The Adequacy of Pushover Analysis to Evaluate Vulnerability of Masonry Infilled Steel Frames Subjected to Bi-Directional Earthquake Loading

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SUMMARY:
Based on experiences in past earthquakes, it can be concluded that infilled steel frame are vulnerable to large earthquakes. Although analytical study based on performing pushover analysis on planar frames indicates adequate resistance, moderate earthquakes induce more damage to this type of structures. Inspecting failure modes of walls revealed the most important influence of perpendicular component of earthquake acceleration on wall plan. In this paper, after verifying the analytical model, the seismic behavior of one-story one-bay masonry infilled steel frame is studied using finite element model under in-plane and out-of-plane seismic excitation. Adequacy of pushover analysis to predict vulnerability of infill wall to in-plane and out-of-plane earthquake components is studied. The results show that utilizing conventional and dynamic pushover cannot correctly predict the vulnerability of this type of structure specially under bi-directional earthquake loading. However former analysis can be used to predict overall response and performance levels of these structures.

Keywords: Infilled steel frame, pushover analysis, In-plane, Out-of-plane, Bidirectional loading.

1. INTRODUCTION

Masonry infill walls of building structures subjected to lateral loads must resist both in-plane and out-of-plane forces. The infills are usually analyzed by evaluating the effect of in-plane and out-of-plane forces independently. The predominant out-of-plane resisting mechanism is arching, which has been observed in both static and dynamic testing (Mc Dowell et al. 1956; Gabrielson and Kaplan 1977; Dawe and Seah 1989; Klingner et al. 1996; Calvi and Bolognini 2004). In-plane drift will also create out-of-plane interstory drift loading on orthogonal panels.

Traditional pushover analysis is performed subjecting the structure to monotonically increasing lateral forces with invariant distribution until a target displacement is reached; both the force distribution and target displacement are hence based on the assumption that the response is controlled by a fundamental mode, that remains unchanged throughout. However, such invariant force distributions cannot account for the redistribution of inertia forces caused by structural yielding and the associated changes in the vibration properties, including the increase of higher-mode participation. The analysis is continued while:

1) Achieving the predicted performance level
2) Structure failure begins
3) Convergence is not achieved

Dynamic pushover analysis is a parametric analysis method in which a number of nonlinear time-history analysis are conducted for structure and the severity of is increased gradually. This type of analysis is carried out to obtain real and accurate dynamic response of a structure under earthquake. Since this analysis is complicated and time-consuming, simple static pushover analysis is generally used instead.

Investigation in the recent years confirms the accuracy of pushover method in structural frames. However, in the majority of these investigations the effect of masonry walls have been ignored and only the structural members have been considered. It is obvious that the behaviour of an infilled frame is different from a bare frame. Despite the significant effect of masonry infills on load-carrying capacity and stiffness of a structure, these members are considered as non-structural elements and their effect is ignored. It should be examined that the results of pushover method are how much accurate when the effect of infills is considered in design and analysis as structural elements. In this paper, this subject is investigated.

In the current paper, finite element model of a one-storey one-bay steel frame infilled by masonry wall is developed. Firstly, verification of the finite element model is carried out using the results of an experimental test results.
Then one storey one bay frame is analysed under two orthogonal directions of earthquake excitation and the results are compared to the results of the model analysed under one directional earthquake record. Then, structure behaviour is investigated using static and dynamic pushover analyses and the results of these two analysis methods are compared.

2. MODELING AND VERIFICATION

One storey infilled steel frame that is tested experimentally by Flanagan & Bennett is selected for verification. The size of the wall is 210×210 cm. the wall is constructed by 30×30×20 cm bricks. The beam and column profiles are W310×52 and W250×45 respectively. The masonry structure is modeled as a discontinuous assembly of blocks connected by appropriate discrete joints. The joints are simulated by appropriate constitutive interface elements so that considerations such as the initiation of fracture, propagation of cracks and sliding at interfaces with different levels of refinement of the assemblage can be taken into account. Therefore, a more realistic and rigorous analysis can be expected since it allows locating exact joint positions and adopting appropriate constitutive models for the blocks and interfaces.Considering the accuracy needed in this study, the micro method was selected for modeling the behavior of infilled brick wall. Shell elements were used to make the frame model. Eight-node solid elements were used to model the masonry units and contact elements were used to simulate the mortar. The tested specimen and the finite element model are shown in Figure 1.

