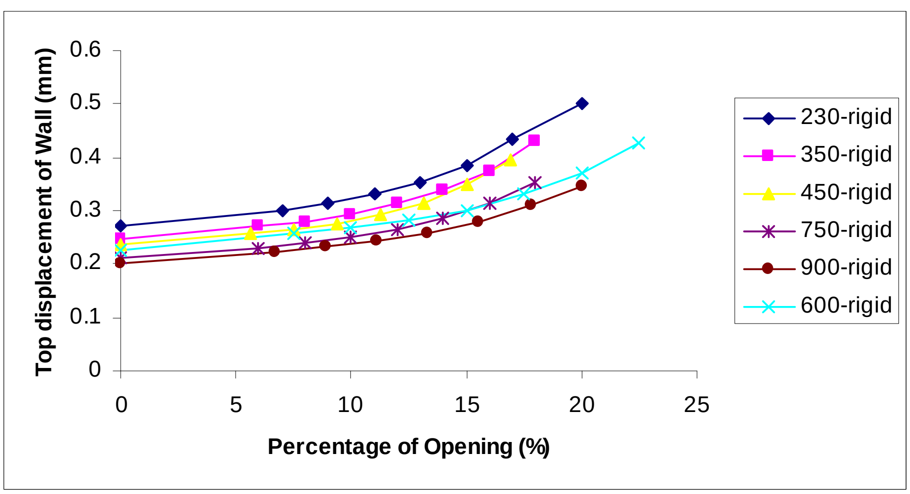


Figure 4.13: Top displacement of the one storey building having rigid floor diaphragm with different amount of opening in longitudinal wall (Earthquake force along X-direction)

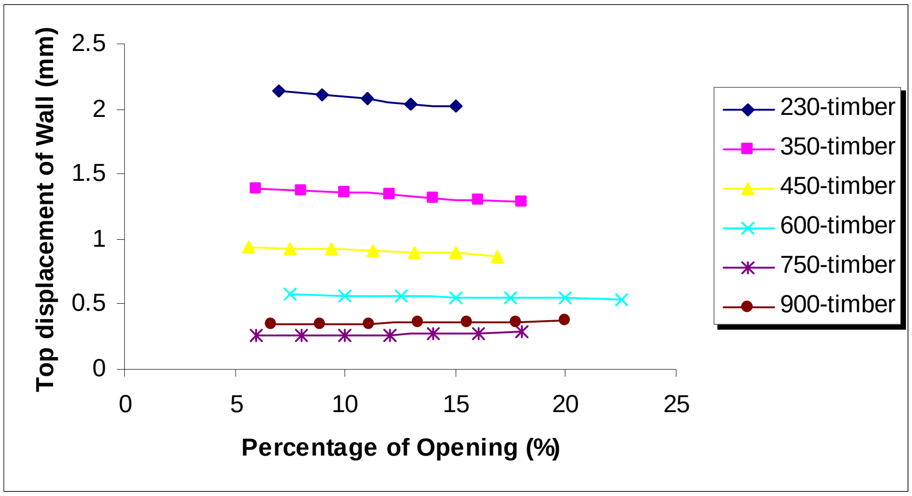
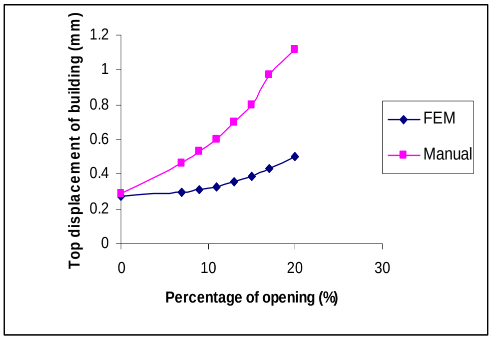
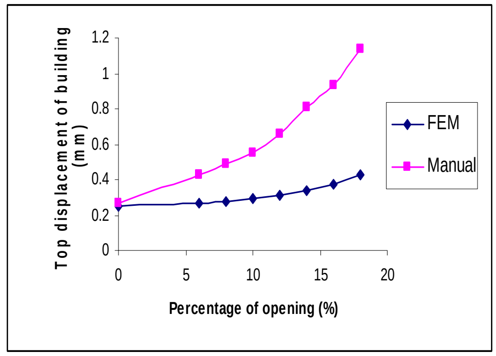


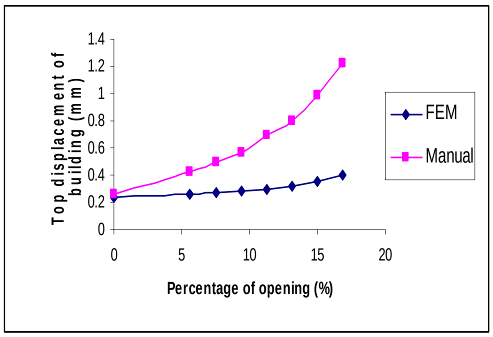
Figure 4.14: Top displacement of the one storey building having timber floor diaphragm with different amount of opening in longitudinal wall (Earthquake force along X-direction)



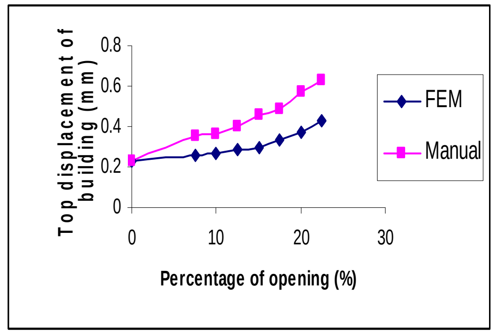
(a)For 230 mm



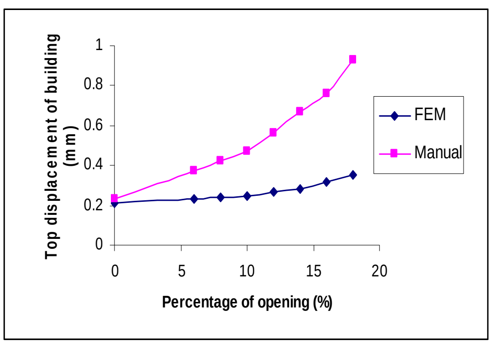
(b) For 350 mm



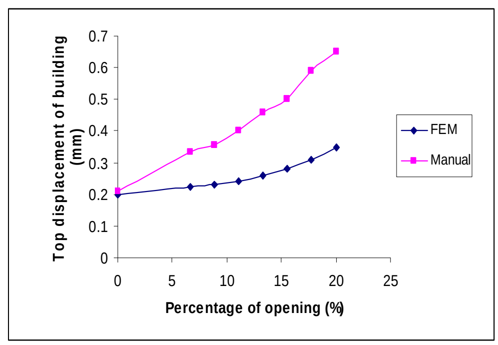
(c)For 450 mm



(d)For 600 mm



(e)For 750 mm



(f)For 900 mm

Figure 4.15: Comparison between the top displacement of the one storey building with the manual method for different thicknesses of wall having the rigid floor diaphragm, opening in the longitudinal wall of the fictitious building.

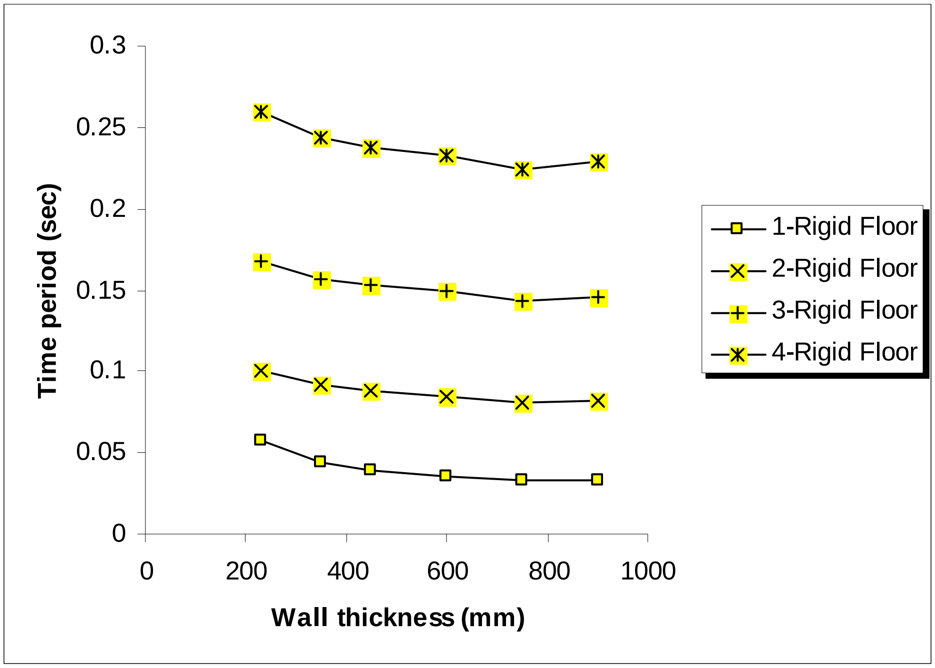


Figure 4.16: The time period of the building having different wall thickness and floor rigidity

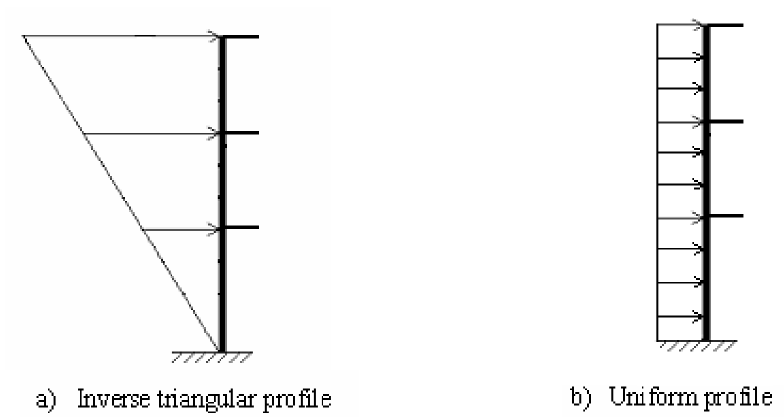


Figure 4.17: The vertical distribution of the inertia forces in the structure.

