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Evaluation of Seismic Behavior of Masonry Building Materials by Using a Plastic Failure Model

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Abstract: In general, most of the buildings in rural areas possess masonry structures and generally this kind of structure is vulnerable against the earthquake with a poor performance. So, studying and understanding these structures in any region due to climate and the relative risk based on the principle of earthquake strengthening and the possibility of basic securing the buildings is inevitable for removing the weaknesses. Using the model of plastic failure behavior of concrete materials which are available in ABAQUS finite element Software, we studied the behavior of buildings structures as their principals are different from the theory of micro and macro models. One of the main Paradoxes is about the access to structural failure output (cracking) which is not available in other behavioral models. This research has numerical studies and models are studied by using ABAQUS Software which then subjected to pushover analysis. Since, then the seismic analysis are taken and results of software analysis are examined after extraction from ABAQUS Software. Items extracted from ABAQUS Software included of stresses in two directions of X and Y, cracks in structures, kinetic energies, accessible strain and the stress. Finally, a three-dimensional seismic analysis of structure was performed by the software and the destruction of buildings was also monitored.

Key words: Continuous model, building materials, finite element method, plastics failure, ABAQUS

INTRODUCTION

Non-reinforced building structures are types of buildings in which all or some parts of the vertical and lateral loads are tolerated by walls of masonry materials generally of brick, brick, concrete block or stone. Construction of building with building materials such as brick or adobe building causes heavy losses during the medium earthquakes. Therefore, improper effects of building materials used in the construction of buildings are obvious to everyone. But for economic reasons, ease of production and construction, good insulation properties and viewing, causes the application of these materials in buildings especially in the rural areas.

Regarding the seismic history of Iran, it was shown that rural buildings exposed to earthquakes with a magnitude of 5.5 or more on the Richter scale have been fully or major damaged. This type of structure does not meet the terms and conditions for seismic stability and as long as the required changes are not implemented, we often observe the painful events similar to those occurred in Manjil and Roodbar. In general, taking a look at the history of earthquakes in the country, it is clear that rural homes located in the central zone of earthquakes with a magnitude of >5.5 Richter are in the risk of cracking and collapsing.

Of course, the urban brick buildings arranged with cement mortar are more resistant and may be more stable against earthquakes with a magnitude of 6.0-6.5 Richter. But for most of non-reinforced brick buildings with it is not possible to tolerate earthquakes with a magnitude of 7 or more and such buildings collapse in the earthquake. Without a doubt, enhancing the quality of materials and method of construction, the integrity of the roof and lightening, as well as embedding the elements which increase the softness of buildings) such as horizontal coils (which can increase the strength of the building) but none of the definitive measures does not guarantee the definite stability of the structure against earthquakes.

Literature review

Reviewing the previous researches: Construction of building with masonry materials such as stone was widespread since the beginning of the twentieth century where the new building materials such as concrete and steel were used instead, for buildings with low to medium height. Given that these buildings experience heavy losses even in average time of the earthquake, therefore, the inappropriate materials for use in construction of building became obvious to everyone.

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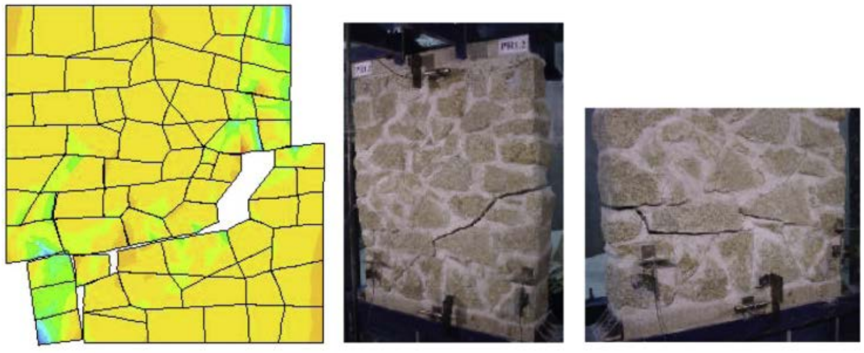

Fig. 1: Stone masonry wall used by Senthivel and Lourenco (2009) in the laboratory and numerical model

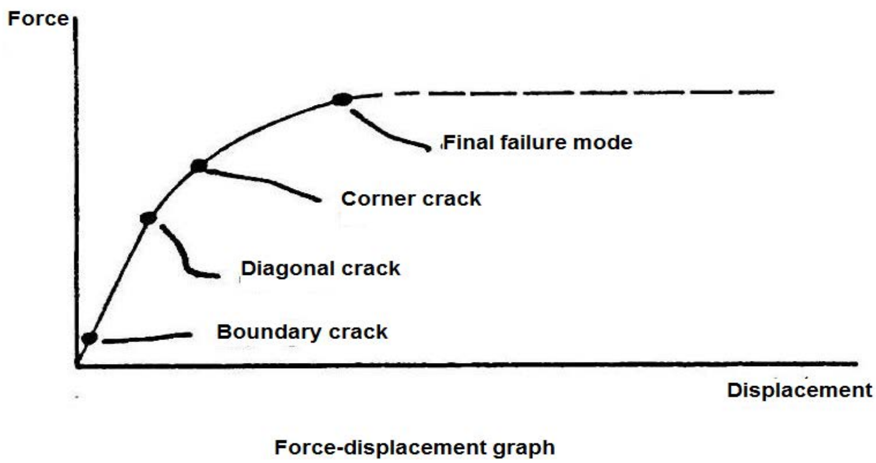

Fig. 2: Graph of force-displacement (Abrams and Shah, 1992)