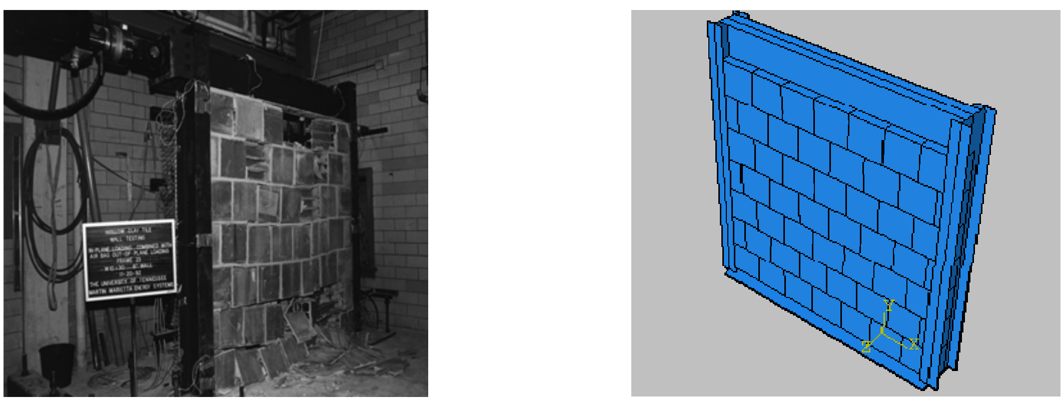


Figure 1. schematic view of test specimen and Finite element model of infilled steel frame

It was assumed that the panel behaves nonlinearly under applied displacement. The mortar was modeled using interaction element which allows for small relative slippage in the contact surface. The properties of this element are defined according to the properties of mortar in the experimental test. The mesh size was fine enough to obtain accurate results. Material properties used in the test are tabulated in Table 2.1. The results of analysis and test results are compared in Figure 2. As can be seen, there is a good agreement between the results. The finite element model is able to predict the strength and also the initial stiffness of the infilled frame.

Table 2.1. mechanical properties of the material

Compressive strength of masonry prism in horizontal direction	5.6Mpa
Young's Modulus of masonry in horizontal direction	5390Mpa
Young's Modulus of masonry in vertical direction	2160Mpa
Masonry material density	817kg/m3

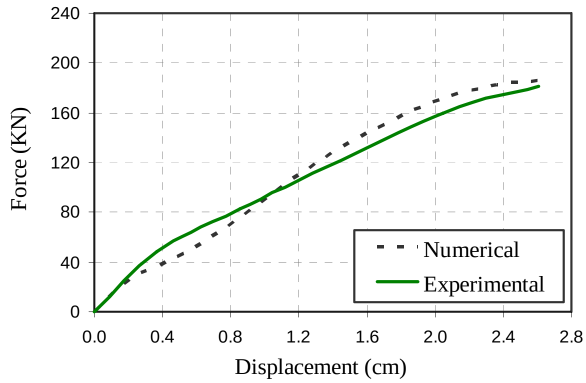
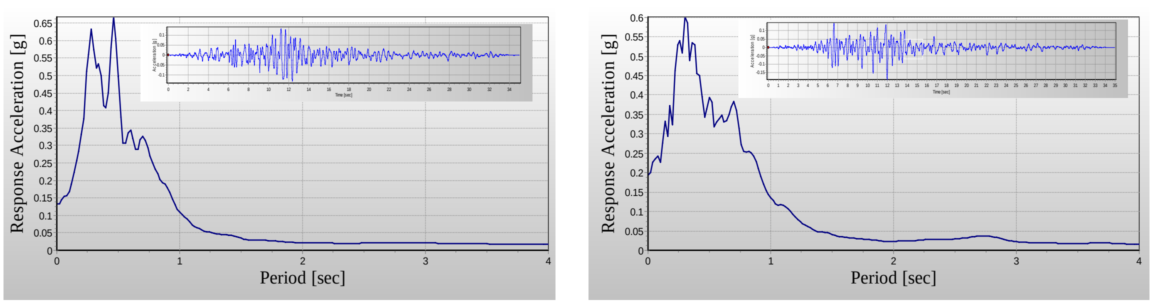


Figure 2. Force-displacement obtained by experimental test and FE model

3. EVALUATION OF INFILLED FRAME BEHAVIOR UNDER EARTHQUAKE LOADING IN ONE AND TWO DIRECTIONS

To analyze an infilled frame under earthquake excitation, a computationally efficient explicit finite element modeling technique that is capable of simulating highly nonlinear events is used. The explicit finite element modeling technique never requires a fully assembled system stiffness matrix; rather it solves for the internal variables using the theory of dynamic wave propagation in solids. Maximum time increment used by the explicit solver related to the stability limit of the structure globally is calculated from the natural frequency corresponding mode shapes of a dynamic system. The loading time was increased up to 100 times the period of the lowest mode. Then, the finite element model is analyzed explicitly under the Northridge earthquake record in two directions simultaneously and also in one direction. The earthquake acceleration records and corresponding spectrum in the two perpendicular directions are shown in figure 3.



