SEISMIC REHABILITATION OF A COMMON TYPE OF URM BUILDING IN IRAN

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ABSTRACT :

Unreinforced masonry (URM) buildings represent the largest portion of residential buildings in Iran. From this stock a large percentage consists of brick buildings with partial steel members. The main structural elements of such construction are brick walls with flat-arch floors and central steel columns. The current Iranian seismic rehabilitation guidelines include sections dedicated to the evaluation of existing unreinforced masonry buildings without specific remarks about this kind of construction. Reinforced shotcrete, on one face of brick masonry bearing walls, is the most popular seismic retrofit technique of URM buildings in Iran. Two documents of evaluation; Iranian Guideline for the Seismic Rehabilitation of Existing Buildings and FEMA 356 are used for the evaluation of this type of building.

KEYWORDS: Vulnerability Analysis, Iran Masonry, Seismic Rehabilitation, Earthquake Design

1. INTRODUCTION

A large portion of Iran's older building inventory is unreinforced masonry (URM). These have been constructed in the absence of any mandatory earthquake design requirements. They are therefore most vulnerable during earthquakes. Partial steel masonry buildings (brick buildings with some steel framing) constitute a large percentage of unreinforced masonry buildings in Iran. The main structural elements of this type of construction are brick walls with flat-arch roofs and central steel columns. These buildings are built with industrial materials, and as a result, are not restricted to one specific climate or region. The main reason for the popularity of these buildings is the low cost of construction and no need for highly skilled workers.

In partial steel masonry buildings it appears that in many cases the collapse of the structure is caused by the out-of-plane failure mode. Since this type of failure can be prevented by properly anchoring the masonry walls to the floor and to the roof system, the in-plane failure of URM walls is the dominating failure mode for the URM buildings. In practice the in-plane failure mode domination may not be the actual behavior of these buildings. Therefore, the goal of this evaluation is to investigate the in-plane behavior of these buildings when the out-of-plane and diaphragm deficiencies are eliminated by some rehabilitation method. To put things in context, especially for readers unfamiliar with this type of construction a brief review of construction technique for partial steel masonry building is presented first.

2. ACTUAL METHODS OF CONSTRUCTION OF PARTIAL STEEL MASONRY BUILDINGS

In the following paragraphs various aspects of construction for the different elements of this building type have been discussed.

Starting with foundation the strip footing of this type of building, is built on the ground dug along the wall until virgin soil is reached. This means usually a depth of 50-100cm for the base of the footing. Since the walls of one and two story partial steel masonry buildings are typically 20-50cm wide, a common width of the footing will be about 60cm. For the central steel columns, the footings will be a square with 100cm x 100cm, with a 50-100cm depth. These footings are filled with a foundation paste which is usually a mixture of in-situ soil plus lime and water. If the soil does not contain enough clay, more clay will be added to the soil.

The walls are solid and are designed primarily to carry the vertical loads of the structure. In a seismic event they will have to carry lateral forces, in-plane and out-of-plane. Since they are not designed to carry any such significant forces, in a strong or semi strong quake they crack and fail. In addition these walls are often weakened by openings that are effected without any consideration of the reduction in strength that they may cause. To make the matter worse it is common to have bearing walls only on three sides of the building with big openings at front side, rendering the wall of this side totally ineffective in carrying any lateral load. For long walls, 30m or longer, at every 5 or 6 meters of the length of the wall, a square section pilaster (30cm x 30cm for 20cm walls) will be built to resist the out of plane loadings, Figure 1. The width of bearing walls in partial steel residential URM buildings is usually 20 to 50cm. Masonry walls are built with perforated or pressed bricks. Mortar used in this type of masonry building is a mixture of cement and sand which is used in higher quality, newer buildings. For older buildings, Clay or Clay and Lime, which has negligible shear strength, has been commonly used. Sometimes the “Batard” mortar, which is a mixture of Lime, Sand and Cement, is also used. The common floors of partial steel masonry buildings is flat arches, which consist of steel joists placed at 90-100cm spanned in between with flat arches.