Some studies have been conducted in Iran and other countries about the reinforcement of buildings with materials as shown: Silva *et al*. (2010) studied the behavior of the inter-plate behavior of three layers stone walls and divided their research to two sections of assessment and evaluation of the behavior of simple wall and evaluation of the walls reinforced with grout injection (Fig. 1). In general, the main purpose of their study was to determine the response of three layers reinforced stone walls to grout injection method. The findings are as follows: In the case of the walls under high vertical overhead that injection grout was not implemented in them, the cause of final failure was ultimate shear failure while rocking mechanism was observed for the injected walls.

The main weakness of masonry buildings against earthquake is not the lack of strength but it is the lack of plasticity (ductility). When a composite frame is put under lateral force on its plate, separation occurs between the frame and cross-frame in the tension corners and a crack is form known as “border crack”. By increasing the force, the diagonal cracks are formed along the pressure diameter, as by developing to the cross-frame, the force is as high as the stresses occurred in the compressive corners crush the cross-frame and failure is happened. After corner failure, the hardness is extremely reduced but is not zero. Gradually, the final failure occurs by loading which can be known as the completely plastic area (Fig. 2).

Evaluating the results of finite element analysis software: Generally there are three ways to define the behavior of nonlinear materials. These three methods are included of distinct model, micro model and macro model. Application of the first two methods is not computationally effective. Therefore the macro model is used which can be applied in ABAQUS Software for continuous models such as masonry and concrete materials using three models of cracks in the concrete, brittle crack model, plastic hybrid model, that since the model of brittle cracks can be applied

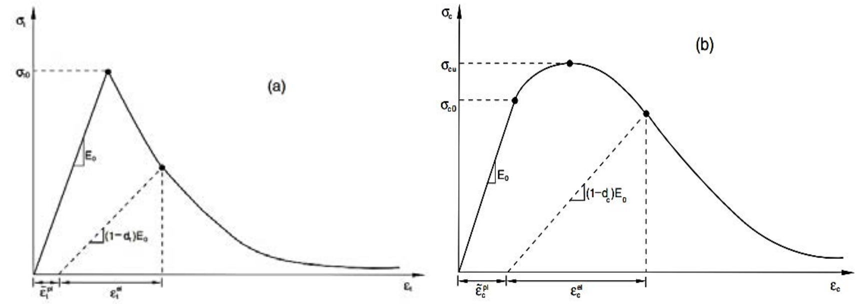

Fig. 3: Concrete responding to the uni-axial loading: a) At tension; b) At compresion

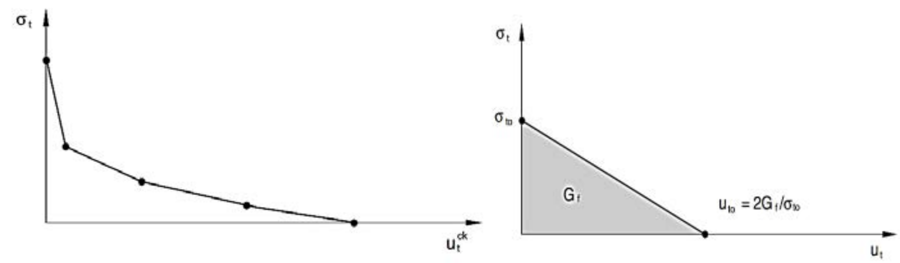

Fig. 4: Failure energy of masonry and building materials: a) At tension; b) At compresion

in implicit method and due to non-linear and brittle concrete, this method is flawed and the brittle crack model is weak in simulation of cyclic behavior that does not consider the crushing of masonry materials, therefore since the plastic hybrid approach can be applied both in explicit and implicit ways, it has a good ability to conduct simulations. This model has the overall capability for modeling of concrete and other quasi-brittle materials in all types of structures. That is why this model has been used. In plastic hybrid model of concrete damage, it was assumed that the uniaxial compressive and tensile response of concrete is as shown in Fig. 2.

