SEISMIC PERFORMANCE OF TRADATIONAL UNREINFORCED MASONRY BUILDING IN NEPAL

Shyam Sundar Khadka

Department of Civil & Geomatics Engineering, Kathmandu University

Corresponding address: sskhadka@ku.edu.np

ABSTRACT

The paper focused on the old traditional buildings, which were constructed more than 100 years ago. 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. 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. A finite element analysis is carried out in order to reflect the characteristics of the unreinforced masonry (URM) building. A commercial software SAP2000 is used for the analysis. It is found that the existing forms of the buildings are highly vulnerable for future earthquake. 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. 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. 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.

Keywords: Unreinforced masonry building, seismic performance, floor rigidity, thick wall, unsupported wall, openings.

INTRODUCTION

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. 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.

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 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;

MASONRY BUILDING IN NEPAL (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. 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. 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 Nivas, which at present accommodates House of President, was re-built after the damage in 1934 AD earthquake. The original old Sital Nivas was built in 1923 AD as shown in figure 1. In 1934 AD earthquake, it was extensively damaged and again reconstructed on the same ground immediately after earthquake. The present condition of Sital Nivas palace is shown in figure 2. 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 Nivas palace presented in figure 3 to illustrate the typical structural plan of the palaces.

Figure 1 Shital Nivas before 1934 AD earthquake

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.

Figure 2 Present condition of Shital Nivas

Structural components of the building
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 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.

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.

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.

Foundation

Generally, for old traditional building the spread foundation is commonly adopted. Foundation may separate 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.

Case study: Shital Nivas

The old Shital Nivas 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 3, without any cross wall and having the maximum number of openings, which is seismically vulnerable. The Shital Nivas 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.
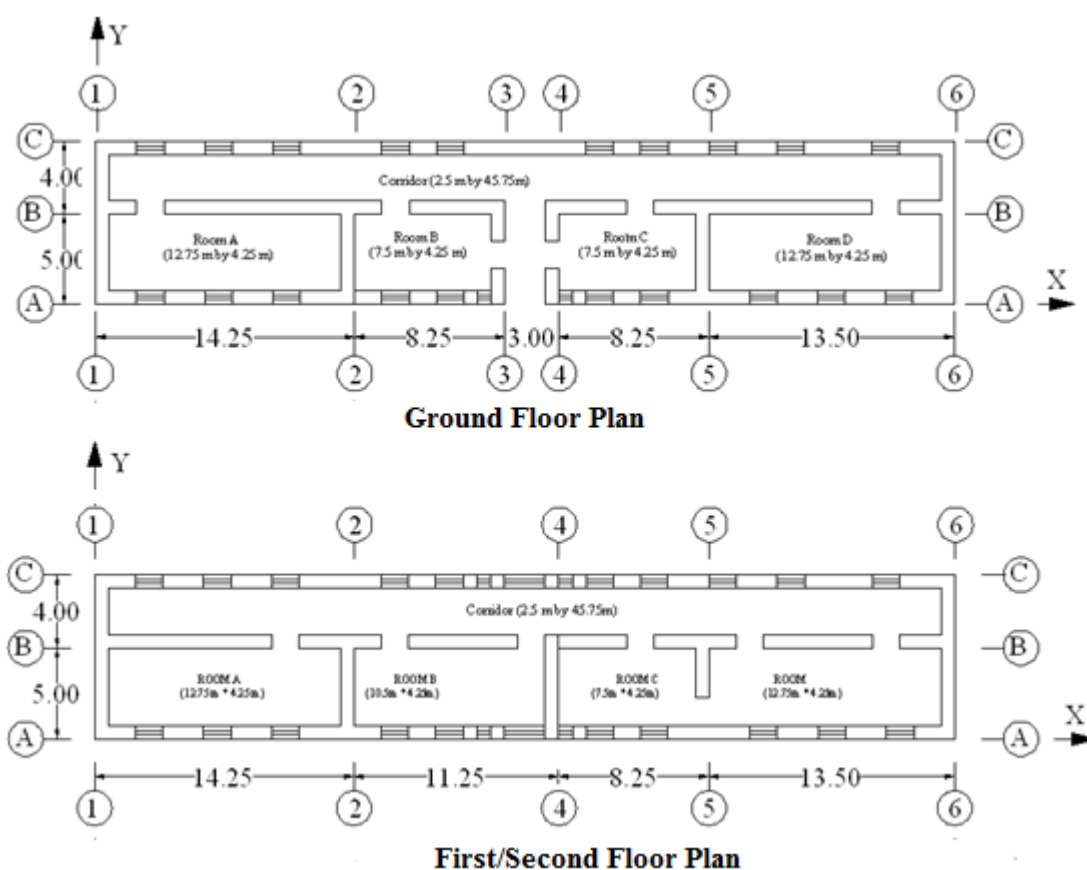
Figure 3 Typical floor plans of the building