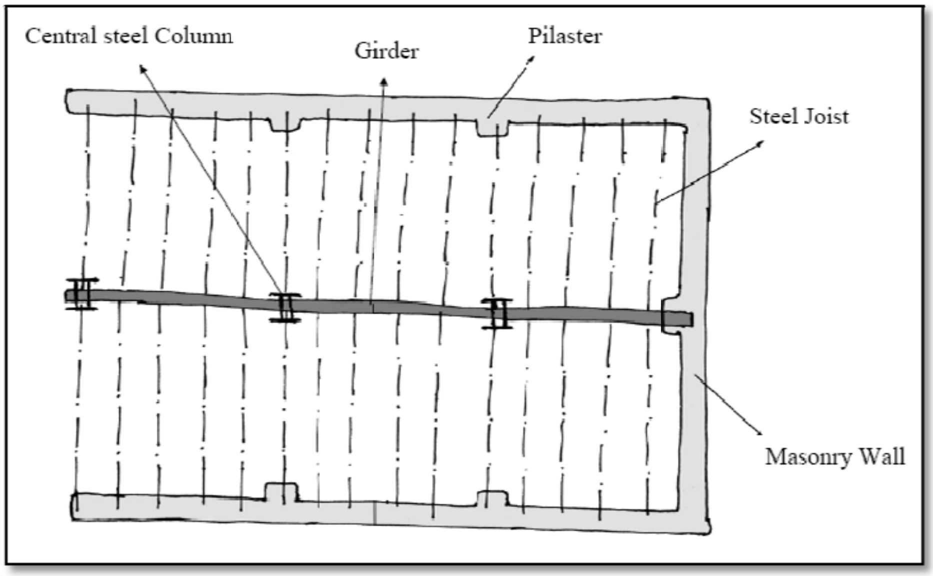

Figure 1 Typical structural plan of partial steel masonry buildings

Steel posts used in this type of building have not been based on any calculations or regulations and are usually selected by empirical knowledge. Central columns usually consist of 2-IPE 120 to 180 profiles, connected together with steel strips or cover plates. Sometimes the profiles are directly welded together. The main girders run from wall to wall, over the central columns. These beams are usually 2-IPE 120 to 160, depending on the span length and roof weight. In some cases the same profile is placed on top of the masonry wall parallel to the main girder in order to provide a seat for the floor joists. Steel joists used in jack arches are IPE 120 to160 based on span length. The aforementioned steelwork is the most common type. However, because of a lack of rules

and regulations in the design and construction of these buildings each case may demand its own investigation.
There are three types of beam to column connections used for connecting the central columns to the main girder, Figure 2. In Type-A a common type of connection in Iran called "Khorjini" is used. In Type-B the central column web is cut to let the girder pass through. In Type-C the column is cut at the floor level and the girder is placed on top of it. The steel joists of jack arch floors are connected by a single angle to the girder. If a girder is put on the wall, the other end of the joist will be connecting by an angle too. If there are no girders on the walls then the steel joist will be simply placed on top of the wall. The central columns are placed on a square base-plate of 20-40cm with 1cm thickness and attached by two or four angles, Figure 2.