Obviously, the stress-strain response under uniaxial tension shows a linear elastic behavior up to failure stress of σ_{t0} (Fig. 3a). The fracture stress corresponds to the nucleation of micro-cracks in the concrete material. It was also observed that stress-strain response under uniaxial pressure is linear up to the initial linear yield stress of σ_{t0} (Fig. 3b). The energy required to open the unit era of crack (Gf) was introduced by Moghaddam in 1976, as the property of matter using concepts of failure of brittle materials. Using this method, concrete brittle behavior is specified by response of stress-displacement instead of stress-strain. Failure energy of cracking model can be presented by post failure stress as a function of displacement caused by cracking, as shown in Fig. 4a. In this case, the failure stress (σ_{t0}) is defined as a function of the fracture energy.

In this case, it is assumed that the strength of the concrete member after cracking is decreased linearly as shown in Fig. 4b. In reality, cracking displacement is happened when the strength is completely faded. In this case, the cracking value is as: $u_{t0} = (2G_f)/\sigma_{t0}$. In ABAQUS Software, the materials non-decreasing local quantities of variables are considered, so the damage factor of the concrete is shown as:

$$d_t\big|_{t+\Delta t} = \max\left\{d_t\big|_t, d_t\left(\tilde{\varepsilon}_t^{pl}\big|_{t+\Delta t}\right)\right\} \tag{1}$$

where, dt is the tensile damage factor (number between 0-1 as zero shows the non-damaged material and 1 represents the completely damaged one). $\tilde{\varepsilon}_t^{pl}$ Shows the tensile equivalent plastic strain. Potential of the plastic flow (G) for this model is the Drucker-Prager hyperbolic function as shown by Eq. 2:

$$G = \sqrt{(\in \sigma_{t0} \tan\varphi)^2 + \bar{q}} - \bar{p}\tan\varphi \tag{2}$$

where, $\bar{p}$ the hydrostatic compressive stress is defined based on the invariables of the effective stress as shown in Eq. 3:

$$\bar{p} = -\frac{1}{3}\text{trace}\bar{0} \tag{3}$$

where, $\bar{\sigma}$ is the matrix of the invariables of the effective stress, obtained by Eq. 4:

$$\bar{\sigma} = D_0^{el} : \left(\varepsilon - \varepsilon^{pl}\right) \tag{4}$$

where, D_0^{el} is the first elasticity matrix (not damaged) of concrete. It should also be noted that in Eq. 6 the effective stress is equivalent to Mizes expressed by Eq. 5:

$$\bar{q} = \sqrt{\frac{3}{2}\left(\bar{S} : \bar{S}\right)} \tag{5}$$

where, $\bar{S}$ is the effective stress of the diversion obtained by Eq. 6:

$$\bar{S} = \bar{\sigma} + \bar{p}1 \tag{6}$$

In addition, φ is the dilation angle measured at p-q plate and σ_{t0} is the uni-axial tensile stress of failure and ε is the Eccentricity-dependent parameter denoting the rate of approximation of potential flow to the Asymptote. Of course, it should be noted that when the Eccentricity moves toward zero, flow potential shifts to a straight line (Abrams and Shah, 1992). In the behavioral model of yield function, the yield function is used to calculate the evolutionary trend of strength under stress and pressure as the evolutionary of the surface is controlled by variables of $\tilde{\varepsilon}_t^{pl}$ and $\tilde{\varepsilon}_c^{pl}$. The yield function based on the effective stresses is shown as:

$$F = \frac{1}{1-\alpha}\left(\bar{q} - 3\alpha\bar{p} + \beta\left(\tilde{\varepsilon}^{pl}\right)\left(\hat{\bar{\sigma}}_{max}\right) - \gamma\left(-\hat{\bar{\sigma}}_{max}\right)\right) - \bar{\sigma}_c\left(\tilde{\varepsilon}^{pl}\right) = 0 \tag{7}$$

In Eq.7, $\hat{\bar{\sigma}}_{max}$ is the main effective stress and parameters of Ψ, β and Ψ are calculated based on Eq.8-10:

$$\alpha = \frac{\left(\frac{\sigma_{bo}}{\sigma_{co}}\right) - 1}{2\left(\frac{\sigma_{bo}}{\sigma_{co}}\right) - 1} : 0 \le \alpha \le 0.5 \tag{8}$$

$$\beta = \frac{\sigma_c\left(\varepsilon_c^{pl}\right)}{\sigma_t\left(\varepsilon_c^{pl}\right)}(1-\alpha) - (1+\alpha) \tag{9}$$

$$\gamma = \frac{3(1-Kc)}{2Kc - 1} \tag{10}$$

where, σ_{bo}/σ_{co} is the ratio of initial equibiaxial compressive yield stress to the uniaxial initial compressive stress, as the default value was set 1.16. In addition, $\bar{\sigma}_t(\tilde{\varepsilon}_c^{pl})$ is the tensile strength and value of is obtained by Eq.11 (Senthivel and Lourenco, 2009).