CHAPTER 5

SEISMIC PERFORMANCE OF RANA PALACES

5.1 General

In Kathmandu valley, the different Governmental and public offices are using old traditional buildings, extensively. In chapter three, many of these buildings are listed with their current status and it shows that the importance of these buildings in daily use. In order to preserve these buildings for their future use and also to the cultural heritage of nation, the assessment of theses building from seismic point of view is very important. In the past, these buildings were constructed without seismic consideration. In this chapter, the original palace constructed in 1923 AD named Shital Niwas is studied which was extensively damaged in 1934 AD earthquake. It was constructed immediately after 1934 AD earthquake on the same ground. Only north wing of the building is taken for the study.

5.2 Description of the building

The old Shital Niwas was constructed in 1923 AD and it was extensively damaged in 1934 AD earthquake and hasty constructed after earthquake without consideration of seismic effect on the same ground. This building is in courtyard type. It has two courtyards of different types as shown in the figure 5.1. The sizes of the Courtyard I and Courtyard II are 44 m by 37 m and 26 m by 25 m respectively. The building is long in length and narrow in width, the aspect ratio is more than five. Ground floor covers the area of 3346 square meter. The Ground, First and Second floor plan of the building are shown in figure 5.2, 5.3 and 5.4 respectively.

The main lateral load-resisting element of the building is unreinforced thick masonry wall. The thickness of wall is 0.75 m. The walls were constructed by high class burnt clay brick with lime Surkhi mortar. There is a long wall of length 47.25 m in corridor, as shown in figure 5.5, without any cross wall and having the maximum number of openings, which is seismically vulnerable.

The Shital Niwas is extended up to three stories. For vertical rise of the building timber floor were used which one of the popular construction during that time. There is large number of openings in the wall of building. The sizes for doors and windows are same and place in same horizontal and vertical alignment throughout the building. The foundation of the building is rigid and in strip footing.

5.3 Modeling and analysis

For modeling the building, only a North wing of the Shital Niwas is taken. The North wing of the building is encircling in the figure 5.3. The Ground floor, the First floor and the Second floor plan of the North wing are shown in figure 5.5, 5.6 and 5.7 respectively. The length and width of the north wing is 47.25 m and 9.0 m respectively. The length of building is more than five times greater than the width of the building. The building has three stories and its story height is 4.0 m. The mechanical properties of the masonry and timber for analysis are tabulated in Table 2.1. There is no opening in the x-direction but in y-direction there are plenty of openings. The size of the window is 1.5 m by 2.0 m and door is 1.5 m by 2.8 m. The building has the internal wall having the 0.75 m thickness and is extended throughout the building. The un-reinforced brick masonry assumed as homogenous and isotropic material.

For modeling, the thick masonry wall is modeled with eight-nodded solid element of SAP2000, as shown in figure 4.2. Ramos, L.F. et al (2004) suggested that it is not realistic to use detailed models of walls and connections in large scale analysis, not only because the geometric and material properties of the constituents/economy constraints. For most large-scale analysis, it is acceptable to model both regular masonry and rubble masonry walls assuming a continuum homogenous material.

The internal composite walls, the timber roofs and others architectural parts associated with the building are not considered in the modeling. However, the loads associates with these elements were included in the analysis. The foundation of building was fully restrained. The whole building is not taken for modeling, for simplicity one wing of the building is modeled. The 3-D homogenous solid element model of the modeled building having the timber floor is shown in figure 5.17.

For modeling the timber floor, a three dimensional linear beam element is used to model the timber joist. The connection of the timber floor/roof with the masonry wall is neglected and assuming that it is simply rest on the wall by considering the fact that the timber nails or iron ties, if present, were heavily deteriorated or damaged over the long years. So, simply supported connection is used for the modeling the joint between the timber joist and masonry wall. For modeling the reinforced concrete floor, the diaphragm constraints is applied in chapter 4 at the storey level of the building, which is not applied for the timber floor.

The model is analyzed by the Seismic Coefficient method, detail describe in chapter 4, in which seismic effect, that is, a horizontal force is considered as the percentage of the total weight of the building. In this method, dynamic forces, which act on the structure during the excitation, are converted into equivalent horizontal force. In this research work the seismic coefficient method is used as described by the IS 1893 (Part I): 2002.

The distribution of lateral load is different for different floor system of unreinforced masonry building. The commonly adopted inverse triangular force distribution is not applicable to the flexible floor diaphragm. Because, the in –plane stiffness of the thick shear wall is relatively larger than the floor and the magnitude of lateral force at all level were nearly equal or same. Therefore, uniform pattern of loading was used for the analysis of the loosely connected timber floor. For simulations of the numerical model a commercial program SAP2000 is used.

5.4 Result and Discussion

In this chapter, actual old traditional building, Shital Niwas, is taken as a case study. The portion of building used for the analysis is shown figure 5.7. Before analyzing an actual structure a preliminary analysis was carried out by using a simple plan of the fictitious building, as discussed in the chapter 4. In finite element model, the prime concern is given to the floor rigidity and the long span wall.

The Ground floor, First floor and Second floor plan of the North wing of the studied building are presented in figure 5.5, 5.6 and 5.7 respectively. The existing plan of the studied building is divided into number of grids as shown in the respective figures. The longitudinal walls of the building are termed as Grid A-A, Grid B-B and Grid C-C. The transverse walls at the end of the building are termed as Grid 1-1 and Grid 6-6 respectively. While, the cross walls, at different location along the horizontal distance of the longitudinal wall, termed as Grid 2-2, Grid 3-3, Grid 4-4 and Grid 5-5 at distance of the longitudinal wall 14.25 m, 22.5 m, 25.5 m and 33.75 m along the longitudinal wall ,from the origin of the building, shown in figure 5.5 , respectively. In the first floor and the second floor, the Grid 3-3 does not exist at a distance of 22.5 m.

The longitudinal walls, termed as Grid A-A and Grid B-B are connected with number of cross walls as shown in figure 5.5 to 5.7 , but the longitudinal walls along the Grid B-B and Grid C-C does not connect with the cross walls except end walls of the building. These walls are connected to each other by means of the existing floor. Due to the absence of the cross wall in between the Grid B-B and Grid C-C, a long corridor of size 2.5 m by 45.75 is exists at all the floors of the building. And also the existence few numbers of the cross walls, along the longitudinal direction of the building, which results the big sizes of rooms. The maximum size of the room is 12.75 m by 4.25 m.

In Ground floor, there are four number of cross walls which connects Grid A-A and Grid B-B, from left of building plan shown in figure 5.5, at 14.25 m, 22.5 m, 25.5 m and 33.75 m respectively. But in the First and the Second floor, there are only three cross wall at 14.5 m, 25.5 m and 33.75 m respectively, from the left of the building plan shown in figure 5.6 and 5.7.

The studied building is investigated for its performance in-terms of displacement for the existing timber floor, position of the cross wall and the unsupported long wall throughout the height and length of the building. In general, the maximum out-of-plane (horizontal) displacement of the longitudinal wall, the in-plane displacement of the cross and end walls of the building, the maximum and minimum vertical deflection of the building, at different horizontal distance of the building, along the height. For the seismic analysis, earthquake force is applied along the y –direction of the building.

Out-of –plane displacement of the building, when seismic force perpendicular to the longitudinal direction

So, due to the less number of the cross wall and large sizes of rooms, the horizontal displacement of the building at the floor level is not same. The out-of-plane displacement at the floor level is different in each span of the wall, along the longitudinal direction. There is maximum displacement at distance of 7.5 m and 39.75 m, from the left of the building plan, along the x- axis, shown in figure 5.5. The out-of-plane displacement is drastically reduced near the cross wall, that is, at 14.25 m, 22.5 m 25.5 m and 33.75 m, at Grid 2-2, Grid 3-3, Grid 4-4 and Grid 5-5 respectively.