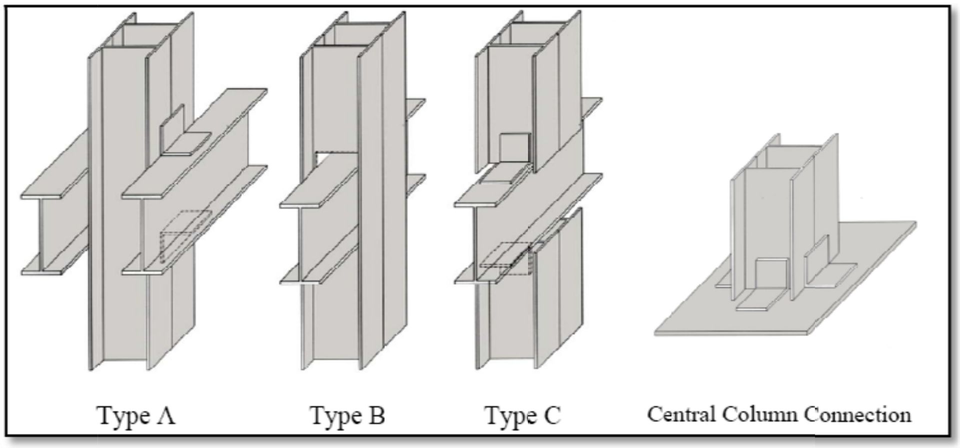

Figure 2 Different types of column connections

3. SPECIFICATION OF THE TYPICAL BUILDING

To analyze the in-plane behavior of partial steel masonry buildings, a typical building which has the common characteristics of this kind of building is presented here. The aim of the typical building was not to replicate a specific structure but rather to create a structure that was representative of common URM construction in Iran.
The typical building is a two story building, consisting of three URM bearing walls with flat-arched diaphragms at both the floor and roof levels. The dimensions of the building are 12 m by 7 m in plan, with story heights of 3 m. In the North side of the building where a large window opening is placed there is no bearing wall. Figure 3 shows the detailed configuration of such a typical building.

The building to be analyzed is assumed in fair condition with material properties per FEMA 356. Material properties are shown in table 1.

Table 1 Assumed material properties of typical masonry buildings in Iran

Masonry compressive strength (f'_m)	40 kg/cm^2
Bed- joint shear strength (v_{te})	2.5 kg/cm^2
Bed-joint sliding strength (v_{me})	$v_{me} = \frac{0.75\left(0.75 v_{te} + \frac{P_{CE}}{A_n}\right)}{1.5}$
Diagonal tension strength (f'_{dt})	v_{me}
Masonry Elastic Modulus in Compression (E_m)	28600 kg/cm^2
Masonry Shear Modulus (G_m)	11440 kg/cm^2