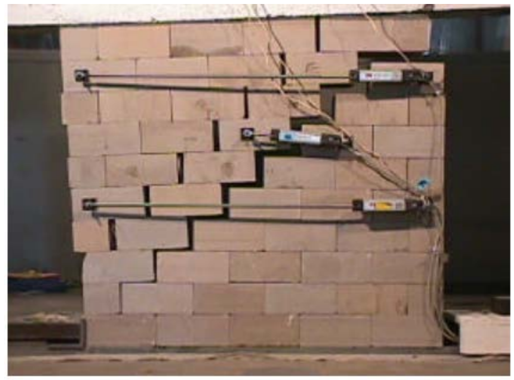

Fig. 5: An image of the wall constructed in the laboratory after loading

$$\bar{\sigma} = \frac{\sigma t}{1-d_t} = E_0\left(\varepsilon_t - \tilde{\varepsilon}_c^{pl}\right) \tag{11}$$

where, E_0 is the initial elastic strength (non-damaged materials) and dt is the parameter of the tensile damage. Also $\bar{\sigma}_t(\tilde{\varepsilon}_c^{pl})$ is the effective compressive stress and value of σ_c is calculated using Eq. 12:

$$\bar{\sigma}_c = \frac{\sigma_c}{1-d_c} = E_o\left(\varepsilon_c - \tilde{\varepsilon}_c^{pl}\right) \tag{12}$$

where, d_c parameter is the compressive damage of the material usually a number between 0-1. Also, it should be noted that K_c shows the ratio of concrete tensile and compress invariables, set as 2.3 as default.

Investigating the accuracy of results obtained by ABAQUS Software analysis: In order to verify the accuracy of modeling and solving method in ABAQUS Software, a laboratory model was used by Vitorion (2010) named "numerical and experimental study of building walls under cyclic loading with mechanical properties and boundary conditions and similar loading". It was made and analyzed by software and is applied to verify the results of numerical modeling with experimental results in this study. Software model is made in homogenous form model and the given mechanical specifications of the wall are listed in Table 1.

Mechanism o failure of laboratory wall at the end of loading under overload of 100 KNand lateral force is shown in Fig. 5. However, the plastic damages of walls after finite element analysis can be seen in Fig. 6. Software could detect faults and cracks in the walls as well. On the other hand, the force-displacement curve obtained by

Table 1: Characteristics of the numerical model

Module of elasticity	Poisson coefficient	Dilation angle	Density of materials	Overload force 100 kN1	Compressive behavior Inelastic strain	Compressive behavior Yield stress	Tensile behavior Cracking strain	Tensile behavior Yield stress
3825000000	0.2	56	1800 Kg m^{-3}	100 kN2	0.00141	297656.25	0.00057	29580

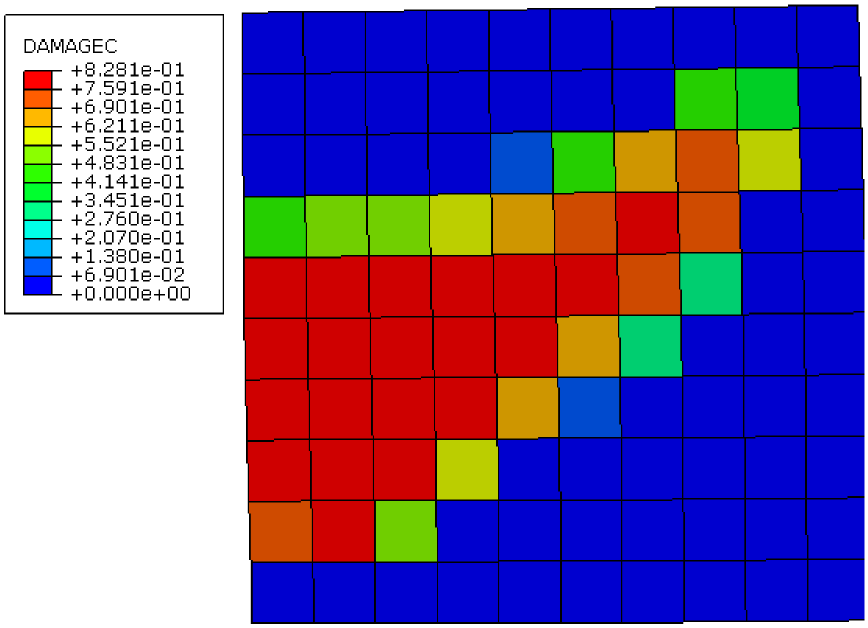

Fig. 6: An image of the wall constructed in the software after loading (diagonal compressive cracks of wall)