The out-of-plane displacement of the longitudinal wall of the building has more significant effect from 0 to 14.25 m and from 33.75 m to 47.25 m, that is, Room A and Room D in compare to Room B and Room C, due to the long span of the wall.

In case of the first floor level, for longitudinal wall along the Grid A-A, at a distance of 7.5 m from the left of the building plan shown in figure 5.5, the top displacement of the building is 9. 7 mm and it reduced to 4.21 mm at a distance of 14.25 m and reached 5.81 mm at 18.75 m. And it is again decreased to 3.1 mm at 22.5 m and increased to 4.9 mm to 5.8 mm at 25.5 m and 28.5 m respectively and it decreases to 4.03 mm at 33.75 m. From 33.75 m to 39.75 m the displacement value is increased from 4.03 mm to maximum of 10.22 mm and then again decreased to 2.06 mm at 47.25 mm distance from the left, that is, next end of the building. The above displacement pattern is described in figure 5.11.

The horizontal displacement of the building, at the floor level, due to walls along the Grid A-A, Grid B-B and Grid C-C, follow the same pattern as described in figure 5.11 to 5.13. But, at Grid 2-2, 3-3 and 5-5, that is, at distance 14.25 m, 22.5 m and 33.75 m along the longitudinal direction of the building, at the first floor level, the displacement value by these three walls are little bit different. The displacement values are shown in figure 5.11 to 5.13. In that location longitudinal wall along the Grid C-C deflects more in compare with Grid A-A and Grid B-B. This is due to that the Grid A-A and Grid B-B are connected with cross walls while there are no cross walls in between Grid B-B and Gris C-C, as already mentioned above.

In case of the second floor level, the displacement pattern of all longitudinal walls are in same pattern, as above, as shown in figure 5.11 to 5.13. As the Grid C-C does not connect with the cross wall, as Grid A-A and Grid B-B, there is no significant different in the displacement pattern at the location of the cross wall. But the longitudinal wall along the Grid C-C has little bit more displacements value, but which is not significant, The internal wall, that is, Grid B-B, behaves differently in the first floor and top floor level. In the first floor level, the displacement pattern of the Grid B-B is out-of-phase with rest of two walls from 33.75 m to 47.25 m and in the top floor it is out of phase from 0 to 24.75 m.

It is clear that, the presence of the cross wall the out-of-plane displacement of the building is drastically reduced. Due to loosely, that is, inadequate, connection of the existing timber joist to the masonry walls, the floor cannot act as a rigid diaphragm and allow for the differential movement and cantilever plate action between walls, leading to collapse of the walls. And also due to larger sizes of the room which separated the cross wall apart greatly, making the walls more vulnerable to overturning during the earthquake.

Due to the insufficient transverse connection to the longitudinal walls and the longitudinal direction of the building is more than five times larger than the transverse direction, the whole building cannot acts as a single unit and the longitudinal and transverse walls behaves independently and leads to failure during the earthquake.

In–plane displacement of the building, when seismic force perpendicular to the longitudinal direction

The in-plane displacement of the building due to the cross walls and end walls is also studied. The story level displacement due to the transverse wall, parallel to the seismic excitation, that is, at Grid 1-1 and 6-6 ,the end walls at distance 0 m and 47.25 m and cross walls, Grid 2-2, 3-3 and 5-5 at distance 14.25 m , 25.5 m and 33.75 m respectively, presented in figure 5.14. The displacement patterns of the end walls are similar and the top floor level displacement is 5.94 mm which is nearly six times smaller than the maximum out-of- plane displacement. Similarly, cross wall at distance of 14.5 m and 25.5 m, along Room B, have the similar displacement pattern. The top floor level displacement value is 13.23 mm, which is more than two times less than the maximum out-of-plane displacement of the building.

But, transverse wall at Grid 5-5, distance of 33.75 m, which separate the Room C and Room D, the displacement pattern is same at the Ground floor level. For the First and Second floor the displacement pattern is different than other two cross walls and end walls of the building. The maximum value of the top floor level displacement is 16.98 m as shown in figure 5.14.

Therefore, the in-plane displacement of the cross walls are more than that of the end walls of the building. And also the displacement pattern of the cross walls are not same, along the height of the building. In Ground floor, there are four number of the cross walls at distance of 14.25 m , 22.5 m, 25.5 m and 33.75 m respectively; but in the First and Second floor the cross wall at a distance of 22.5 m is not exists. Due to this fact the displacement pattern of the cross walls are different.

Vertical displacement of the building, when seismic force perpendicular to the

longitudinal direction, along the height

Since the existing floor does not have sufficient rigidity, therefore the inter-storey displacement of the building, along the height of the building is different. The inter-storey displacement between the first floor level and the second floor level is different with the second floor level and top floor level.

Figure 5.15 to 5.16 describes the vertical displacement pattern, at the story level, of the building of the longitudinal walls, Grid A-A, Grid B-B and Grid C-C respectively, at different location, where the maximum and minimum out-of- plane displacement are observed respectively. It shows that the vertical displacement pattern is different in these locations with respect to each floor. The vertical displacement near or at the cross wall is nearly half than the maximum value. And also it has different displacement value at the distance of 7.5 m and 39.75 m, along the Grid A-A, of 27.53 mm and 30.9 mm respectively. Therefore along the longitudinal direction, Grid A-A, Grid B-B, and Grid C-C, the vertical displacement pattern is different, that is, different value.

The inter-story displacement with respect to floor is also different. At 7.5 m, where maximum out-of- plane displacement is expected, along the Grid A-A, the inter-story displacement difference with respect to the second floor to the first level is 11, while the difference between the top floor and the second floor is 6.8. At 39.75 m, where maximum out-of- plane displacement is expected, the difference of the storey level displacement between the second floor and the first floor level is 12.5 and it is 8.2 in between the top and the second floor level.

Due to the presence of the timber floor, the longitudinal wall along the Grid C-C, this is not connected with the Grid B-B through cross wall, behaves like Grid A-A and Grid B-B. Therefore, the failure of such old traditional building is collapsed by the unequal vertical deformations of the outer walls and cross walls.

Fundamental Time period

The fundamental time period of the building depends on the floor rigidity. The value of the fundamental time period of the building, for different floor rigidity, for the first five modes, is tabulated in table 5.1. The fundamental time period for the timber floor is quite longer as compare to the rigid floor diaphragm. The fundamental time period (in seconds) of the first and the fifth modes for the timber floor and the rigid floor are 0.35182, 0.19725 and 0.22039, 0.06195 respectively.

Figure 5.19 (a) to 5.19 (e) shows the first five mode shapes of the building with the fundamental time period. The fundamental time period of the study building is 0.157 seconds as per the IS code which is less than the free vibration analysis of the first mode of vibration.

5.5 Conclusion

For assessing the seismic performance of the old traditional building Shital Niwas is taken as a case study. Before, analyzing such building a preliminarily analysis is carried out for the fictitious building having simple plan in chapter 4. The initial outcomes are used for the interpretation of analysis of the actual structure.

For the simplicity, a North wing of the Shital Niwas is chosen for modeling. The whole building is not taken for the model due to modeling complexity and the interpretation of the results. The result obtained from the North wing 3- D model could be generalized for the global behavior of the whole building. The prime concern is given to the floor rigidity and the long unsupported wall of the building. The following conclusions are made after the analysis.

1. In order to simplify the analysis and interpreting the results, the whole building is not taken for the study. Only the North wing of the building, encircle in figure 5.3, is chosen for the study.

2. The seismic response of such building highly depends on the floor rigidity, geometry of the longitudinal and transverse wall and their position. URM wall mainly failed by the excessive out-of-plane deformation.
3. The North wing of the old traditional building Shital Niwas, is collapsed with the excessive deformation of the outer wall.
4. For such building, having long in length and narrow in width, the number of the existing cross walls are not sufficient form the seismic consideration.
5. The large sizes of the rooms without cross walls are the main draw backs of the building.
6. The in-plane wall, which is parallel to the seismic excitation, of the building resists the out-of–plane failure. Figure 5.18 shows the deformed shape of the building.
7. For the full scale modeling and analysis of such old traditional building, the effect of the existing floor should be considered. It may be conservative to exclude the effect of the timber floor for the long and unsupported URM wall of the building.
8. The global behavior of such building is entirely depends on the semi rigid or loosely connected timber floor and unreinforced masonry wall.
9. It is difficult to model the semi rigid joint of the timber joist and the masonry wall without the test results.
10. The fundamental time period of the existing form of the building is reduced by replacing the rigid reinforced concrete floor or increasing the existing floor rigidity.

Table 5.1: Comparisons of the fundamental time period of the building having timber and rigid floor for first five modes.