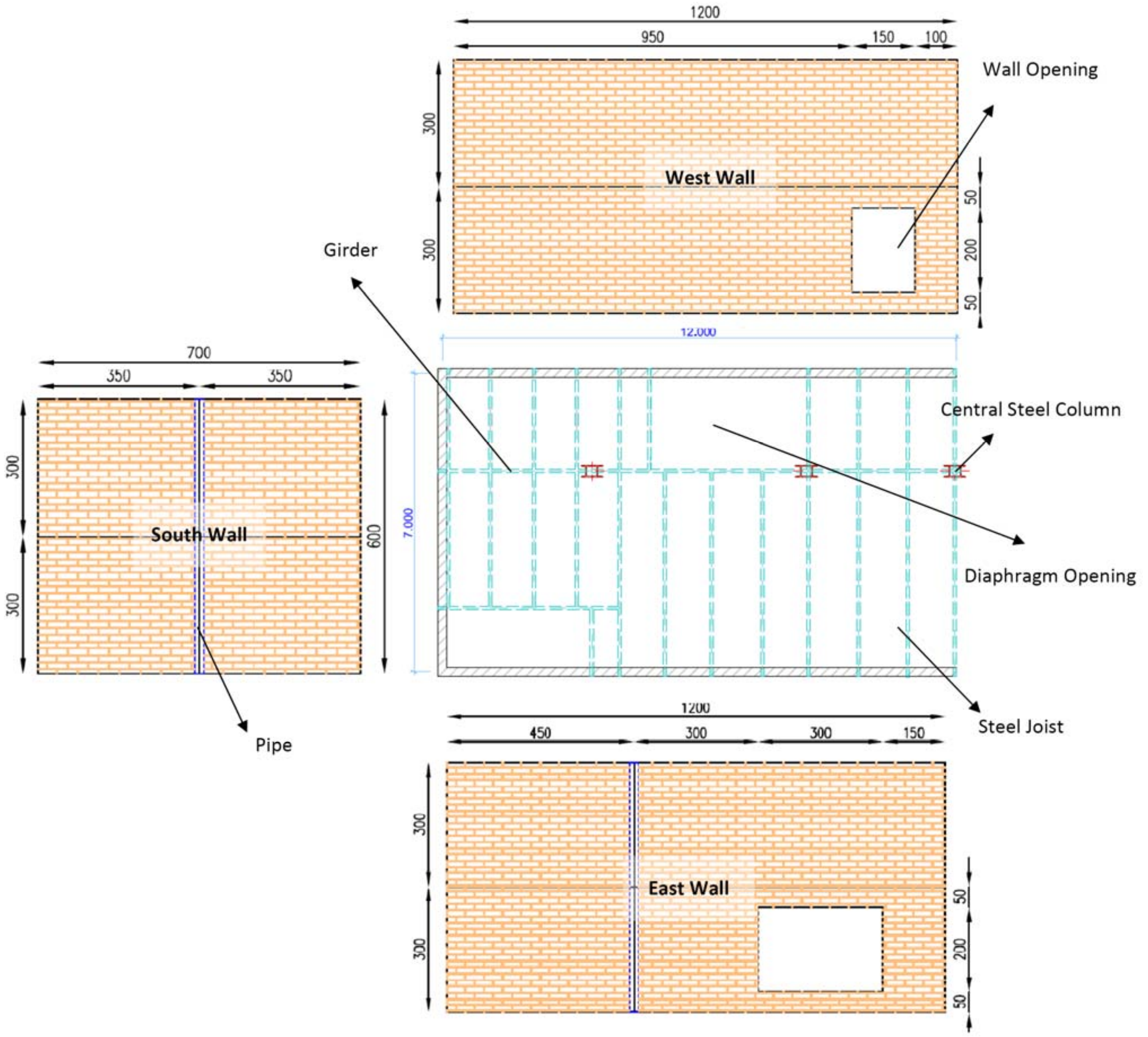

Figure 3 Structural configuration of assumed typical building

4. BASIC ASSUMPTIONS

During an earthquake, the walls may vibrate in or out of the plane. The vibration and the associated bending deformation may lead to cracking and out-of-plane collapse of the wall. If the out-of-plane motion is restricted the dominant behavior of the wall is in-plane. In this paper it is assumed that out-of-plane motion is restricted and as a result the wall will not become unstable and collapse under out-of plane vibrations.

FEMA 547 categorizes steel-joists flat-arched unreinforced masonry spanning between the joists as rigid diaphragms. It also considers this kind of diaphragm as “quite stiff as well as extremely heavy”. Because there is very limited information about how flat-arched floors have performed in actual earthquakes, the FEMA 547 assumption seems to be the most reasonable conservative choice. As we know, the distribution of the horizontal forces to the vertical elements depends on the geometry and the rigidity of the diaphragm. Since rigid diaphragms do not deform appreciably, it is assumed that their behavior remains elastic and the earthquake-induced internal forces are distributed to the vertical resisting elements in direct proportion to their rigidities.

5. SEISMIC EVALUATION OF THE TYPICAL MODEL BUILDING

The pre-standard FEMA 356 provides a full set of methodologies for the evaluation of the seismic resistance of existing buildings. In this standard, four earthquake hazard levels are defined to describe probabilistic seismic hazard. These include, the Basic Safety Earthquake 1 (BSE-1) with a 10% in 50 year exceedance level, which was selected for the model structure. The corresponding response spectrum is obtained from the Iranian Guideline for the Seismic Rehabilitation of Existing Buildings, which is referred to the “Iranian Code of Practice for Seismic Resistant Design of Buildings”. The mentioned earthquake hazard is used to check the acceptance criteria for the Life Safety performance level (LS).

