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Parametric Study and Dynamic Analysis of a Historical Masonry Building of Kathmandu

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A historical brick masonry house located in Kathmandu World Heritage site was modeled by FEM and analyzed for various earthquake ground motions. Bricks and interfaces between the bricks were modeled as solid and joint elements respectively, and non linear dynamic analysis of two models - existing and modified were run to satisfying the famous Mohr-Coulomb failure criterion. The result showed that the existing masonry building was found very weak on all kinds of earthquake loadings and modified building by wooden frame can reduce the seismic force significantly and demonstrates a good strengthening choice for those kinds of constructions.

Key Words: *brick masonry, finite element analysis, joint elements, Kathmandu*

1. Introduction

Kathmandu, the capital city of Nepal is located in one of the world's most active seismic zone, but most of the buildings in this city are not capable to resist the earthquake as they are not designed for the seismic forces. The existing buildings are of two categories – old ones are brick masonry constructions and the new ones are reinforced concrete frame structures. It is obvious that the old buildings were not designed to resist the earthquake loads because of the lack of knowledge and regulations. Even in the present days, most of the buildings are designed traditionally and the constructions are not following the building codes for the seismic requirements. Reinforced concrete structures with brick infill walls are the most common in constructions. However, just using frame structures without proper design cannot be seismic resistant and these types of structures add the risk only.

Brick masonry temples, royal palace and residential houses with beautiful architectural appearances are the main identity of Kathmandu valley. They were constructed in Malla period, about 300 years ago; these structures have become one of the main tourist attraction sources from the time beginning. Historical records show that many earthquakes occurred in or in the vicinity of Kathmandu. Written evidences of earthquake occurrences and damages had been described since 1223 (Pant 2000). Big earthquakes hit the Kathmandu Valley in 1255 and 1344 which killed one third of the population at that time and damaged many temples and houses. Since then, many earthquakes have been reported and the most damaging is 1934 Nepal Bihar earthquake which killed more than 8000 people in Nepal (Rana 1935). Recently, as revealed from Chinese literature, great earthquake occurred in 1408 in the Western Nepal and it had damaged five hundred kilometer length of Himalaya (Ambraseys and Jackson 2003). If the age of masonry temples and history of earthquakes are compared, most of the historical temples should have passed through severe earthquake ground motions. They should have sustained damages and renovated afterward. From the evidences, we can see some traditional techniques for earthquake resistance such as square shaped buildings, use of wooden elements along with the wall to discontinue vertical joints. However, these simple techniques used for the construction without proper knowledge are not sufficient to resist severe earthquake motion which is expected in the near future. Thus, the strengthening of these houses against probable earthquake is most important task under the risk management. So, as a part of effort to access the vulnerability and suggesting the mitigation measure for cultural heritage, a historical masonry house (Fig. 1) located in Lalitpur Sub Metropolitan of Kathmandu City is selected as a model study. In the study, firstly, the required parameters are investigated from non-destructive tests and then numerical model is prepared and analyzed by using Finite Element Method (FEM) providing various earthquake ground motion inputs which are described in detail in below sections.

2. Numerical Methodology

Masonry behaves distinct directional properties due to the interfaces between the elements. The large number of influencing factors such as interior voids, anisotropy of bricks, dimension of bricks and joints, arrangement of bed and head joints and quality of workmanship make the numerical model of masonry wall very complex. Limited number of variables that are used in the numerical model cannot catch the actual behavior of brick walls due to its variety of properties. Basically, two methods are used for analyzing masonry houses – one, distinct element method (DEM) which considers the brick units as non-deformable solids and their movements are evaluated through equation of motions and the other, FEM which considers the masonry wall as a deformable element. FEM analysis carry out from very simple method such as considering masonry as a single phase material to very complex method such as considering each element and joint separately, has been widely used and become a well accepted tool.