S.N.	Description of the Model	Fundamental Time Period (seconds)				
		Mode 1	Mode 2	Mode 3	Mode 4	Mode 5
1	Building having existing timber floor	0.35182	0.3328	0.29491	0.24162	0.19725
2	Building having rigid floor	0.22039	0.17026	0.13025	0.07385	0.06195

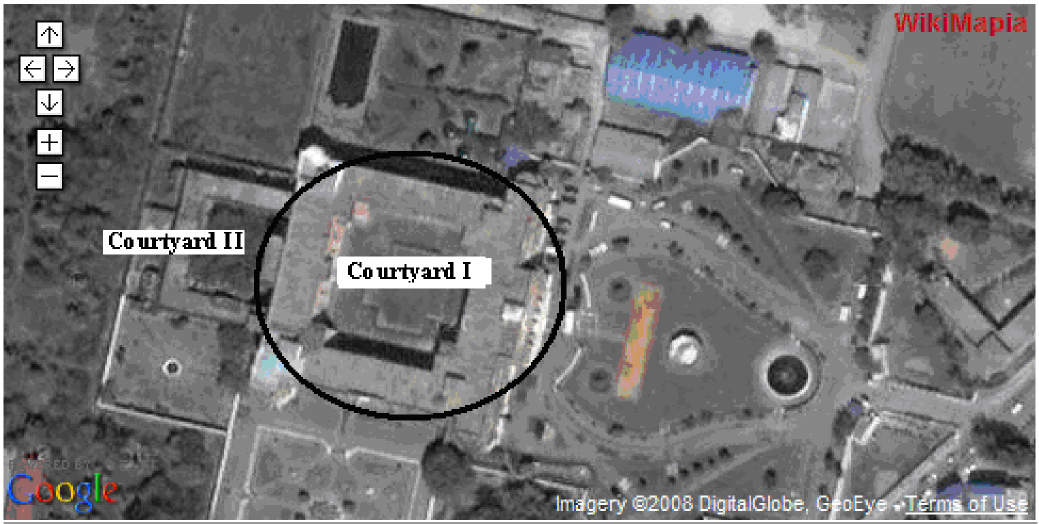


Figure 5.1: Aerial view of the Shital Niwas

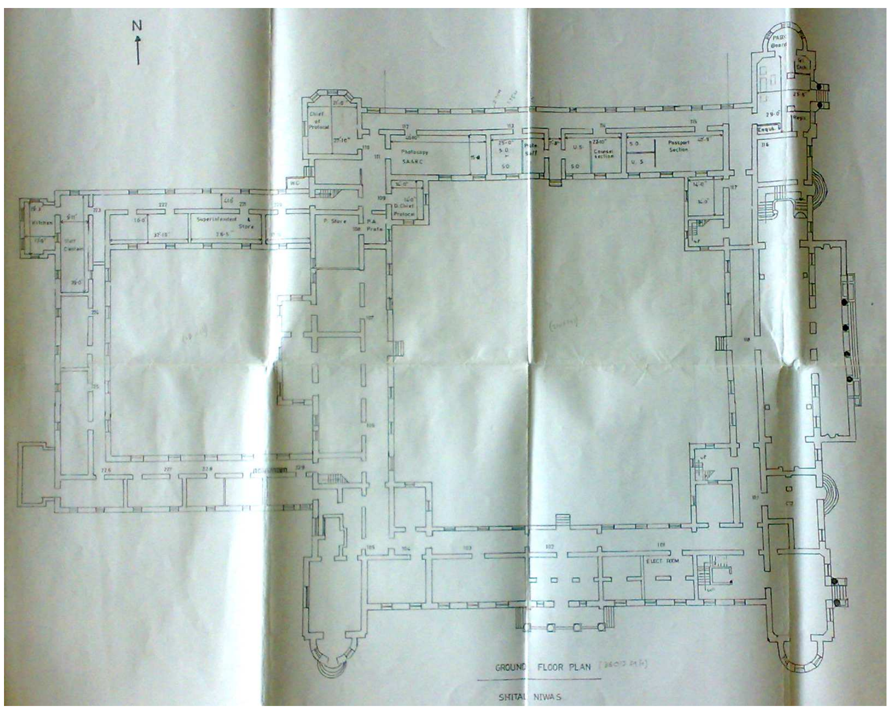


Figure 5.2: Ground floor plan of Shital Niwas

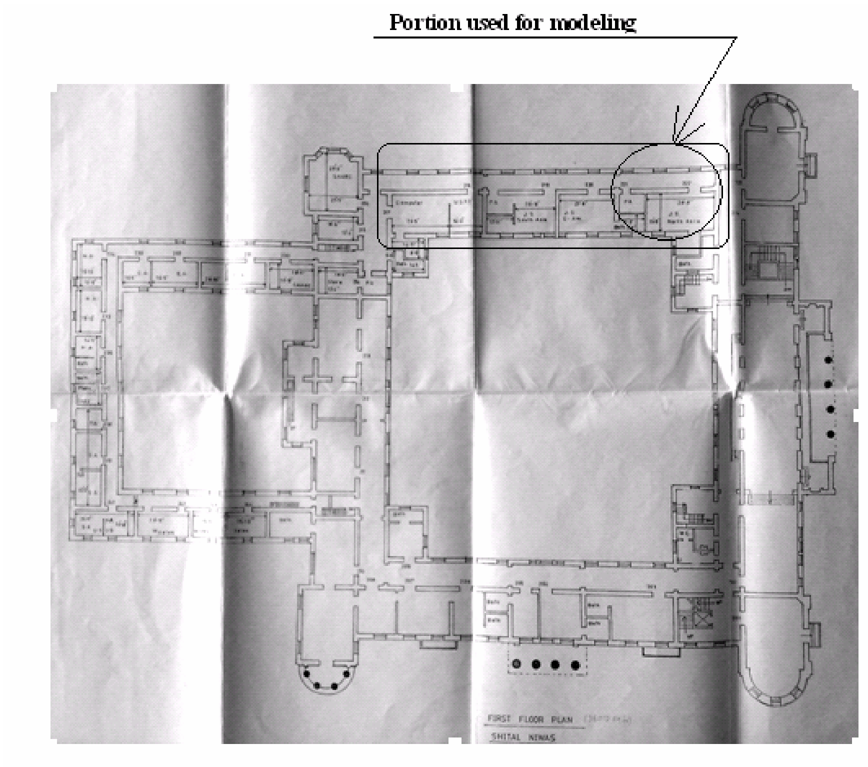


Figure 5.3: First floor plan of the Shital Niwas

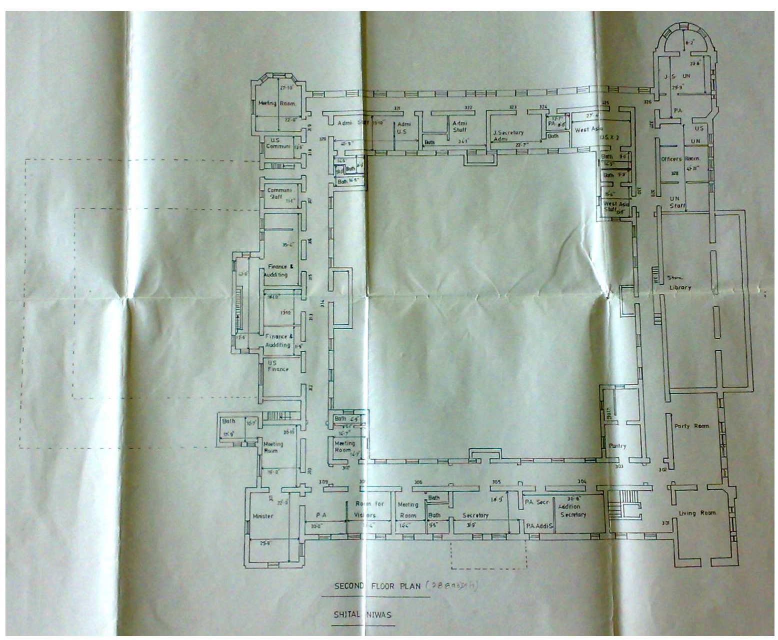


Figure 5.4: Second floor plan of Shital Niwas

(Source: Ministry of Foreign affairs, Government of Nepal)