(a) N-S component (b) E-W component

Figure 3. Northridge Acceleration spectra for the two orthogonal accelerograms

The in-plane natural period of infilled frame is 0.12 sec. From Iranian seismic code, the design acceleration spectra for high seismic area, corresponding to the period 0.12 is 0.87g. The normalization factor for the record is then calculated to be equal to 3.17. By multiplying this coefficient in both earthquake record components, the normalized earthquake acceleration records in X (in-plane) and Z (out-of-plane) directions are obtained. The beam-to-column and column-to-base connections are rigid. The stress condition in frame and infill in the case where two-directional loading is used is shown in figure 4. In this loading case, the values of displacement in X and Z directions as well as the base shear values are compared in figure 5.

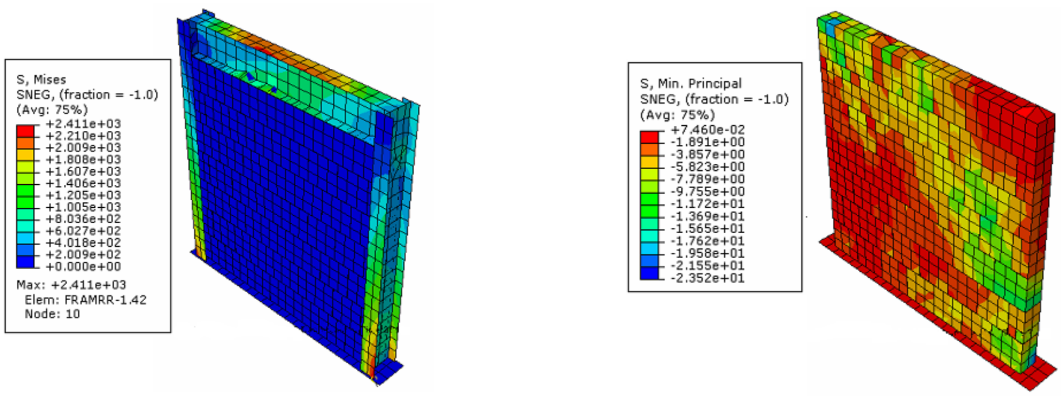


Figure 4. Stress condition in infilled frame and infill wall under simulate application of earthquake components in two directions

In the case where two-directional earthquake loading is used, mortar cracking and separation along the top of infill wall were initially observed, followed by compression cracking in the mortar. Next, diagonal mortar joint cracks developed in direction of loading. On the other hand, high stresses occurred in bottom course. Failure of the bottom course initially is quite reasonable. Strut forces are transferred directly to the base, not into the column. This creates high vertical compression at the base. Vertical arching also creates high vertical compression. The combined effect of these two reasons leads to failure of infill wall.

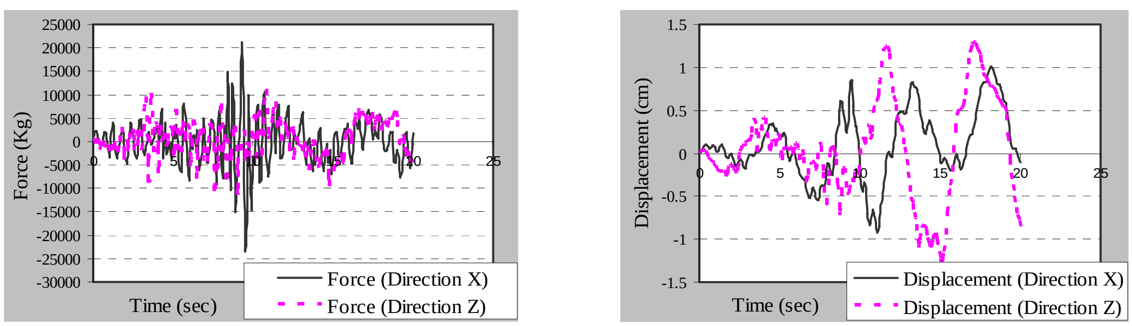


Figure 5. Comparison between structure's force and displacement in X and Z directions (bi-directional acceleration record is applied to the structure)

As it can be observed in figure 5, the maximum relative displacement in x and z directions are 0.48% and 0.61% respectively. As it can be seen from the figure, the displacement in x direction is lower than that in z direction due to the existence of masonry wall in x direction and stiffness increase in this direction. The values of base shear in x and z directions are 231 and 112 kN respectively. As it can be seen, the value of base shear in x direction is much greater than that in z direction.

Then, earthquake record is applied in x direction and z direction separately. Figures 6 and 7 show the results obtained from the two analyses cases, when earthquake is applied in two directions and when earthquake is applied in one direction. In the former case, the value of maximum force and displacement in x direction are 231.2 kN and 1 cm. these values are 231.4 kN and 0.8 cm in the latter case. The values of maximum force and displacement in the former case in z direction are 112 kN and 1.3 cm. these values for the latter case are 151 kN and 1.29 cm respectively. It can be concluded that lower base shear is induced when earthquake record is applied simultaneously in two directions.