MODELING AND ANALYSIS

For modeling the building, only a North wing of the Shital Nivas is taken. The floor of the studied part is shown in fig 3.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 1 [5]. 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. For modeling, the thick masonry wall is modeled with eight-nodded solid element of SAP2000. 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 [12]. 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 4. 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.

Table1 Mechanical properties of the materials (Mean Values)

Materials	Youngs Modulus (kN/m^2)	Poisson ratio	Mass Density(t/m^3)	Compressive Strength (Mpa)	Tensile Strength (Mpa)
Wood	9000 000	0.3	0.5	24	46.5
Brick Masonry	1708 000	0.15	2.1	4	0.4

The model is analyzed by the Seismic Coefficient method, 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.
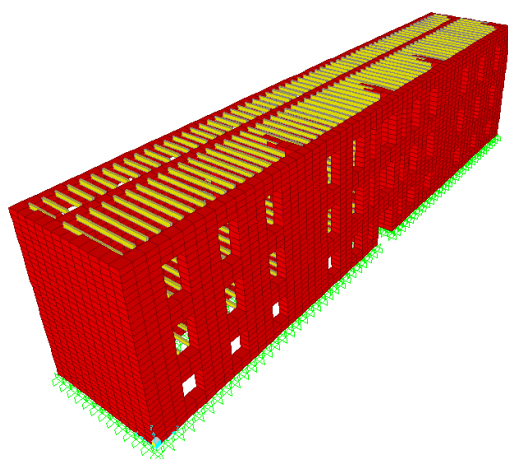

Figure 4 3D Finite element model of the studied building

RESULTS AND DISCUSSION

In this chapter, actual old traditional building, Shital Nivas, is taken as a case study. The portion of building used for the analysis is shown Figure 3. In finite element model, the prime concern is given to the floor rigidity and the long span wall. 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 3 , 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 3.

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 3. 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 3, 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.
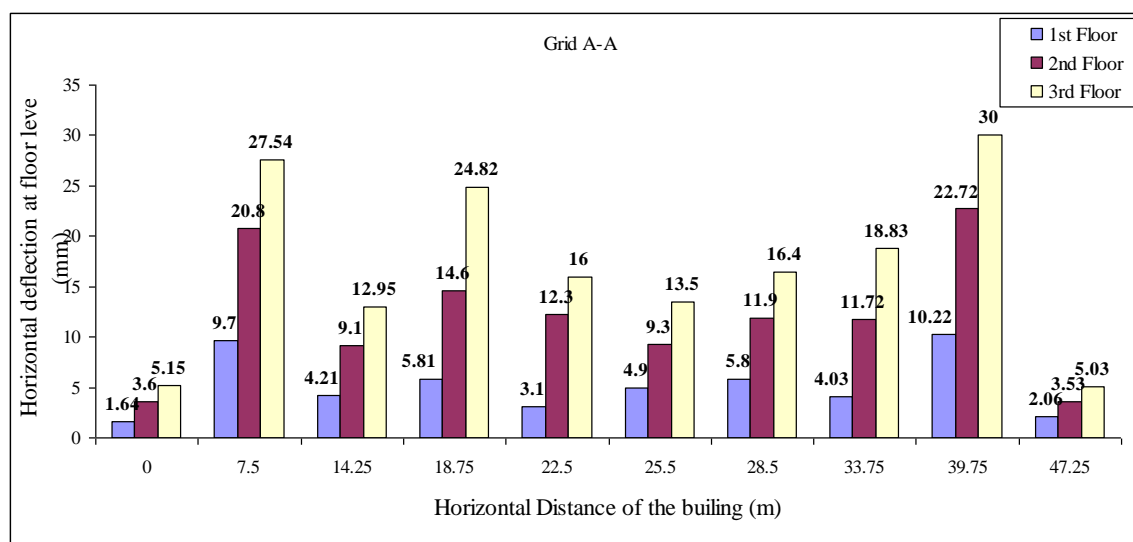
Figure 5 Horizontal displacements (out-of- plane) of the longitudinal wall along the Grid A-A at the floor level of the three dimensional building having the timber floor.