Fig. 1 Historical brick masonry house

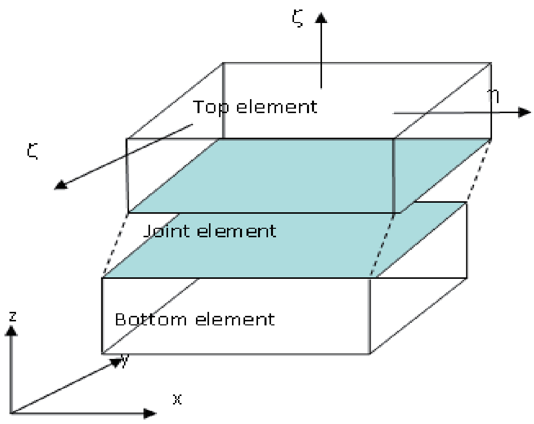

Fig 2 Formulation of solid and joint elements

Since the brick masonry walls are composed of brick units and joints (Fig. 2), they behave non linearly governing the entire deformation phenomenon by weak joints. The ordinary FEM which is based on continuum mechanics cannot be applicable for such kind of problems. Thus, the modified FEM considering discontinuities has been used in the field of rock mechanics (Zienkiewicz 1970) and recently, it has been used in brick and stone masonry (Tzamtzis and Nath 1992, Tzamtzis and Asteris 2004, Senthivel et al. 2006) to simulate time dependent sliding and separation along the mortar joints. This concept has also been applied to investigate the effectiveness of wooden beams in dry stone masonry houses (Parajuli et al. 2008). Thus, similar idea of modeling the brick units as solid elements and interfaces between them with zero thickness of joint elements is employed here. However, there is large number of bricks in a house and modeling of each brick separately is very complex and almost impossible. Thus simplified numerical model is developed making equivalent eight node elastic solid elements for brick wall blocks and eight node joint elements (Tzamtzis and Nath 1992) for interfaces between brick elements.

3. Parameters

For FEM modeling, the parameters such as density, modulus of elasticity, Poisson ratio, spring constants for joints along normal and shear directions are very much essential. To get precise analysis results, exact parameter inputs are most important factors. The model building is a very old one and has been renovated many times. Each wall has different strength and even within a wall; strength varies significantly at different locations because of repairs and maintenance at different times. The usual way to find out the material properties is to test the wall. There are two kinds of testing- destructive and non-destructive. Since, it is heritage building any kind of destructive test is not allowed. Thus, non destructive test by elastic wave measurement was done to investigate the properties of the walls.

3.1 Measurement by Pocket AE

Pocket AE is a handheld instrument for acoustic emission testing and performs advanced wave-form based signal acquisition and processing. It records the elastic waves produced by a sudden redistribution of stresses in a material due to external forces such as pressure, load, temperature etc.; it releases energy in the form of stress waves and propagates through the surfaces. It has two channels which are set two sides of wall (Fig. 3), then, stress waves are generated by hitting the wall near one sensor by a small spherical ball attached hammer and then the generated stress waves are recorded by the sensor on opposite side. Primary (P) wave velocity is calculated from thickness of wall divided by the difference in wave arrival time between two sensors.

In Fig. 3, Pocket AE instrument (left) and sensor arrangement (right) are shown. At first, the wall was hit by a hammer near by the sensor location and the wave arrival times for both sensors were noted. Since, there are many joints in the wall, the arrival times were differed by the locations where the sensors had been put subsequently, calculated velocities also varied in wide ranges. Thus, series of measurements were taken at various locations in the same wall and in different walls with different thickness. P wave velocities are calculated from thickness divided by difference between arrival time of two sensors. From average value of primary wave velocity, unit weight (measured 19KN/m^3), and Poison's ratio (assumed 0.2), modulus of elasticity (E) and shear (S) wave velocity (V_s) of the material are calculated, and shown in Table 1. Primary and shear wave velocities for three kinds of walls are shown in Table 1. Shear wave velocity for single brick wall (23cm long) is found 1531m/sec while the similar thickness wall which has one joint has shear wave velocity nearly half. 11cm wall has has very close P wave velocity with single brick because it does not have joints through cross section. As the wall thickness increases, number of joints in the wall increases and primary and shear wave decreases (Fig. 4).