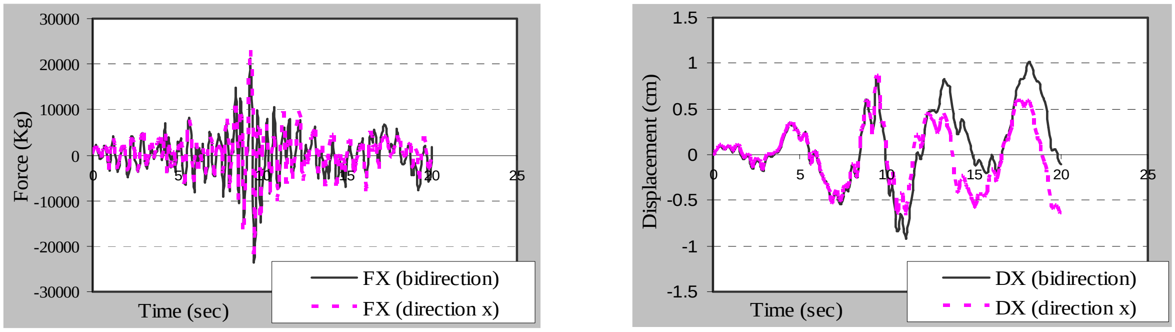


Figure 6. Comparison of force and displacement in X direction in two case: 1- applying earthquake record in X direction, 2- applying earthquake record in two directions

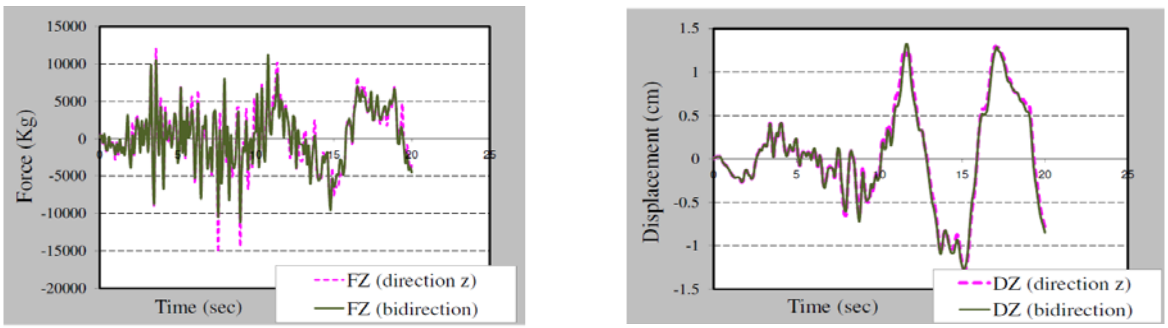


Figure 7. Comparison of force and displacement in Z direction in two case: 1- applying earthquake record in Z direction, 2- applying earthquake record in two directions

4. BEHAVIOR OF INFILLED FRAME USING DYNAMIC AND STATIC PUSHOVER ANALYSES

In plane displacement is applied to the structure in static pushover analysis. However, in the case of dynamic analysis, Northridge earthquake record is applied in two directions simultaneously (in plane and out of plane directions). Considering the obtained time-displacement curve, the value of maximum displacement is read and then the related base shear is obtained using time-base shear curve. This procedure is repeated several times by increasing the severity of earthquake record and in this way, the points of displacement-force curve for dynamic pushover analysis are obtained.

Comparison between the displacement-force curves related to static and dynamic pushover analyses is shown in figure 8. As it can be seen from the figure, the curves are initially the same and after lateral displacement of about 0.5%, the curves are separated and have difference.

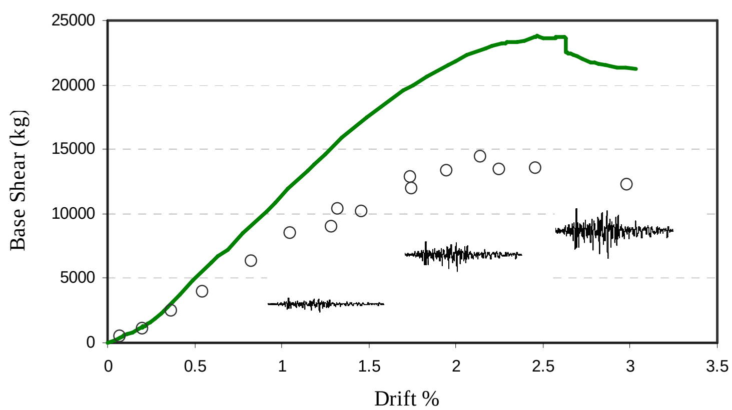


Figure 8. Comparison between base shear-displacement curves of static and dynamic pushover analyses

The difference between the results obtained from the two analyses and the error percentage of static pushover analysis in different displacement are presented in table 4.1. As it can be seen from the table, the error percentage of static pushover analysis is increased in larger displacements.