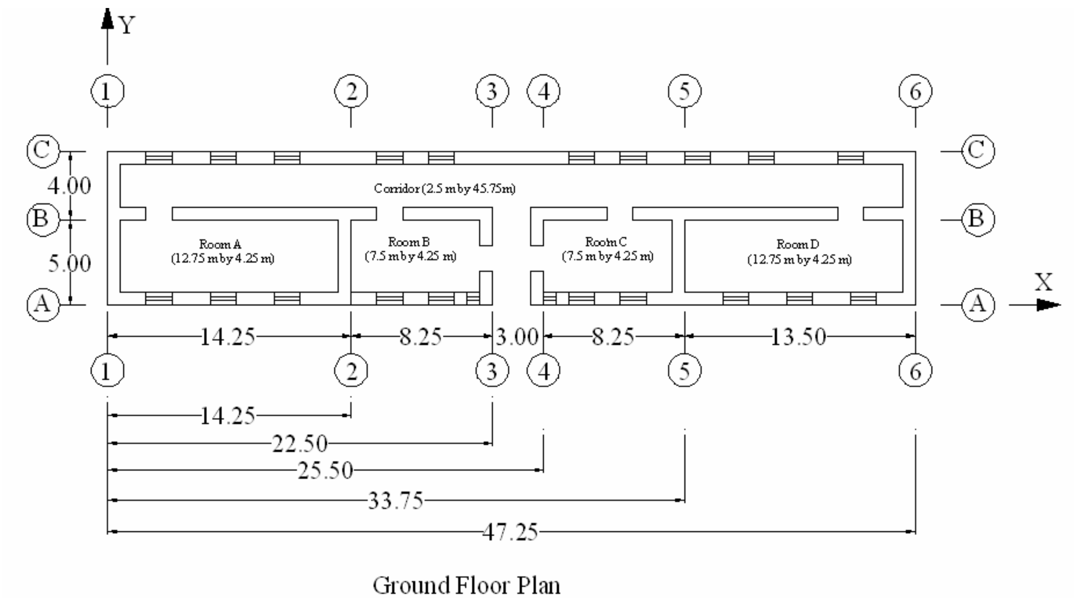


Figure 5.5: Ground floor plan of the North Wing of the Shital Niwas

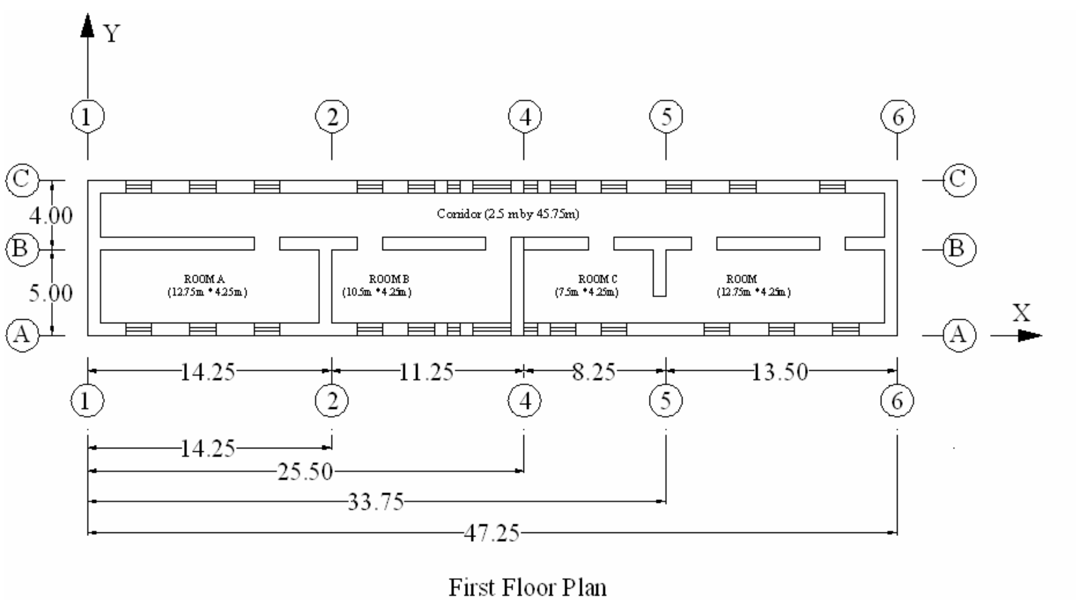


Figure 5.6: First floor plan of the North Wing of the Shital Niwas

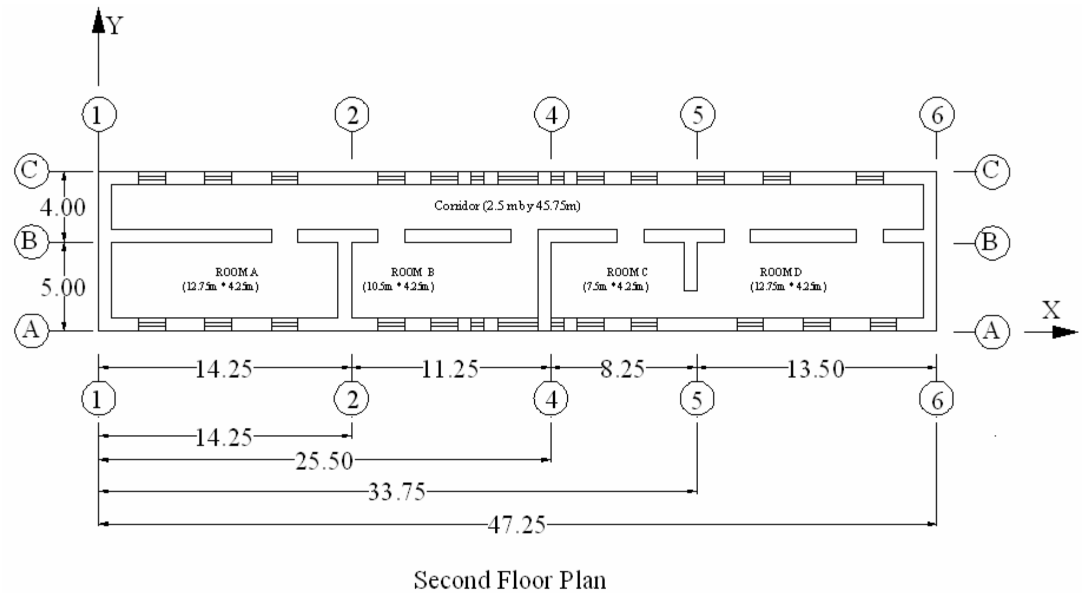


Figure 5.7: Second floor plan of the North Wing of the Shital Niwas

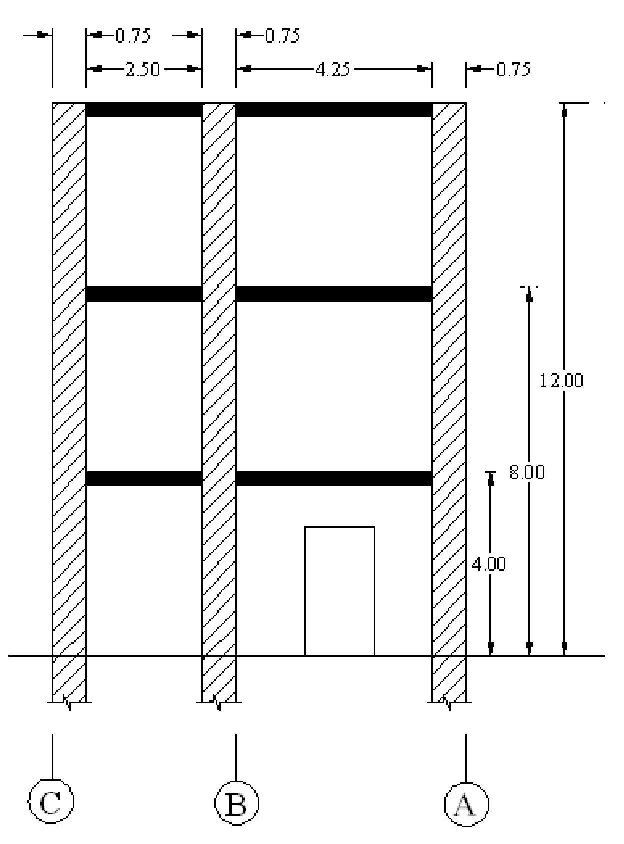


Figure 5.8: Section at Gird 3-3 of the North Wing of the Shital Niwas

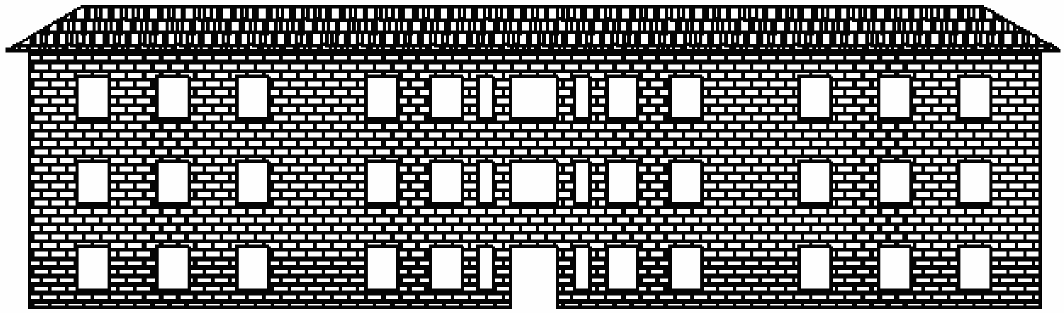


Figure 5.9: South elevation of the model building

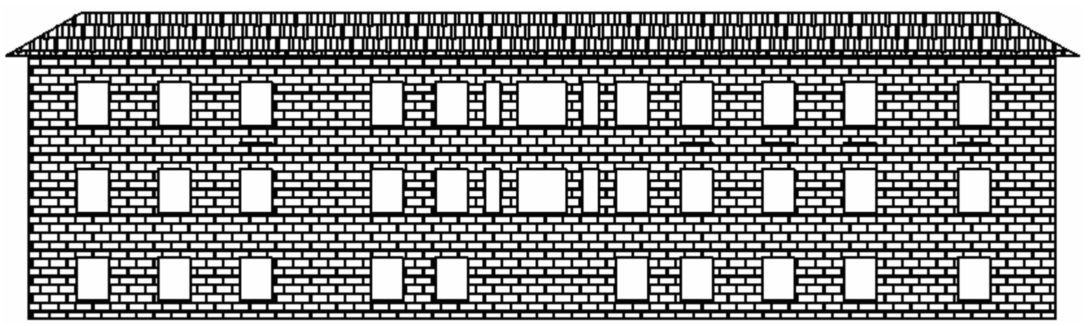


Figure 5.10: North elevation of the model building

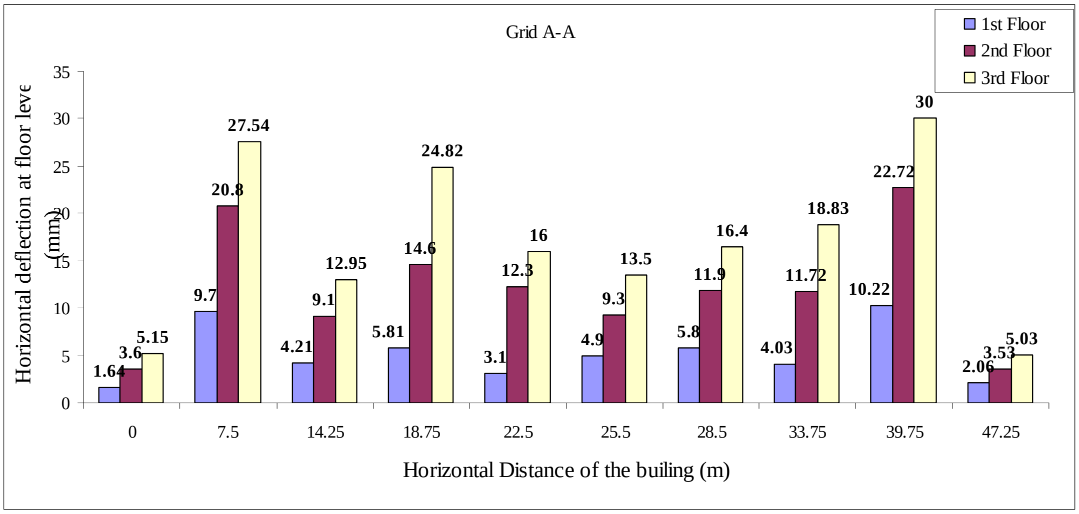


Figure 5.11: Horizontal displacements (out-of- plane) of the longitudinal wall along the Grid A-A at the floor level of the three dimensional building having the timber floor. Earthquake force applied along the y-direction