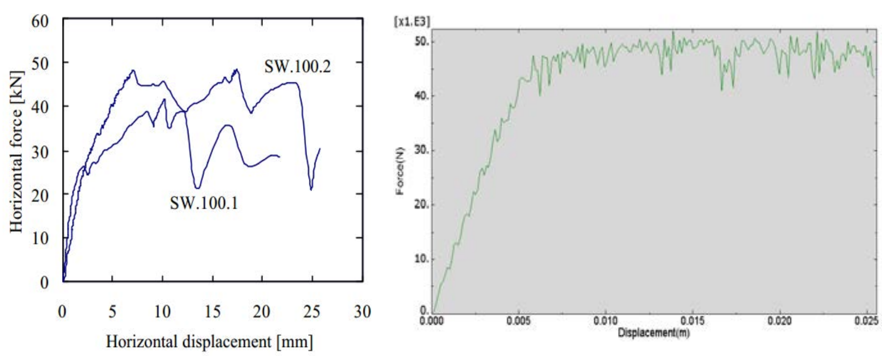

Fig. 7: The force-displacement graph of control sample compared to the results obtained by ABAQUS

laboratory work of Vitorion and software modeled diagram is shown in Fig. 7. Obviously, no significant difference can be seen in the detection of capacity of the wall and therefore, modeling and testing in laboratory do not differ from each other and are well approximated.

MATERIALS AND METHODS

Numerical models derived by the software: In general, today there are two methods among researchers to model the wall; macro modeling; in this way the whole set wall is considered as a homogeneous body and for the intended homogeneous body, all mechanical properties such as modulus of elasticity intended all, yield stress and strain and final strain are imported to the software by standard special test.

The micro modeling; in this case, wall materials are defined individually, as for each of them mechanical properties and relevant behavior is imported to software. Application of this method is much more complicated than

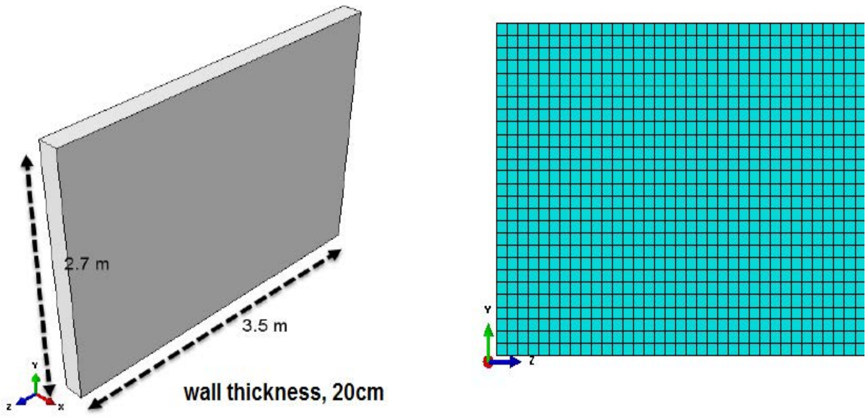

Fig. 8: A continues simple wall with meshed configuration

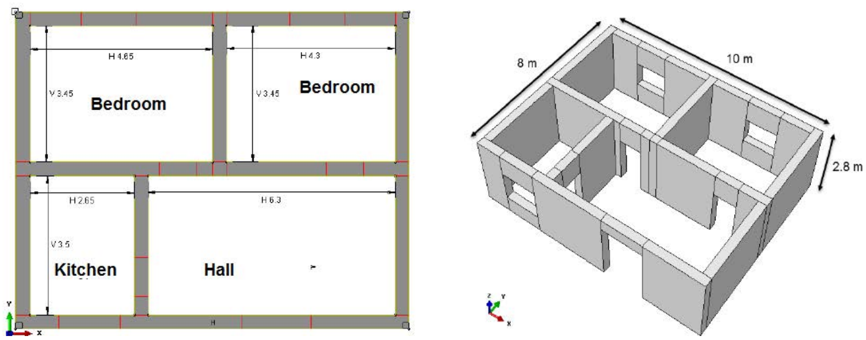

Fig. 9: Sample of a 3D model of a building simulated in the software (Values are in meter)

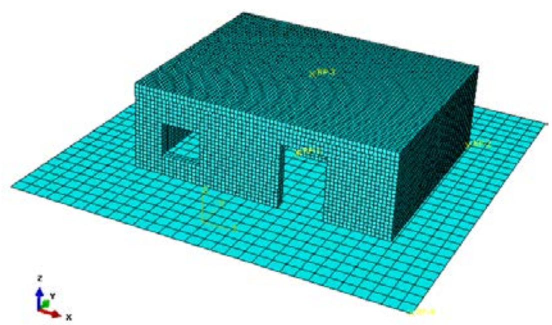

Fig. 10: Meshing of the building structure modeled in the software

the previous method and the time it takes to finite element analysis in this way is so much longer than macro model. Therefore the macro modeling approach is used.

Characteristics of material and dimensions of the wall modeled in the software: The modeled wall was considered with a thickness of 20 cm length of 3.5 m and height of 2.7 m without openings as a concrete bundle was located on it. The mentioned wall was a bearing wall as the 200 KNgravity load was widely places on the on coils over the wall. It was clamped at the top and down sides under a simultaneous lateral loading. The maximum compressive and tensile strains were max 0.005 and 0.22, respectively.