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 to 7. 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 to 7. 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 to 7. 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.
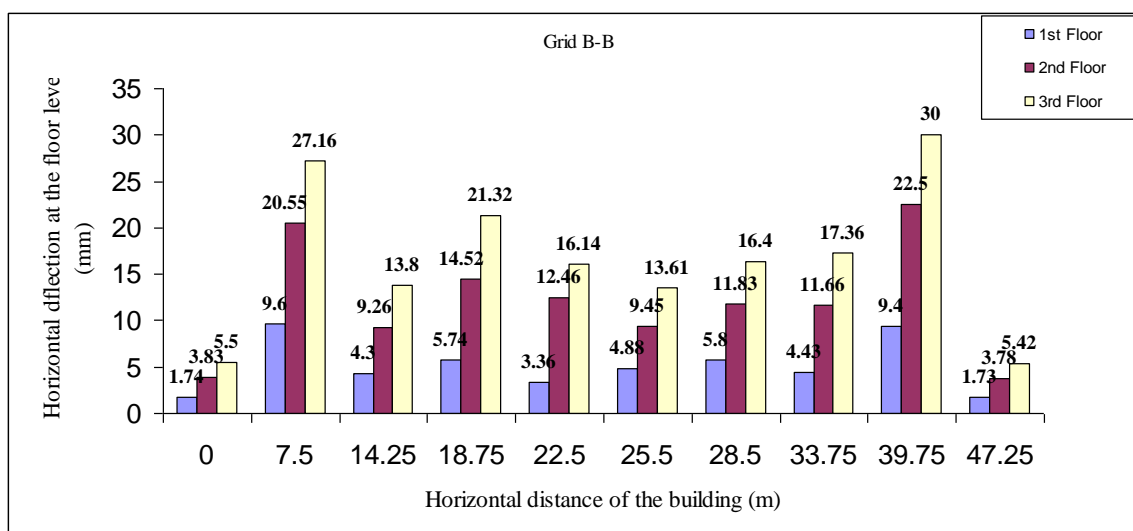
Figure 6 Horizontal displacements (out-of- plane) of the longitudinal wall along the Grid B-B at the floor level of the three dimensional building having the timber floor.

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.
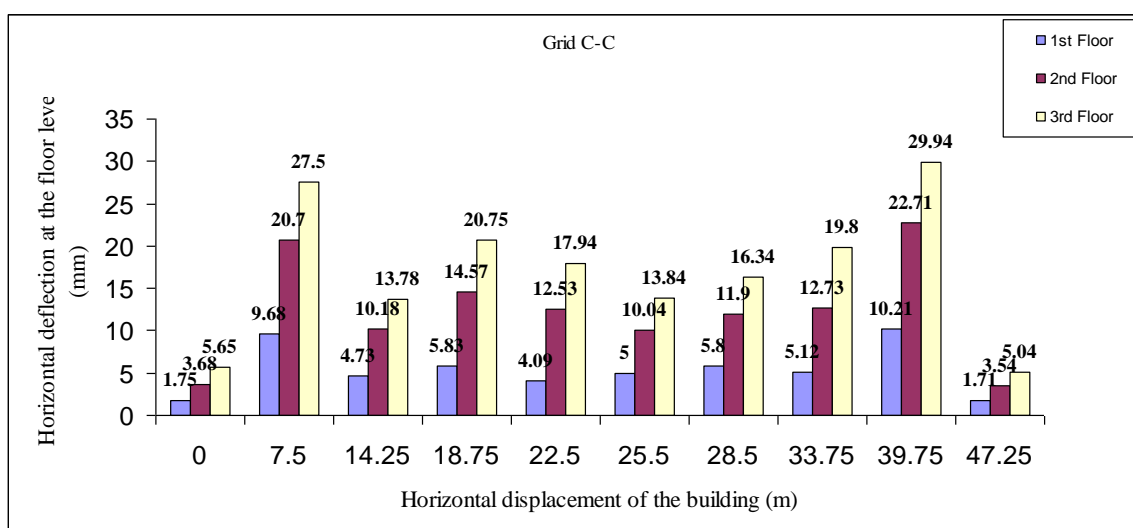