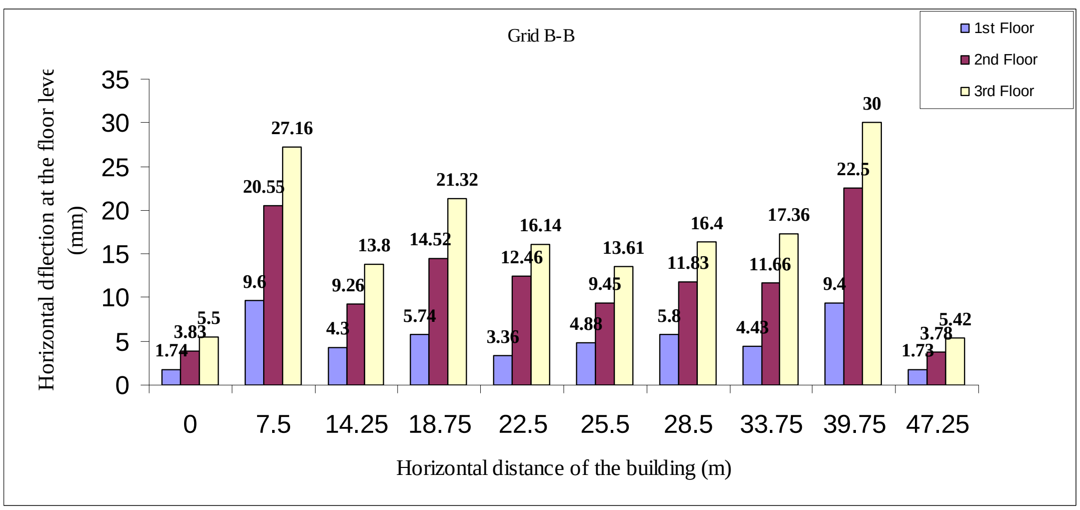


Figure 5.12: Horizontal displacements (out-of- plane) of the longitudinal wall along the Grid B-B at the floor level of the three dimensional building having the timber floor. Earthquake force applied along the y-direction

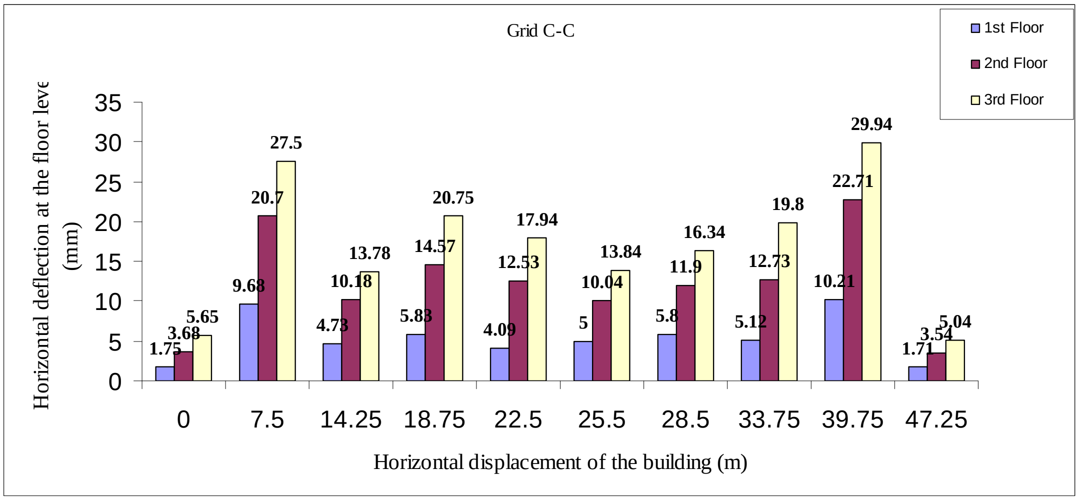


Figure 5.13: Horizontal displacement (out-of- plane) of the longitudinal wall along the Grid C-C at the floor level of the three dimensional building having the timber floor. Earthquake force applied along the y-direction.

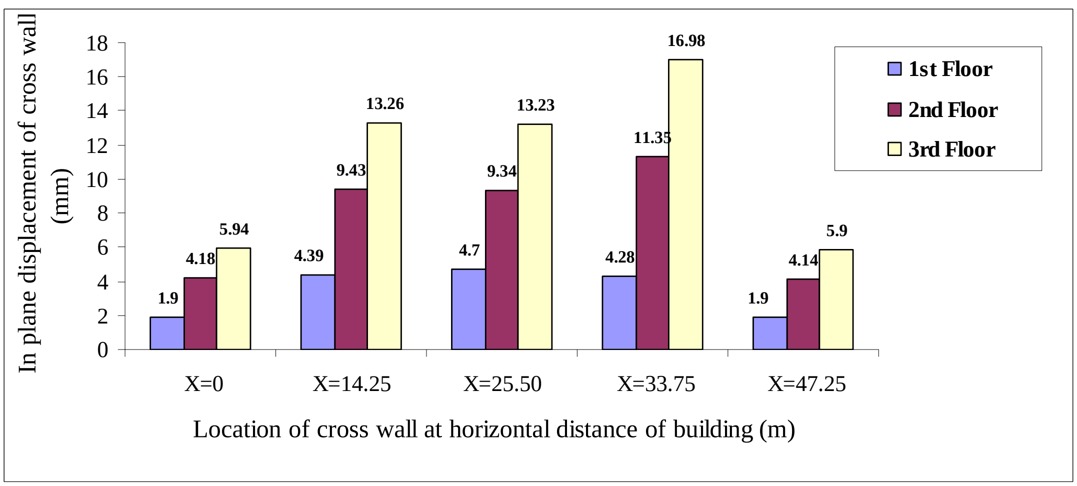


Figure 5.14: In-plane displacement of the transverse wall at the story level of the three dimensional building having the timber floor at different location of the horizontal distance, shown in figure 5.5 to 5.7. Earthquake force applied along the y-direction