Elements used in the mesh was C3D8R for walls and concrete beams, S4R for the bed, C3DR8 for three-dimensional ceiling (Fig. 8). The purpose of this model was to detect the cargo and cracking model of a simple wall. In addition, a building with all its openings under seismic load was simulated by software, as the size of the investment plan and perspective is shown in Fig. 9. Figure 10 also shows the model meshed model of this building.

Choosing the accelerograms of earthquakes in numerical analysis: Selected earthquakes have been extracted from seismic bank of strong motion in PEER Land Bank. Selected earthquake was happened on the Soil type III (such as old Bushehr texture). Since for the region with relatively high risk, based on the standard 2800, the PGA value was equal to 0.3 g. The chosen PGA earthquake was about 0.3 g to meet the dependency

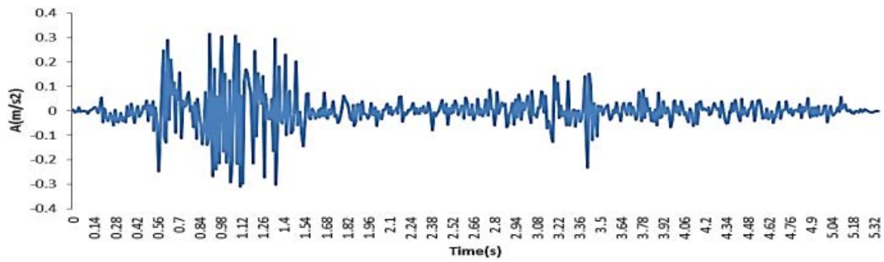

Fig. 11: Horizontal acceleration component of napalm earthquake

effects of earthquake frequency to PGA. The napalm earthquake was occurred during 5.32 sec at a distance of 10 km from the source with the magnitude of 6 Richter. The earthquake occurred in 1986.07.08. Figure 11 shows the quake's horizontal component.

RESULTS AND DISCUSSION

First, in one step the overload force of 100 kN was applied to simple wall and after the analysis of stresses and deformations, the lateral force was applied to the wall as displacement and the value of imposed force and deformation were calculated at the end. Finally diagram of force-displacement were drawn based on the two parameters. Accordingly, the maximum shift in the end of analysis was 2.5 cm and a maximum shear stress was predicted to be 0.62 Mpa (Fig. 12). In addition the ultimate behavior of wall under a basic shear graph (force-displacement) was shown in Fig. 13. The border crack of structure was occurred after 5mm and diagonal cracks were also developed followed by increase of displacement as about 1 cm.

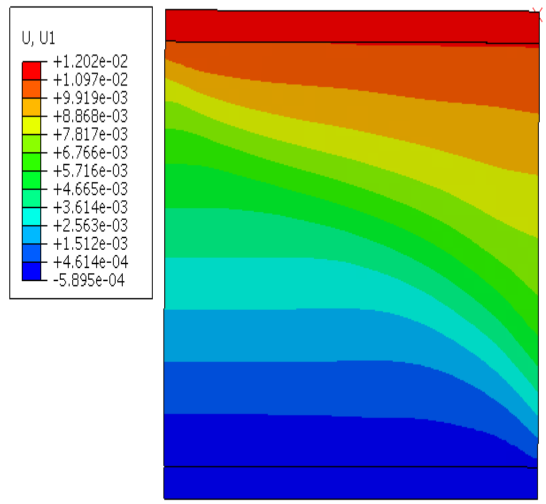

Fig. 12: Contour of simple displacement (in meter)f

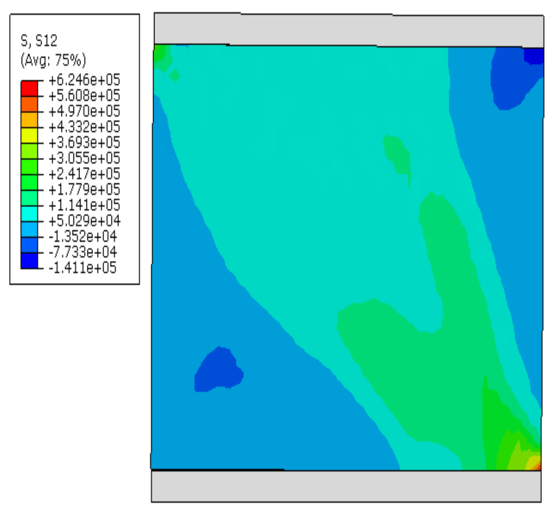

Fig. 13: Contour of shear stresses created in simple wall (in Pascal)

Studying the seismic response of three dimensional building modeled by using continuous model: In this section, behavior of the walls of the three-dimensional buildings is described and it will be examined in the case of the tension on the wall affected by earthquake. In this regard, Fig. 12-16 described the stress contours in 1, 3.67 and 5.45 sec after the earthquake. In moments of 1 and 3.67 sec, the value of obtained stress was more than compressive stress which can be tolerated by the walls. The amount of stress are 2.67 and 1.53 MPa (Fig. 14 and 15).