Table 4.1. Comparison between base shears in different lateral displacements

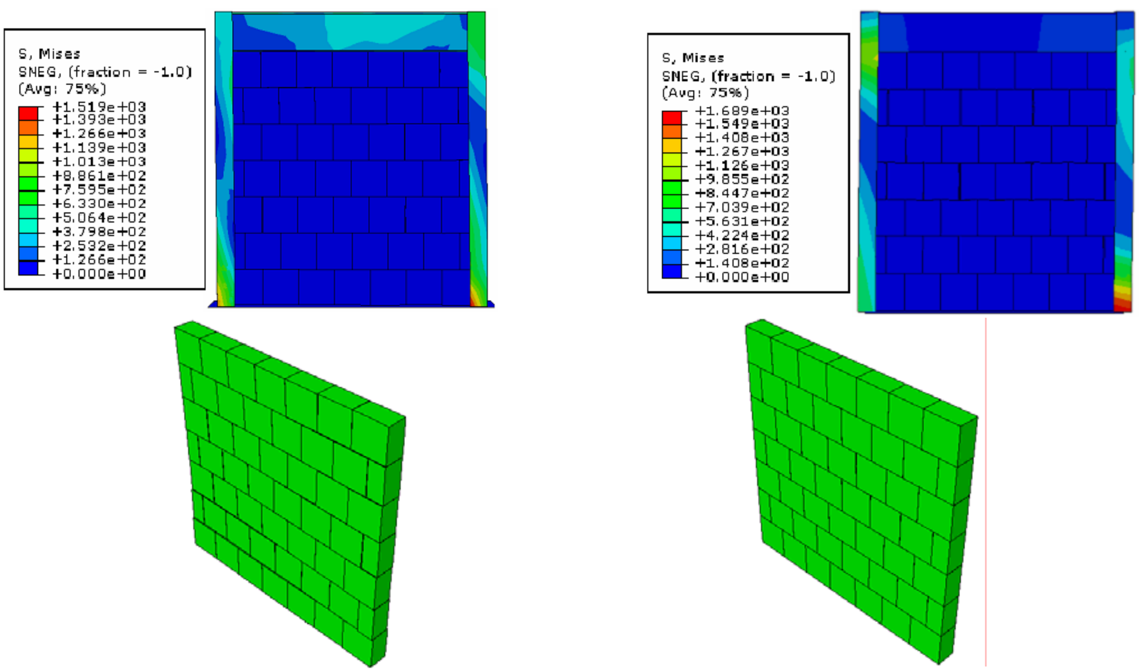
Error (%) / Force (kg)	0.1	0.2	0.5	1	1.5	2	2.5	3
Dynamic pushover analysis	485	1120	4210	8600	10400	12850	13600	12300
Static pushover analysis	500	1215	4810	11800	17200	21900	23600	21300
Error percentage	3	8.4	14.2	37.2	65.3	70.1	73	73

5. COMPARISON BETWEEN INFILLED FRAME PERFORMANCE OBTAINED BY STATIC AND DYNAMIC PUSHOVER ANALYSIS

In the following, cracking pattern of infill walls and stress condition in lateral displacements of 0.5, 1, 2, 2.5 and 3% are compared in figures 9 to 13 for the two analysis methods.

5.1. Lateral Displacement of 0.5%

In lateral displacement of 0.5%, the values of the maximum stress in frame and infill are very close for the two analysis cases. These values in static pushover analysis are occurred at column base and the compressive corner of infill that is in contact with column base. In dynamic pushover analysis, the maximum stresses in frame and infill are formed at the column base and the compressive corner of infill that is in contact with top of column.