The typical building described earlier, was analyzed in order to predict the in-plane response of its walls. The analysis was done based on a simplified model developed by Mid-America Earthquake Research Center. In-plane behavior of the URM wall/piers were modeled with two DOF elements with one DOF at the floor and roof levels. Perpendicular walls under out-of-plane action may increase the stiffness and strength of in-plane walls. This effect is known as “flange effect”. The latter effect was not considered here, because FEMA 356 suggests that the out-of-plane stiffness of walls shall be neglected in analytical models of the global structural systems in the orthogonal direction. As mentioned earlier roof and floor diaphragms are assumed to be rigid. The resulting model is shown in Figure 4.

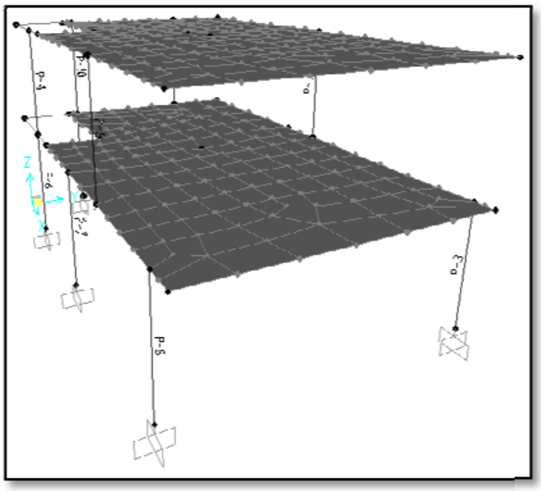

Figure 4 Left: Simplified 3D model of typical building, Right: Configuration of structural elements in partial steel masonry buildings

The stiffness of individual piers is calculated based on simplified strength of materials idealizations suggested by FEMA 356. FEMA 356 provides equations for calculating the strength of URM piers corresponding to different failure modes. It should to be pointed out, however, that these failure modes are not mutually exclusive. Failure of in-plane masonry piers are often a combination of these modes. The possible failure modes for each pier include rocking, sliding, toe crushing and diagonal tension. The first two failure modes are deformation-controlled modes, while the latter two are force-controlled failure modes. The strength of the pier corresponding to each failure mode can also be calculated. The strength and failure mode for each pier is determined from the lowest value of the four failure modes considered. The force-deformation relations of FEMA 356 incorporate the effects of ductility on deformation-control modes. Calculated stiffness and strength of piers are shown in table 2.

Table 2 Calculated stiffness and strength of piers

	Pier No.	L (m)	h_{eff}(m)	Width (m)	Stiffness $(\frac{kg}{cm})$	$\frac{L}{h_{eff}}$	Failure Mode	Strength (kg)
West Wall	P-1	12	3	0.2	832000	4.00	Bed-joint Sliding	31559
	P-2	9.5	3	0.2	624836	3.17	Bed-joint Sliding	36784
	P-3	1	2	0.2	44000	0.50	Rocking	1801
East Wall	P-4	4.5	3	0.2	200571	1.50	Toe Crushing	10745
	P-5	7.5	3	0.2	455414	2.50	Bed-joint Sliding	22038
	P-6	4.5	3	0.2	200571	1.50	Bed-joint Sliding	22460
	P-7	3	2	0.2	291396	1.50	Bed-joint Sliding	14964
South Wall	P-8	1.5	2	0.2	100286	0.75	Toe Crushing	4879
	P-9	3.5	3	0.2	122699	1.17	Toe Crushing	4550
	P-10	3.5	3	0.2	122699	1.17	Toe Crushing	4550
	P-11	3.5	3	0.2	122699	1.17	Toe Crushing	10520
	P-12	3.5	3	0.2	122699	1.17	Toe Crushing	10520

The fundamental period of the building was obtained by eigenvalue analysis of the model building and the Iranian Guideline for the Seismic Rehabilitation of Existing Buildings. The Iranian guideline uses an empirical method that gives a 0.2 sec fundamental period in both directions. FEMA 356 suggests an eigenvalue analysis of the mathematical model of the building for obtaining the fundamental period. Based on participation factors of that analysis, fundamental period in each direction of the typical building is obtained. The natural periods of the structure and modal participation factor in each direction of the model building are given in Table 3.