The tensions are gradually reduce and earthquake velocity decreases, as at the times of 5.45 sec, it reached 1.32 MPa (Fig. 17). It should be noted that tolerable compressive stress in building stone walls was considered about 1 MPa in the software. Although, regarding the contours shown in early times, the wall showed an elastic behavior in most areas but at the initial seconds, nonlinear performance of wall in lots of areas such as boundaries, heel and hotspots was subjected to plastic tensions. In addition to the surrounding walls, this

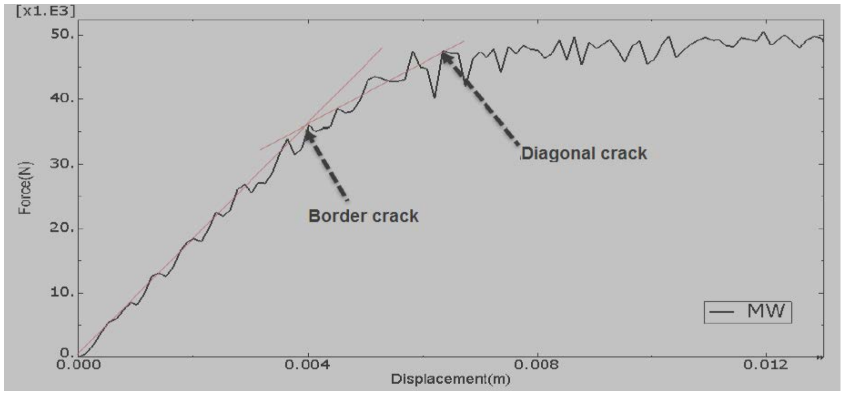

Fig. 14: Graph of force-displacement of a simple wall

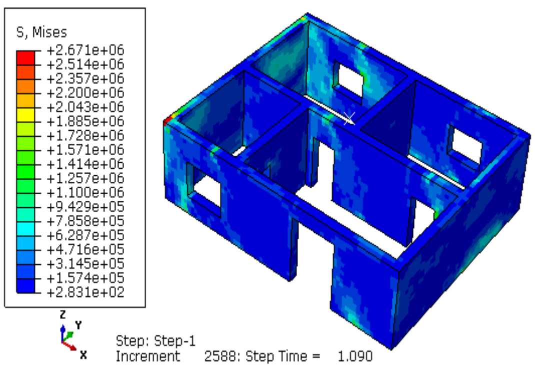

Fig. 15: Mises stress Contour-1 sec after occurrence

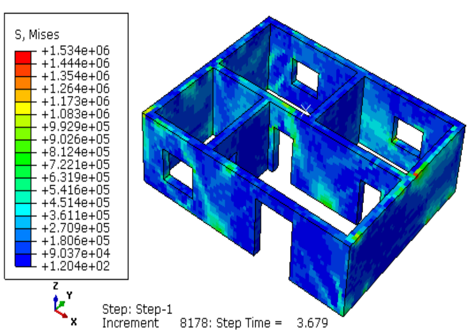

Fig. 16: Mises stress contour-3.67 sec after occurrence

point is also dominant for the internal walls of the building. But the stress value is considerably lower than the surrounding walls (Fig. 18).

By observing the contours of 14-16, we can find out that the junction of walls and openings margin have chosen tensions. According to Fig. 18 10 d, maximum stresses were 2.67, 1.63 and 1.52, imposed at the wall-to-wall connection points, door and window openings margins, respectively. It is important to note

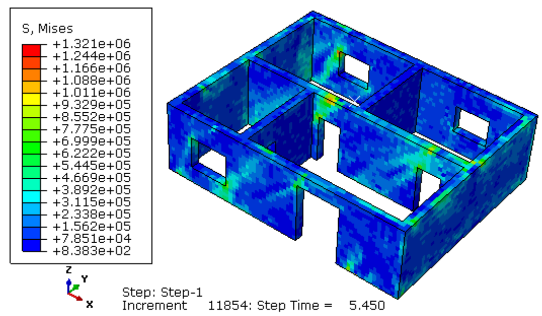

Fig. 17: Mises stress contour-5.45 sec after occurrence

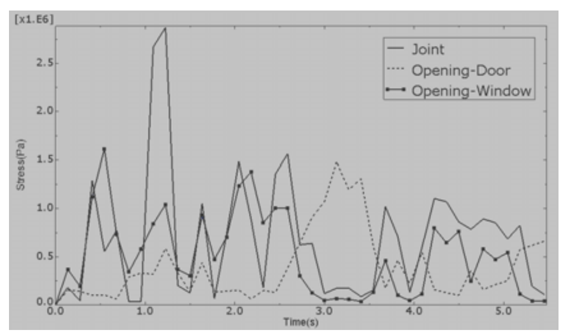

Fig. 18: History of stress changes in areas of the connection

that due to the acceleration peak of earthquake was occurred at 1-2 sec, the maximum stress on the same joints of wall-to-wall moments was also occurred at these moments (Fig. 17).