Similarly, measurement of P wave velocity by a sophisticated instrument having 16 channel sensors were also measured in 1.5mX 1.5m area on one of the walls of the same building (Parajuli et al. 2009). The wall near a sensor was hit by a spherical ball attached hammer and P wave arrival times in all sensors were noted. The procedure was repeated for all sensors. The diameter of hitting hammer was also altered and series of measurements were taken. In Fig. 5, velocity tomogram was obtained by hitting 8mm diameter hammer on outer (left) and inner (right) side of the wall. Different color pattern shows different qualities between inner and outer surfaces of the wall. The velocity varied from 500 to 1000m/sec. However, the velocity range is wide, if the results from both experiments are compared, it can be observed that the primary wave velocity is around 700m/sec and shear wave velocity is around 400m/sec.

Fig. 3 Instrumentation of two channel Pocket AE

Table 1 Obtained parameters

Type	V_p (m/sec.)	G (N/mm^2)	V_s (m/sec.)	E (N/mm^2)
Single brick	2499	4537	1531	10889
11 cm wall	2576	4820	1677	11567
23 cm wall	1312	1250	803	3000
44 cm wall	670	326	410	782

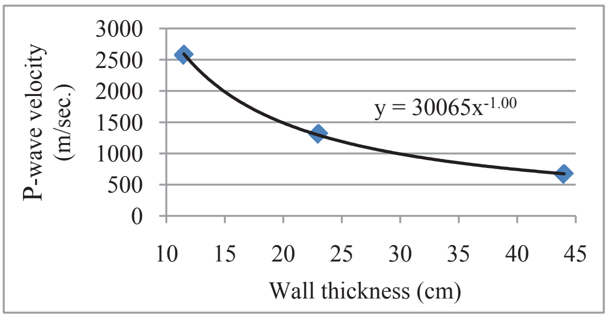

Fig. 4 P-wave velocity with wall thickness

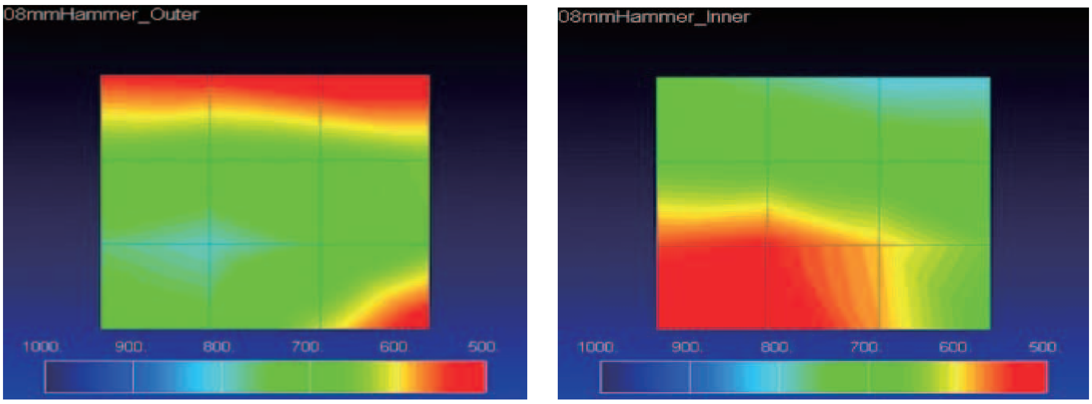

Fig. 5 Elastic wave tomography in 2D

The experiment results show that the P wave velocity decreases as the wall thickness increases. The joints and voids inside the wall sharply decrease its strength; as a result P wave velocity is found decreasing (Fig. 4). If the P wave velocity is projected for 55cm wall, it becomes 547m/sec. which lies in the range of velocities shown in Fig. 5 and seems reasonable. Though, interpolation and extrapolations would not be the case always, rather vary wall to wall depending upon its own properties. However, the trend of curve in the Fig. 4 shows, elasticity decreases with the increase of joints and voids in thicker wall.