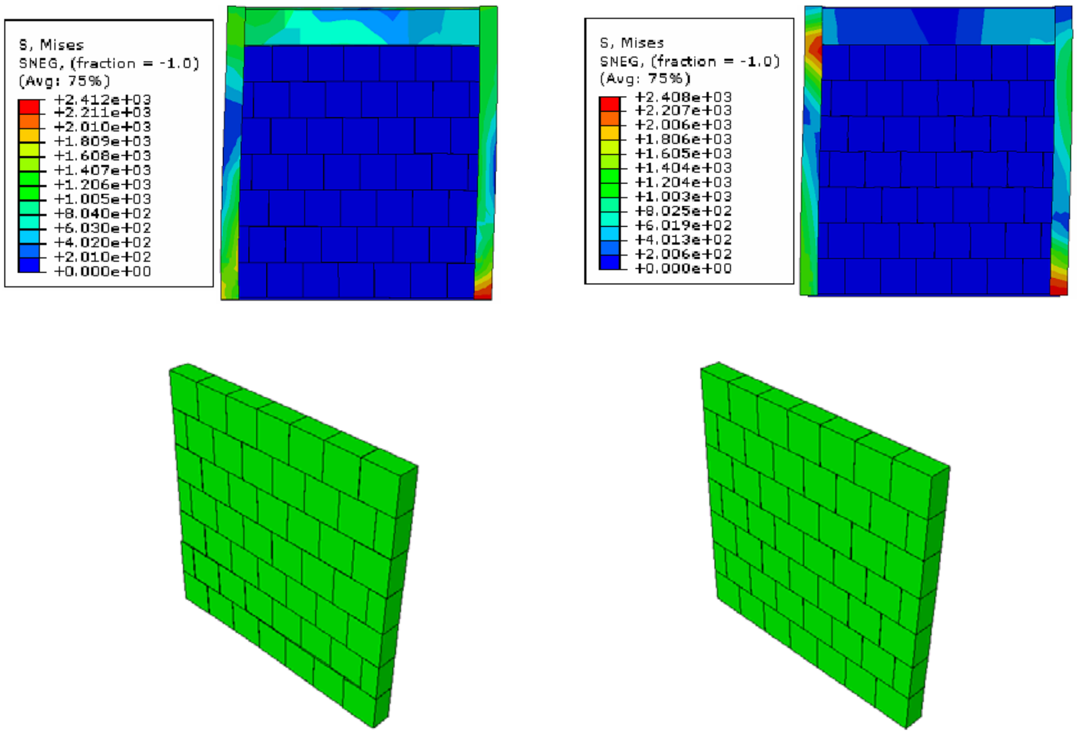
a) Dynamic pushover analysis

b) Static pushover analysis

Figure 9. Comparison between stresses and cracking pattern in static and dynamic pushover analyses (0.5% drift)

In the model related to static pushover analysis, some very fine diagonal cracks can be observed in infill and in the case of dynamic pushover analysis, horizontal cracks between the first brick row and ground and upper brick row is observable in addition to the fine diagonal cracks. The damage of infill in this region can be attributed to the force transition along the infill diagonal that is directly transferred to the ground instead of being transferred to the columns. This induces compressive stresses in the elements that are connected to the ground.

5.2. Lateral Displacement of 1%



a) Dynamic pushover analysis

b) Static pushover analysis

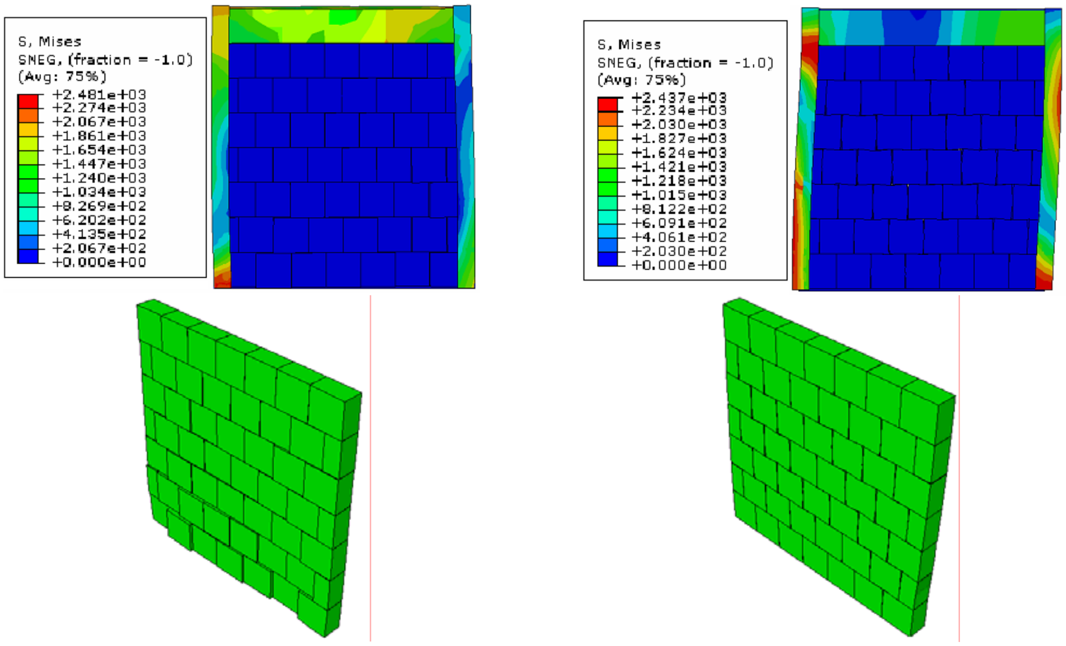
Figure 10. comparison between stresses and cracking pattern in static and dynamic pushover analyses (1% drift)

In lateral drift of 1%, the values of the maximum stresses in both cases are approximately the same and they are observed at the column base. However, the values of the maximum stresses in infill are some different and the value obtained by static pushover analysis is a little larger than that in dynamic pushover analysis.