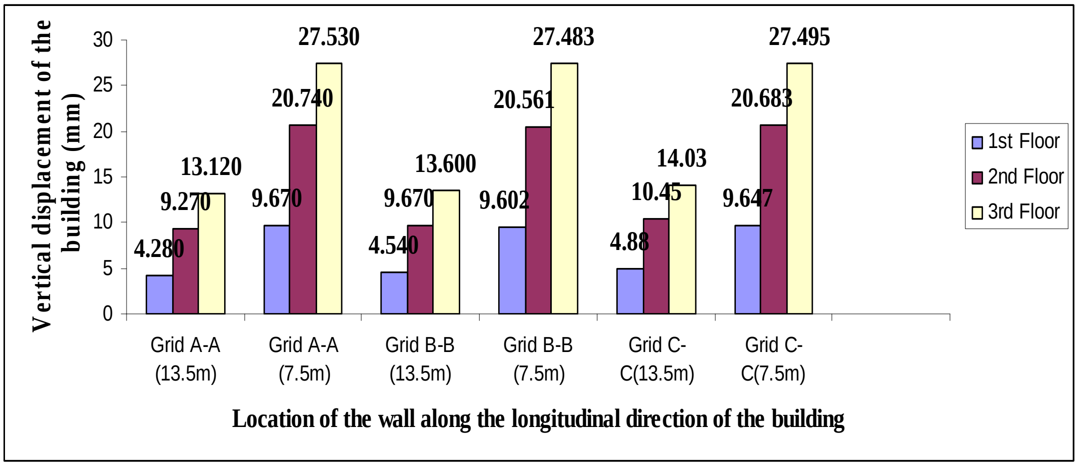


Figure 5.15: Vertical deflection of the building due to longitudinal wall along Grid A-A at different location shown in legend, along the X-direction of the building, at the story level of the three dimensional building having the timber floor. Earthquake force applied along the y-direction

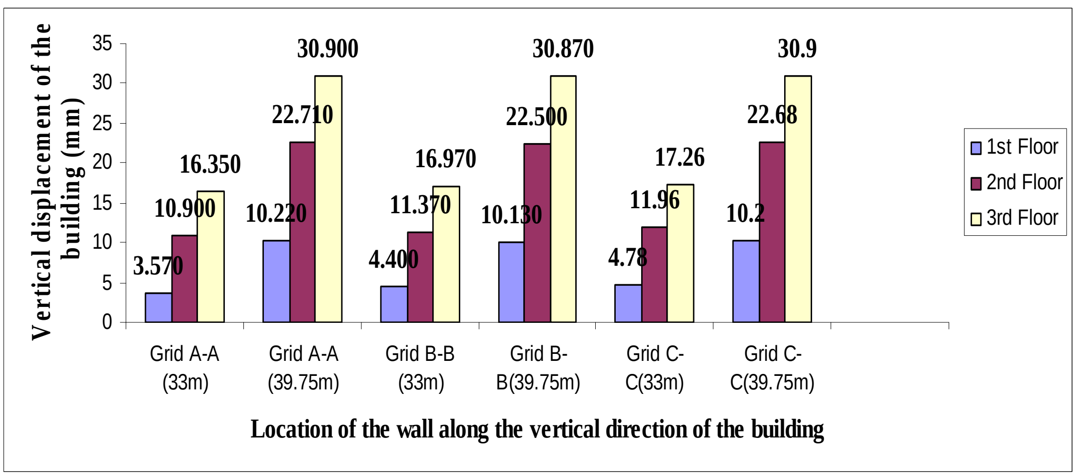


Figure 5.16: Vertical deflection of the building due to longitudinal wall B at different location shown in legend, along the y-direction of the building, at the story level of the three dimensional building having the timber floor. Earthquake force applied along the y-direction

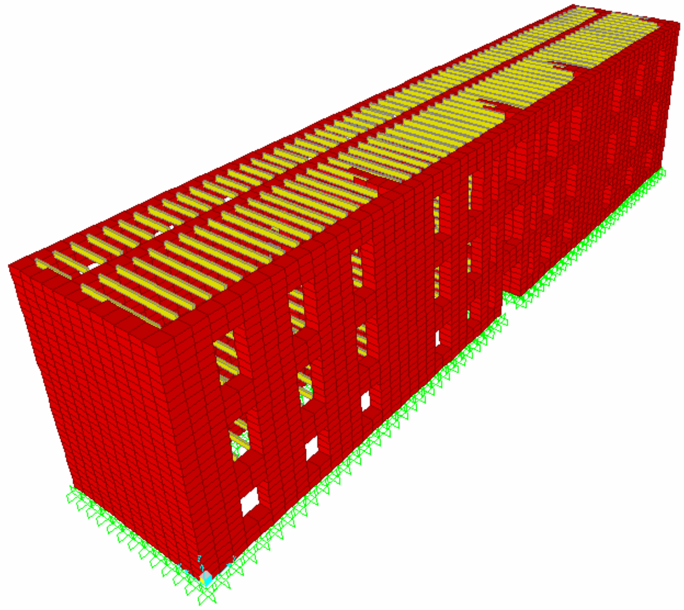


Figure 5.17: Three dimensional model of the studied building with timber floor diaphragm.

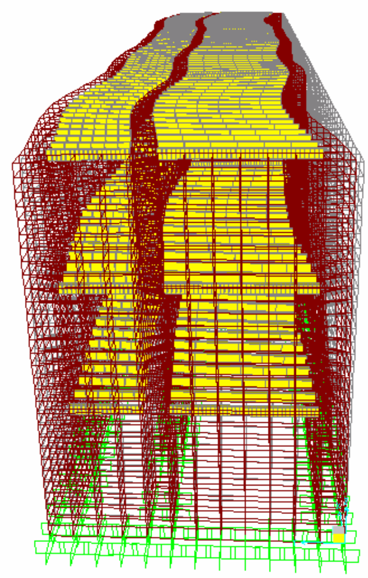


Figure 5.18: Deformed shape of the three dimensional unreinforced masonry building, North wing of the Shital Niwas, with timber floor and reinforced concrete floor, subjected to seismic loading.

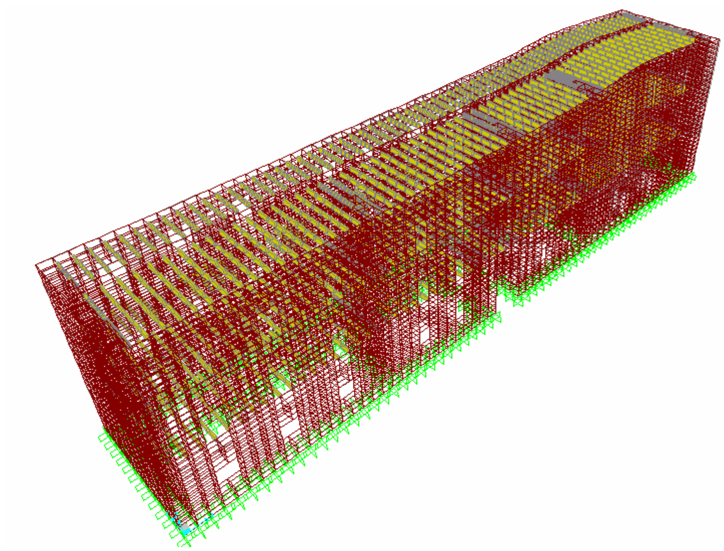


Figure 5.19 (a): Mode 1 (Time period =0.35182 sec)

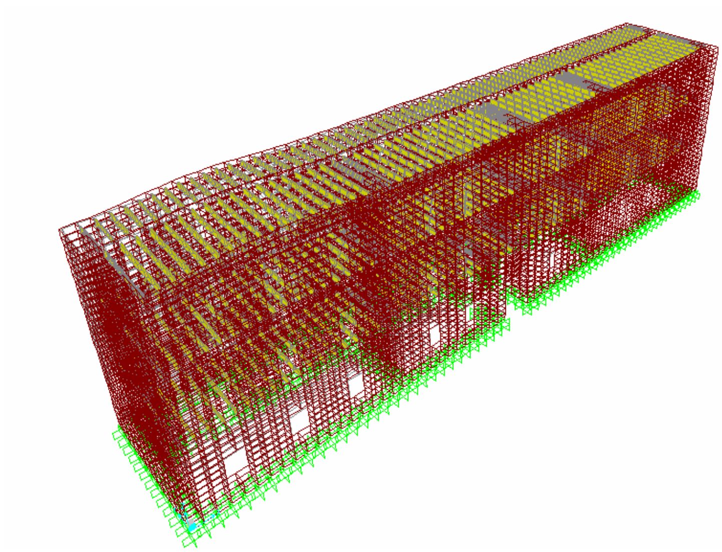


Figure 5.19 (b): Mode 2 (Time period = 0.3328sec)