3.2 Other Parameters

The stiffness constants along shear and normal directions are calculated by considering the differences of module of elasticity between a single brick and wall. The wall is represented by series of two springs, one by brick element and other by joint (Senthivel et al. 2006) as shown in Fig. 2. The concept of joint element is to represent the non linear behavior of the adjacent elements. The model has elements with varying thickness, therefore separate values of coefficients are required depending upon their depth. However, in this study an average value of depth 25cm has been taken and calculated corresponding values of normal and shear stiffness coefficients are 3.4GN/m^3 and 1.4GN/m^3 respectively. Unit weight and modulus of elasticity for wooden elements are 4.47KN/m^3 and 8.1xGN/m^2 respectively. Regarding the damping, very limited information is available in linear solid mechanics problem, and even very less information is available in non linear dynamic analysis. For problem under consideration, the Raleigh coefficients α=0.0174 and β=0.172 are taken maintaining the damping approximately 3% following Wakai and Ugai 2004.

4. Description of Model

The brick masonry building (Fig. 1) is located in Lalitpur Municipality of Kathmandu city. It has been divided into two rooms longitudinally (Figs. 6-7). It is a double storeyed building; 16.5 m long and 5.6 m wide. The wall is made of traditional brick and 60cm thick at the bottom and 50 cm at the top, a slight tapering from bottom to top. An average thickness 55cm is taken for an analysis. It was constructed three hundred years ago. It sustained damages in earthquakes and repaired many times. Recently, its original roof has been replaced by corrugated galvanized iron sheet which rests over wooden planks and battens and the interior wall has been plastered by cement sand mortar. The floor has been recently replaced by concrete which rests over wooden boards supported by planks and beams. Now, it is repaired hiding its original construction and has been using as public purpose. The building has very large opening in the front side. Wooden posts are supporting the wall of upper storey. In the upper storey, there is a big wooden window placed at mid span of the wall and is slightly projected to outside showing a nice aesthetic view.

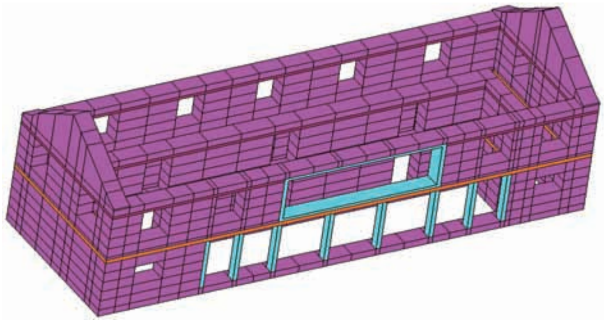

Fig. 6 Simplified FEM model

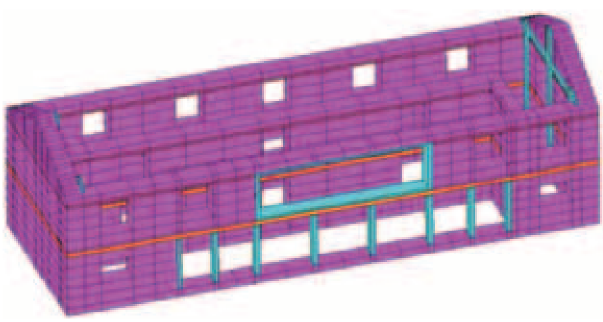

Fig. 7 Modified FEM model

In the numerical model, vertical wooden posts have been put in the ground floor and an equivalent wooden frame has been put to replace the big window of the upper storey. Separate modelings of walls, floor, roof, and their component-bricks, windows, doors, posts are extremely complicated jobs. Small partition walls have not been considered in the model. Simplified model considering load bearing walls have been constructed (Fig. 6). The wall is discretised into small number of solid brick elements, vertical posts are modeled as wooden solid and the big window of the upper storey is modeled as equivalent solid elements. Total elements are 2995, solids are 1186 and joint elements are 1809. As a possible strengthening solution, two wooden frames have been added inside the building (Fig. 7). The different colors shown in the models show the different materials. In the first model (Fig. 6), there are joint elements between the different materials whereas in the second (Fig. 7), model has been modified putting the rigid joint between the posts and connecting elements, floor elements and window elements. The added frame should be fixed into the wall and thus the connectivity between them is considered rigid. These changes make significant changes in the numerical model though they look similar in the Figs. Wooden elements and bricks are inter-connected at floor and roof levels, they behave like semi-rigid floor diaphragm. If theses element are considered separately the model becomes very complicated and thus, floor mass is lumped at wall where the floor and roof rest. To differentiate the material properties separate color can be observed in the model (Figs. 6-7). During lateral loadings, floor acts rigidly and the corresponding solid elements at floor and roofs are assigned rigid with same material properties. Joint elements are provided to connect the floors with walls. Total loads of floors were calculated 1.5KN/m^2.