The width of diagonal cracks are larger in static pushover analysis. In dynamic pushover analysis, in addition to the diagonal cracks, cracking and separation can be observed at the location where infill is connected to the beam. Vertical cracks can also be observed between cricks in the first brick row. Besides, the separation of first brick row and the ground and the second row is also increased.

5.3. Lateral Displacement of 2%

In lateral drift of 2%, the values of the maximum stresses in both cases are approximately the same and they are observed at the column base. However, in the static pushover analysis case, large stresses are formed at the left corner of beam flange. The values of the maximum stress in infills are large and have negligible difference in the two cases. In static pushover analysis, the width of diagonal cracks in infill are increased. In the case of dynamic pushover analysis, separation between the first brick row is obviously observable and it is significantly increased compared to the previous step. Horizontal crack between the first and second brick rows as well as between the bricks in the second brick row are formed.

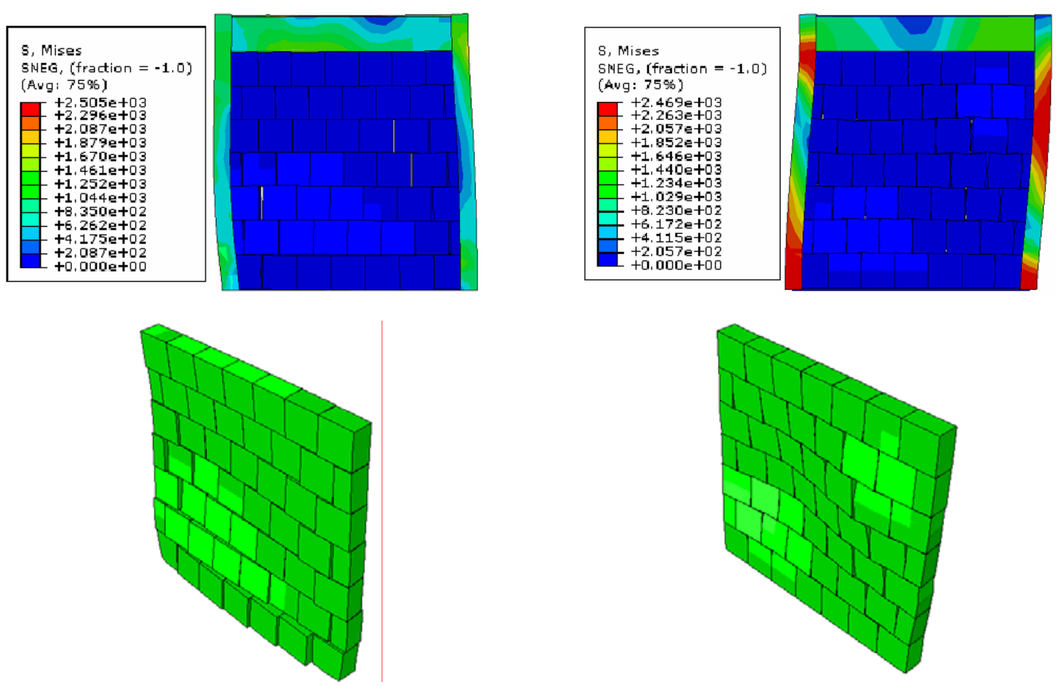


a) Dynamic pushover analysis b) Static pushover analysis

Figure 11. comparison between stresses and cracking pattern in static and dynamic pushover analyses (2% drift)

5.4. Lateral Displacement of 2.5%

In this drift ration, the values of the maximum stresses are the same for static and dynamic pushover analysis and these maximums are induced respectively at the column base and the upper beam flange. The stresses in column flange are also noticeable in static pushover analysis.