Table 3 Modal participation factor in N-S and E-W directions

Mode	Period	N-S	E-W
Unitless	Sec	Unitless	Unitless
1	0.199	1.434E-07	0.85
2	0.076	0.0003671	0.03214
3	0.075	0.97	0.000006088
4	0.063	0.000004419	0.11
5	0.028	0.03421	8.107E-07
6	0.025	0.000003142	0.004278

The fundamental natural period in the longitudinal direction is 0.075 second, which suggests that the considered URM structure is very stiff in that direction. The Iranian guideline does not provide a good prediction of the fundamental period in that direction. Vibration modes for the model building are shown in Figure 5.

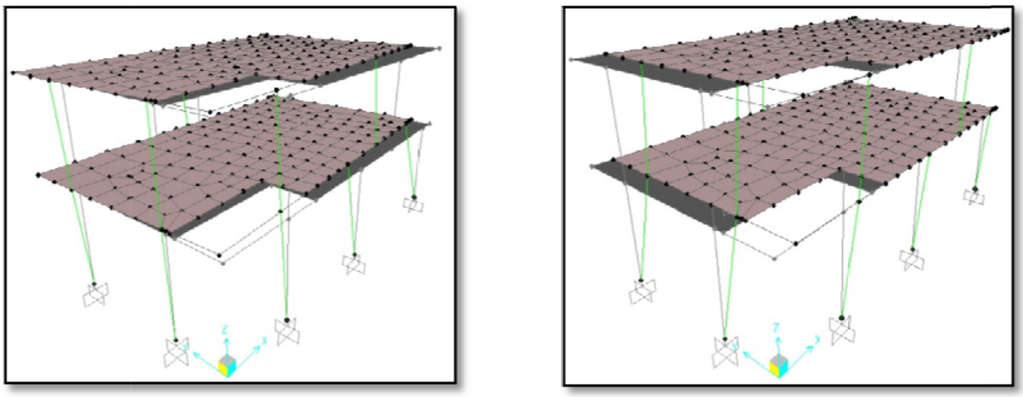

Figure 5 Vibration modes for the model building

5. RESULTS OF THE ANALYSIS

A Linear Static Procedure (LSP) is allowed for buildings without irregularities in plan and elevations. Because of asymmetric distribution of structural walls in the East-West direction of the building, severe torsional strength irregularity is presented in the structure. Therefore, according to FEMA 356 regulations, only nonlinear methods are allowed for analyzing these buildings. The Nonlinear Static Procedure (NSP) of FEMA 356 uses simplified nonlinear techniques to estimate the deformation of a structure under seismic loads. In order to use this procedure, the higher order modes of the structure should not have significant effects on its response. The URM building under consideration satisfies this requirement. Therefore, NSP was used to analyze this structure.

The pushover curve in the E-W direction shows that even if out-of-plane failure is controlled, in-plane walls in this direction will become unstable at early stages of earthquake loading. This can be seen in the E-W pushover curve of Figure 6. On the other hand a significant capacity exists in the N-S direction, Figure 6. The pushover curve in N-S direction showed more ductility, as the building pushed to its target displacement. Pushover analysis in longitudinal direction also shows that structural walls in that direction are able to satisfy the life safety performance level but failure of perpendicular walls will lead to total collapse of the whole building. Failure of the first pier during pushover analysis in each direction is shown in Figure 7.