5. Analysis, Discussion and Conclusion

At first, static analysis was run for vertical loads and self weights. And obtained stresses were used in dynamic analysis as initial stresses. In the second step, dynamic analyses were run inputting Kobe 1995, El Centro 1940, simulated 98 and 475 years return period earthquakes (Fig. 8). Equations of motions were evaluated at 0.01 interval of time by Newmark's beta method satisfying the Mohr-Coulomb criterion for slide and separation (Tzamtzis and Nath 2004). The residual forces obtained from deducting actual force developed and permissible force calculated from constitutive relationship produces non linear deformation at the joints which are evaluated by Newton Raphson method.

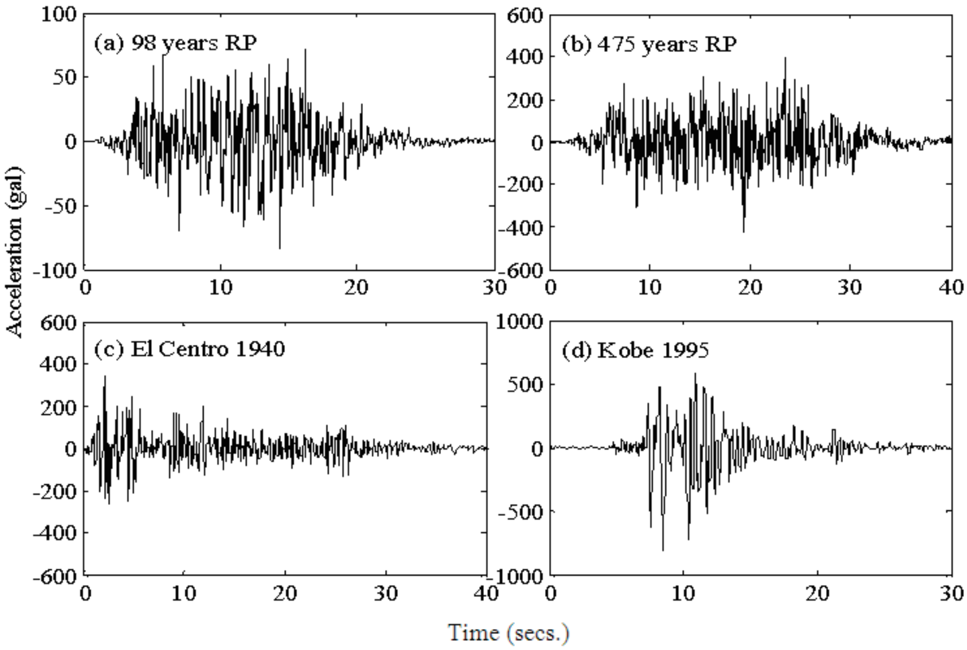

Fig. 8 Input ground motions

In full three dimensional analyses, there is possibility of obtaining tensile and compressive forces right from the beginning. So, the building experiences tension at some areas, most likely near openings and at other weak zones which goes on iteration and finds non linear deformations. If the residual forces are big, iteration takes very long time and ultimately computation becomes very lengthy. Since, there are no predefined criteria to define failure of masonry buildings; a ceiling value-30cm displacement has been set in the program. Normal length of brick is 23cm, and, if deformation exceeds 30 cm, it completely dislocates by its original position. However, it is arbitrarily assumed value and one can take its own definition and value. The purpose of termination of analysis is just to save the time only. The building experienced more than 30cm displacements in El Centro 1940, Kobe 1995 and 475 years return period earthquakes in few seconds.