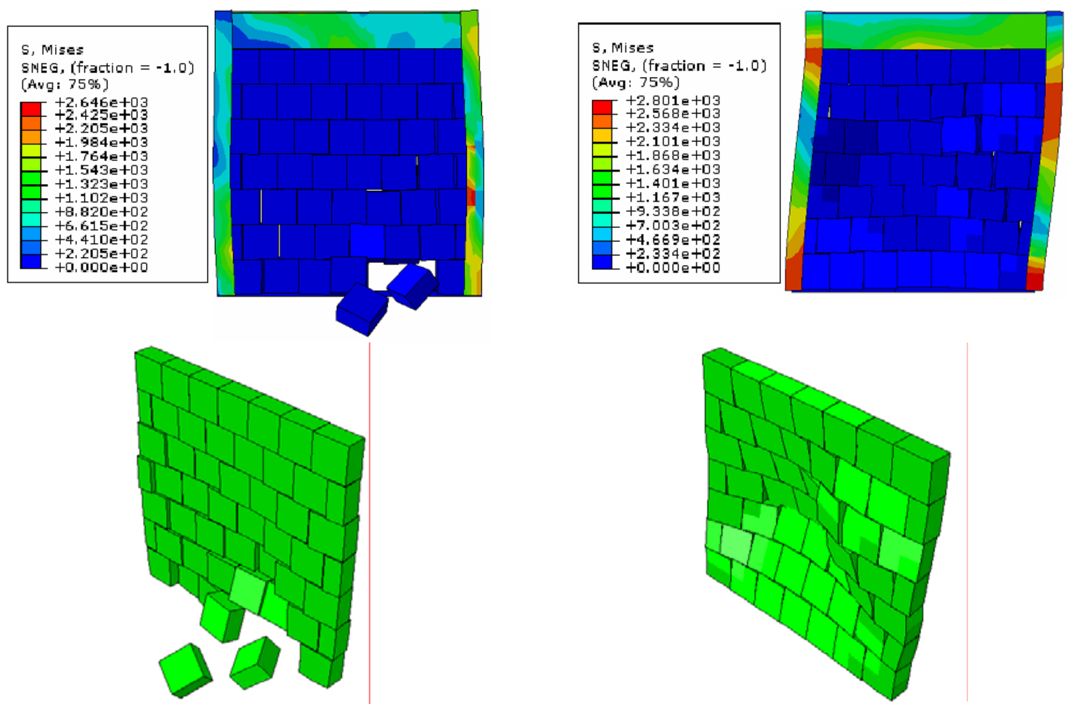
a) Dynamic pushover analysis b) Static pushover analysis

Figure 12. comparison between stresses and cracking pattern in static and dynamic pushover analyses (2.5% drift)

In the case of static pushover analysis, the width of diagonal cracks is increased in the infill and displacement is observable in middle bricks. In the case of dynamic pushover analysis, the separation value between the first and second rows is significantly increased and vertical cracks are induced between upper bricks. Considering the cracking situation, both infills are at the collapse threshold.

5.5. Lateral Displacement of 3%

Collapse is occurred at this drift ratio in both cases. In static pushover analysis, collapse occurred at the mid span of infill and in the case of dynamic pushover analysis, collapse occurred at the infill base.



a) Dynamic pushover analysis　　b) Static pushover analysis

Figure 13. comparison between stresses and cracking pattern in static and dynamic pushover analyses (3% drift)

6. CONCLUSION

Comparison between the behaviour of infilled frame under two loading cases, two directional simultaneous loading and two separated one dimensional loading, shows that in the former case the cracking pattern of infill wall as well as the structure failure mechanism is in fact a combination of the two cases where the two loads are separately applied to the structure. In other words, out-of-plane loading induces forces around the panel perimeter in infill, while in-plane loading induces forces along the infill diagonal and the combination of these two forces cause compressive stresses to be formed at the vicinity of infill base. It was observed that at the beginning of loading process, the difference between the results of two dimensional loading and loading along the infill is negligible. However, after 10 seconds from loading beginning, displacement is larger in two dimensional loading due to the damage of infill along z direction. Since the stresses were larger in two dimensional loading case, the damaged regions and failure mechanisms are different in the loading cases. Consequently, the traditional method of applying earthquake load in in-plane direction is not accurate due to the interaction of in plane and out of plane forces and it is better to simultaneously apply the in-plane and out-of-plane forces to the structure when the seismic behavior of these structures is to be studied. In this way, the prediction of finite element modeling is closer to the real behavior of structures during earthquakes.

To compare the results of static and dynamic pushover analysis, force-displacement curves obtained by the analysis methods are drawn in a same coordinate system. It was observed that the curves are initially fit each other. However, after the lateral drift ratio of 0.5%, the curves are separated and have noticeable difference. Static pushover analysis predict larger base shears than dynamic pushover analysis for a specific displacement. Considering the stress distribution condition, it can be concluded that for the studied structure, the results of static pushover analysis is similar to the results of dynamic pushover analysis with acceptable difference. However, the cracking pattern of infill in the two analysis cases are completely different. Generally, although the results of static pushover analysis are never as accurate as that of a complicated and time consuming dynamic pushover analysis, the results of nonlinear static analysis can be used for prediction of stresses and general behaviour of structure and performance levels of the structure to obtain acceptable results.

As it was observed in the current study, load-carrying capacity of a masonry infilled steel frame is much more higher than that considered in the related codes. In fact, the code values are very conservative. According to the Iranian seismic rehabilitation code, transition lateral displacement for masonry infill walls for immediate occupancy, life safety and collapse prevention performance levels are 0.1, 0.5 and 0.6% respectively. It should be noted that the definitions of code from immediate occupancy, life safety and collapse prevention are respectively formation of cracks with maximum 3-mm thickness in wall and cover, extensive cracks and crushing of some wall regions without wall displacement and extensive cracking and failure. According to these definitions and considering the figures presented in this paper, it can be seen that the values related to these performance levels are much larger than the values mentioned in the code.

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