Investigating the status of cracking of walls: In general, most of the walls of the buildings have been damaged severely in the direction of the horizontal component of and this important point in Fig. 18 and 19. Figure 18 shows the moment and location of diagonal cracks in the wall. In

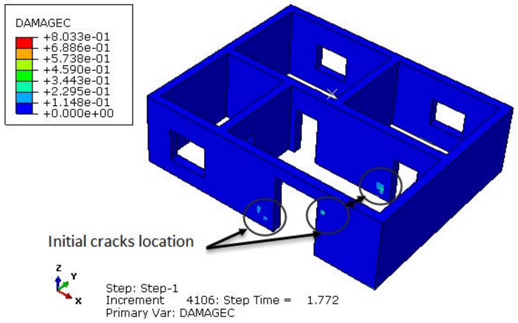

Fig. 19: The time and location of cracks in the direction of vibrations

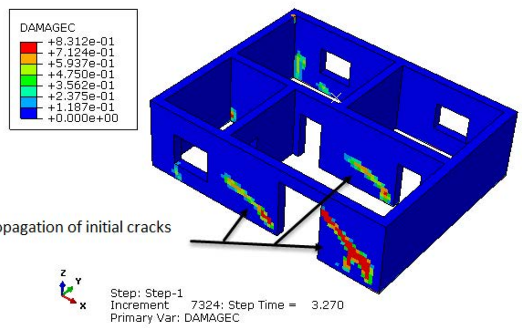

Fig. 20: Development of cracking of walls in the direction of vibration

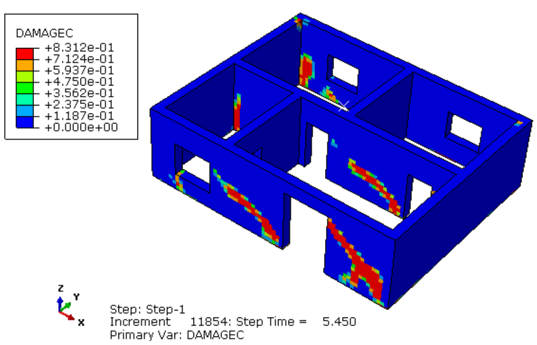

Fig. 21: Cracks of the building at the end of seismic analysis

the case of type of cracks in walls it must be stated that the dominant failure in Iranian building structures is because of the mortar slip and diagonal tension. As shown in Fig. 19 and 20, the walls of buildings in this case have been subjected to diagonal tensions, as the direction of building cracks in direction of acceleration has rotated as 45° in relation to the horizontal components of the earthquake. In Fig. 20 all of the cracks of the building at the end of the seismic analysis are shown.

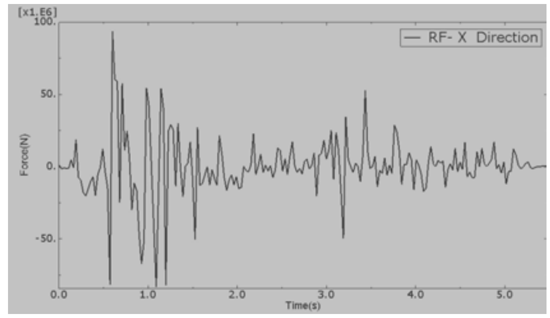

Fig. 22: History of base shear of Building

One of the most decisive factors in the improvement of existing buildings is to determine the forces of the dynamic analysis. Figure 21 and 22 shows the history of horizontal forces (base shear) induced by seismic causes of the building.

Figure 21 shows the maximum horizontal cut of the structure was determined at the moment of 0.67 sec after earthquake occurrence, was 89MN determined, that the figure is not illogical given the huge amount of weight of this structure.

CONCLUSION

In general, most of the buildings in rural areas are composed of are masonry structures and generally this kind of structure have a poor performance against the earthquake, so studying and understanding of these structures in each region according to the climate and the relative risk of earthquake, exploring the possibility of securing buildings and fixing the weaknesses is inevitable. However, the technique of implementation of these structures is the same in every region of the world but the materials used to build the structures are different.

Usually the type of these materials (climate stones, bricks or blocks with diverse mortars used for construction of these buildings) causes that the accurate evaluation methods of these structures becomes a bit complicated. In three-dimensional buildings, Mises stresses of the wall after seismic vibration of the building were assessed in the moments of 1, 3.67 and 5.45 sec. The values of these tensions were 2.67, 1.53 and 1.32 Mpa. It should be explained that the mentioned building was tested by seismic analysis in its strong direction, in which direction the inner walls failed against the seismic vibrations, despite the weaker direction (X) as the most failure was happened due to large openings and surrounding wall.

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