The deformations obtained by various time histories are shown in Figs. 9-14. The deformations of elements are shown in different colors (values are in meters). The deformation depends upon amplitude, frequency content and duration of earthquakes. The building sustains very large deformation quickly in Kobe 1995 earthquake since it has has highest amplitude. It also gets large deformation in El Centro 1940 and simulated 475 years return period earthquakes after few seconds (Figs. 9-12). The difference between them is time only. In all cases the large deformations can be seen in the gable wall and near the large openings which is quite expected. The deformations are bigger along in Y (shorter) direction than in X (longer). It is usual because the wall stiffness is greater along X than in Y.

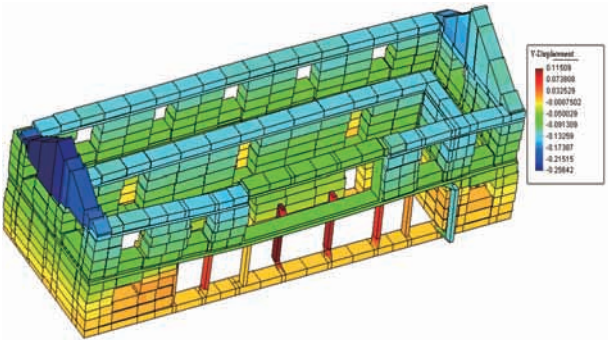

Fig. 9 Deformations in 475 years RP earthquake

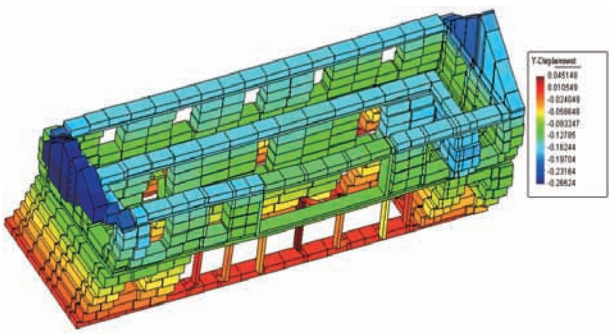

Fig. 10 Deformations in Kobe 1995 earthquake

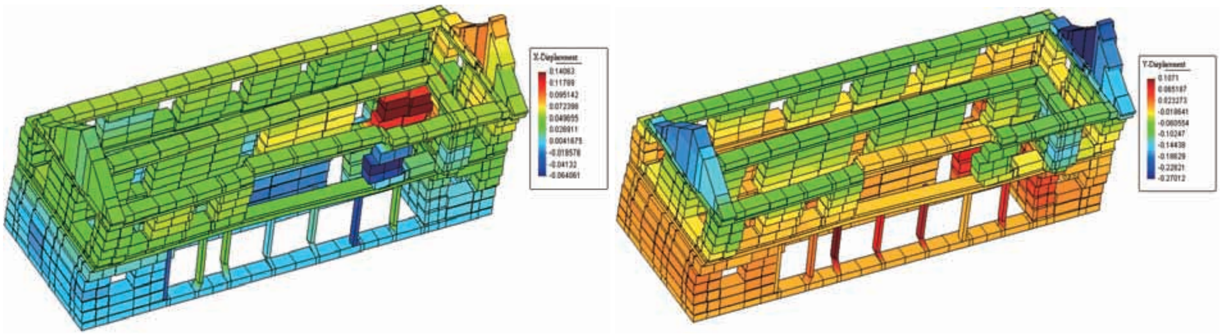

Fig. 11 Deformations in El Centro 1940

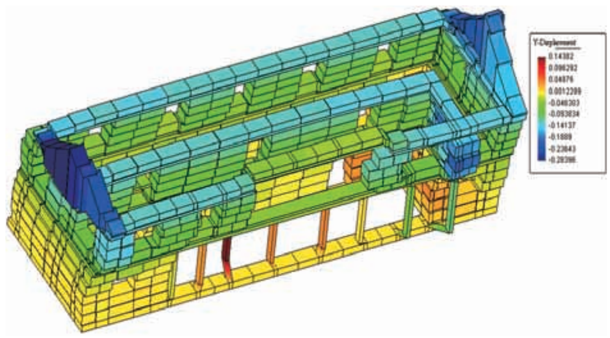

Fig. 12 Deformations in 50% de-amplified El Centro 1940

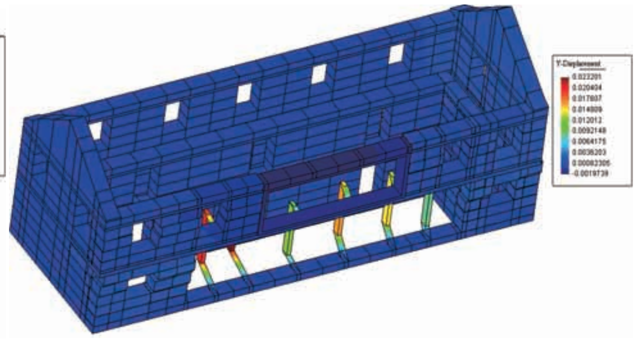

Fig. 13 Deformations in 98 years RP

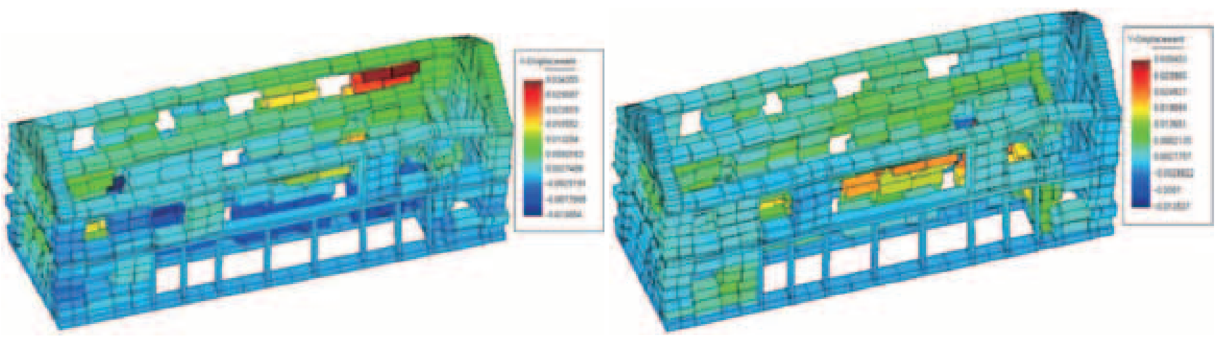

Fig. 14 Deformations on modified in El Centro 1940 earthquake

Dislocation of vertical posts and initiation of deformations also can be seen in the simulated 98 years earthquake, however, the values are very small (Fig. 13). This shows that the building is able to resist the around 100 gal or less amplitude earthquakes, but, it cannot resist even greater than 150 gals acceleration since it has sustained large deformation in 50% reduced El Centro earthquakes (Fig. 12). However, duration and frequency content should also be considered while defining complete failure. As reported in