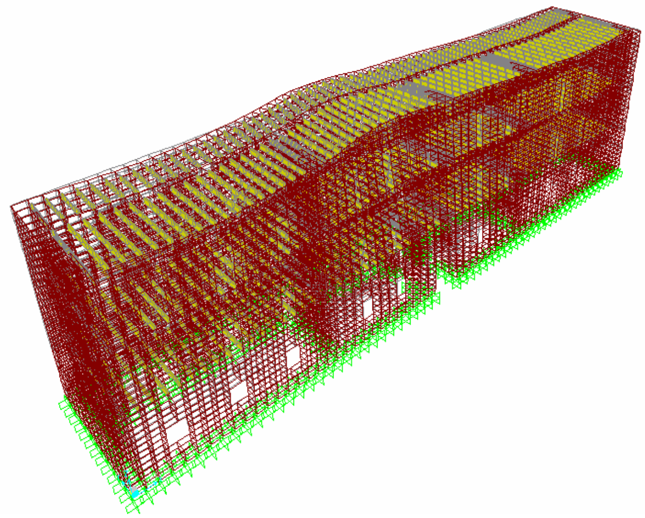


Figure 5.19 (c): Mode 3 (Time period =0.29491 sec)

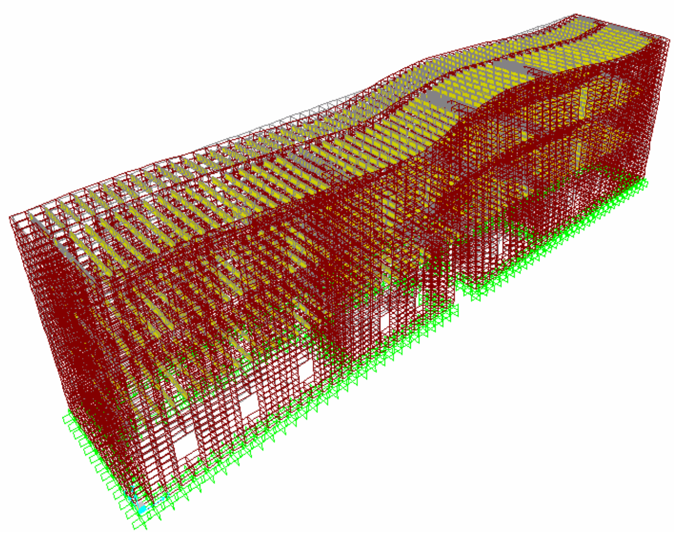


Figure 5.19 (d): Mode 4 (Time period = 0.24162sec)

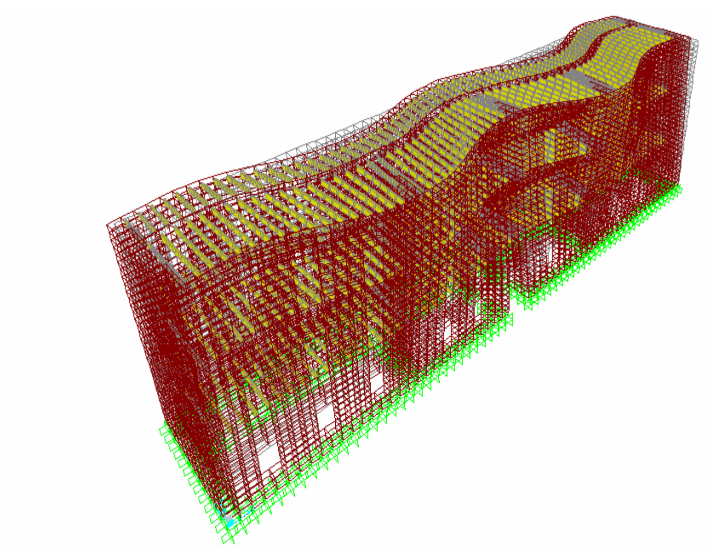


Figure 5.19 (e): Mode 5 (Time period = 0.19725sec)

Figure 5.19: First five mode shapes of the building having the timber floor from figure 5.19 (a) to figure 5.19 (e)

CHAPTER 6

CONCLUSION AND RECOMMENDATION

6.1 General

The study is focused on the old traditional building constructed during the Rana regime (1850 AD -1950 AD) in Nepal. In order to increase the performance of the building for future earthquake, seismic evaluation of these buildings is essential. Before analyzing the actual structure, a fictitious building is studied in detail in chapter 4. A parametric finite element analysis is carried out in order to reflect the characteristics of the unreinforced masonry (URM) building. The preliminary findings are considered in the real structure. In this study a real building, Shital Niwas is analyzed. After studied, it is found that the existing forms of the buildings are highly vulnerable for future earthquake. The insufficient floor rigidity, the long span unsupported wall in corridor, inadequate number of the cross walls and may be ongoing deteriorated structure elements reduces the overall performance of the building.

6.2 Major Conclusions

The following points are concluded from this study are as follows:

1. A fictitious building having simple plan of 6 m by 3 m of story height of 3 m is analyzed. In order to investigated the characteristics of the unreinforced masonry (URM) building number of parametric analysis is carried out. This includes: (a) the effect of the wall thickness, (b) the effect of the floor rigidity (c) the effect of opening (d) the effect of number of stories and, (e) the effect of the lateral load distribution on different floor condition.

2. The top displacement of the URM building is drastically reduced as the thickness of wall increased. The effect of different floor system on the same structure is also studied for various thickness of wall. In this case, three cases are studied:(1) assuming no floor in building for loosely connected timber joist in the masonry wall,(2) the presence of timber floor, and (3) the presence of the reinforced concrete floor.

3. For the high thickness of the wall the in-plane rigidity of the timber floor is not significant. The displacement of the building is governed by the thick wall. But in case of the low thick wall, like 230 mm and 350 mm thick wall, the in – plane rigidity of the timber floor exists. Beyond the 450 mm thick wall the effect is negligible as indicated in figure 4.3 to 4.6.

4. As the floor rigidity of the URM building is improved by introducing the rigid floor, that is, reinforced concrete floor, the thickness of the wall has no significant effect. As from the figure 4.3 to 4.5, the displacement pattern of the building is like a constant value for different thicknesses wall. It may not be necessary to construct the high thick wall, for low rise building, with rigid floor from seismic point of view.

5. For reducing the excessive out-of – plane deformation of the URM building, the loosely connected timber floor should be replaced by the rigid floor or increase the rigidity of the existing floor.

6. As shown in figure 4.6, for four story building, the top displacement of the building due to the timber floor, no floor and rigid floor are same in case of the thick wall more than 750 mm. Therefore for the higher story building having thick wall, replacing the existing floor with rigid floor may not be good option. The performance of the building should be increased by the proper tying the connection between the existing floors with the masonry wall of the building.

7. The in-plane displacement of the building is depends on the opening of the building rather than the floor rigidity. As the amount of the opening is increased, the floor rigidity decreased and the displacement of the building is control by the thickness of the wall. If the percentage of opening is increased the in-plane displacement is increased greatly. For the larger opening, the manual methods overestimate the response of the building as compare to the finite element method.

8. For the distributed mass along the height of the masonry structure having flexible floor the lateral force should be distributed along the height of the building instead of applying at the centre of mass of floor. There are two types

of lateral loading are discussed one is uniform distribution and another is inverse triangular loading. The uniform loading distribution gives the higher stiffness probably due to the distribution of seismic forces along the structure.

9. For a real structure, Shital Niwas, the old traditional building constructed on 1923 AD is taken for the case study. It was extensively damaged during the 1934 AD earthquake. After that earthquake, the building was immediately constructed without considering the seismic effect. For the simplicity of modeling and analysis, the North wing of the building is taken and generalized the results for the global behavior of the whole building.

10. From the analysis, it is concluded that the collapses of the outer wall of the North Wing of the Shital Niwas is due to the excessive out-of-plane deformation.

11. Inadequate number of the existing cross wall which makes the large size of the rooms, long unsupported walls and loosely connected existing timber floor are the main drawbacks of the existing form of building.

12. Due to the less number of cross wall connected with longitudinal walls and also the longitudinal direction of the building is more than five times larger than the transverse direction, the whole building cannot acts as a single unit and the longitudinal and transverse walls behaves independently and leads to failure during the earthquake.

13. For modeling the old traditional URM building, the loosely connected existing timber floor with masonry wall should be considered. In the analysis of the fictitious building it is found that, it has negligible effects for the high thick wall. But in the large scale analysis, it has considerable effects. Therefore, the existing timber floor should be analyzed with proper behavior.

14. The global behavior of such building is entirely depends on the semi rigid or loosely connected timber floor and unreinforced masonry wall. But, it is difficult to model the semi rigid joint of the timber joist and the masonry wall without the test results.

15. The fundamental time period of the building depends upon the rigidity of the floor. Diaphragm flexibility increases the fundamental time period of the building as compare to the rigid floor diaphragm. The fundamental time period for the first five modes is shown in table 5.1, for different floor rigidity.

6.3 Future Works

The following works are carried out for future works:

1. The whole building can be modeled for the analysis in order to retrofitting or strengthening the building for possible future earthquakes.

2. The different joint models can be carried out for modeling the timber floor and connection between the timber floors with masonry wall.

3. Necessary strengthening and retrofitting works should study for future works.

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