Figure 7 Horizontal displacement (out-of- plane) of the longitudinal wall along the Grid C-C at the floor level of the three dimensional building having the timber floor.

24

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 8. 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 8.

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.
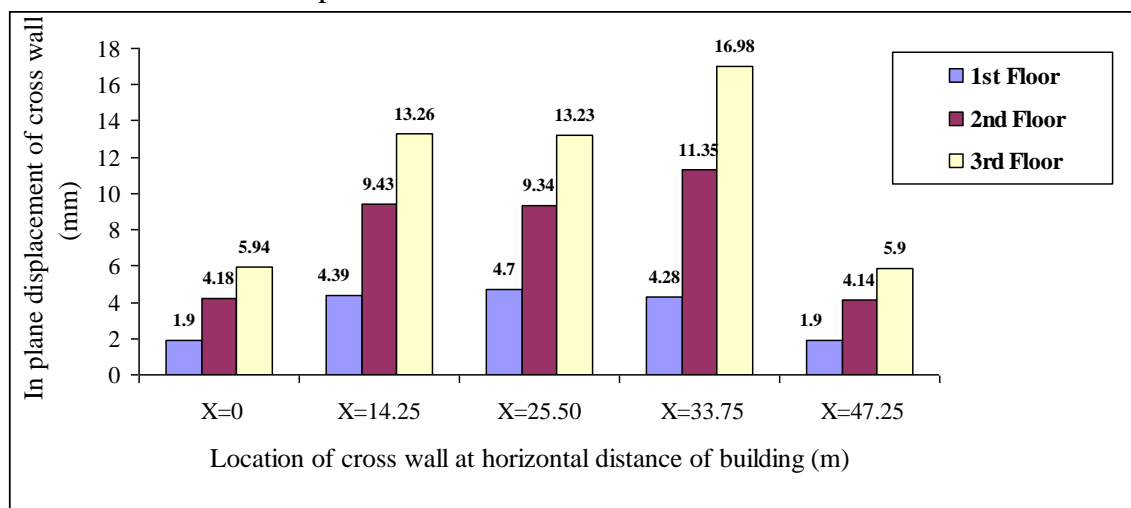