contemporary literatures, El Centro 1940 earthquake is the first recorded earthquake and 40% houses had been damaged. It is quite reasonable to say that this brick masonry house sustain very large cracks and deformations and cannot survive. The building sustains very large deformations under all given earthquakes and it is proved to be very weak and cannot take any kind of severe earthquake loadings.

Being historical building, it has heritage value and should be protected against future earthquake. There could be many possible methods of strengthening of buildings, however, archeologists and conservationists do not allow intrusion by all kind of materials such as concrete, steel, FRPs etc. Thus, there are very few options remained; for example, addition of wooden beams and column internally could be one of the possible options. Thus, looking at the weak zones, near the openings and top of the shorter walls, strengthening measures are applied; joints between the posts and the connecting elements are made fixed, the floor elements are connected with the wooden elements placed around the openings, wooden beam and posts (Fig. 7). Then, the house is analyzed again in the El Centro 1940 earthquake ground motion. During full cycles of analysis it gets maximum displacement 3.4 cm (Figs. 14) along Y direction. It shows that simple method of strengthening can contribute significant strength and reduce the large deformation. Being old brick masonry, the wall is already stressed and propagation of crack is obvious even in small deformations. Though it might not be serviceable after earthquake, it may protect the lives. And also, wood is easily available in local areas and easily acceptable by the heritage conservation community, thus, becomes good strengthening alternative for those kinds of buildings.

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