Figure 8 In-plane displacement of the transverse wall at the story level of the three dimensional building having the timber floor at different location of the horizontal distance, shown in figure 5.5 to 5.7. Earthquake force applied along the y-direction
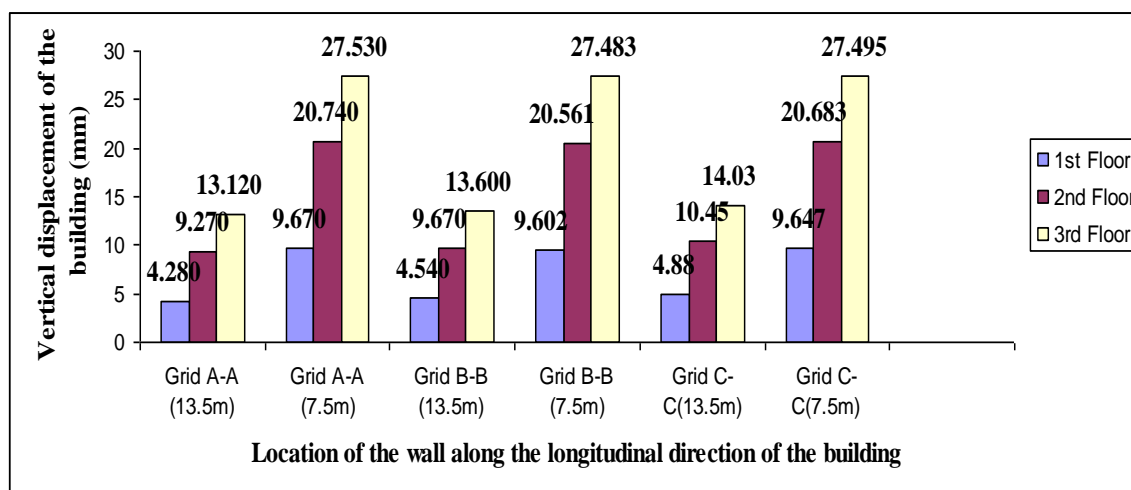
25

Figure 9 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 9 and 10 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.
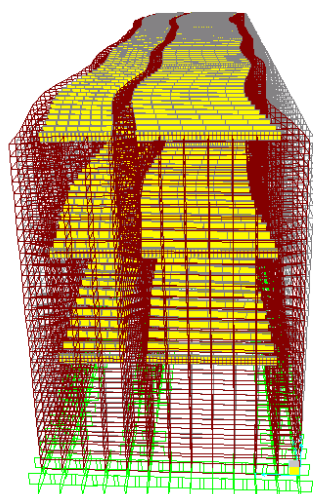

26

Figure 11 Deformed shape of the three dimensional unreinforced masonry building, North wing of the Shital Nivas, with timber floor subjected to seismic loading.

CONCLUSION

For assessing the seismic performance of the old traditional building Shital Nivas is taken as a case study and a North wing of the Shital Nivas 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.

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, 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.

REFERENCES

[1] Cardoso R, Lopes M & Bento R Seismic evaluation of old masonry building. Part I: method and description and application to a case study, *Engineering Structures 27*, (2005)2024.

[2] Giordano A, Mele E & Luca, A Modeling of historical masonry structures: comparision of different approaches through a case study, *Engineering Structures 24* (2002)1057.

[3] Kappos A J, et al. Evaluation of simplified models for lateral load analysis of unreinforced masonry buildings, *Journal of structural Engineering*, (2002) Vol.128, no.7 July

[4] Magenes G & Calvi G M, In-plane seismic response of brick masonry walls, *Earthquake Engineering and Structural Dynamics*, (1997) vol.26, (1997)1091.

[5] Makarios T & Demosthenous M, Seismic response of the traditional building of Lefkas Island, Greece , *Engineering Structures 28* (2006) 264.

[6] Page AW, Finite Element Model For Masonry, *Proceeding of American Society of Civil Engineers*, (1978), Vol.104 ,No ST8 August pp 1267.

[7] Mehrabi A B & Shing P B, Finite Element Modeling of Masonry- Infilled RC Frames , *Journal of the structural Engineering* (1997) Vol.123, No.5 May pp 604.

[8] Lotfi H R & Shing P B, Interface Model Applied to fractures of Masonry Structure, *Journal of the structural Engineering*(1994), Vol.120, No.1 January pp 63

[9] Tenga- Colunga A & Abrams D P, Seismic Behavior of structures with flexible diaphragms, *Journal of the structural Engineering* (1992) Vol.122, No.4 April

[10] Bruneau, M, Seismic Performance of Unreinforced masonry Building, *Journal of the structural Engineering* (1994)Vol.120, No.1

[11] Yi T, Moon F L, Leon R T & Khan F L, Lateral Load test on a two- storey unreinforced masonry building, *Journal of the structural Engineering* (2006) Vol.132, No.5 May

[12] Ramos L F & Lourenco P B, Modeling and Vulnerability of historical city centers in seismic areas: a case study in, *Engineering Structures 26* (2004) 1295.