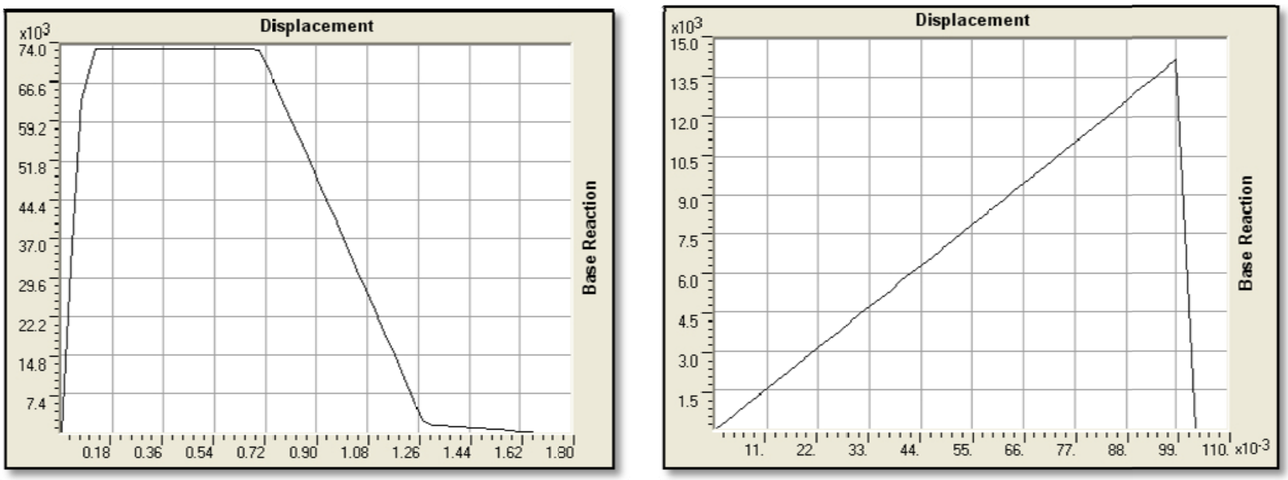

Figure 6 Pushover curves, Left: N-S direction, Right: E-W direction

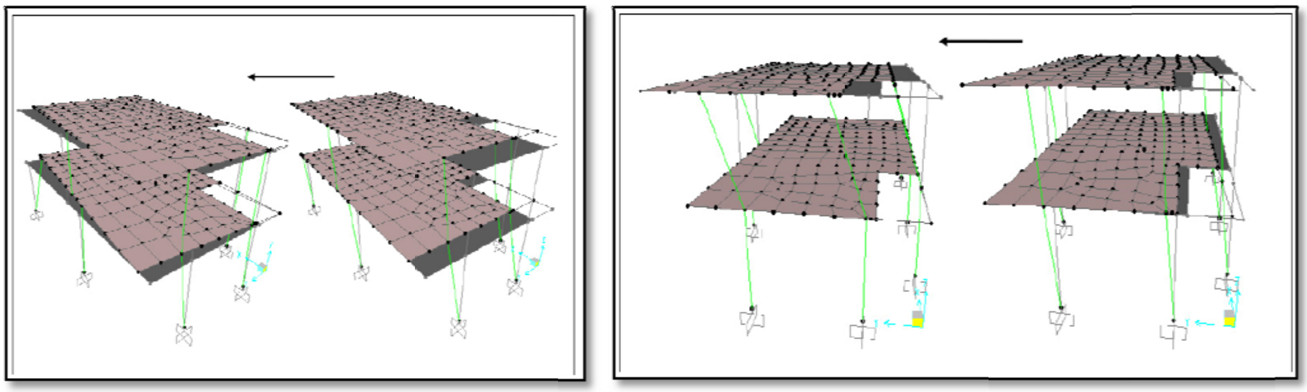

Figure 7 Failure of the first pier during pushover analysis, Left: Failure in N-S direction, Right: Failure in E-W direction

6. CONCLUSIONS

Evaluation of a partial steel masonry building, using FEMA 356 and the Iranian Guideline for the Seismic Rehabilitation of Existing Buildings, shows that:

1. The building analyzed may fail because of: Lack of anchorage between masonry walls and diaphragms, anchor failure, out-of-plane failure of masonry walls, in-plane failure of URM walls and combined in-plane and out-of-plane forces.

2. Most often the main cause of collapse of partial steel masonry building is out-of-plane failure of the walls. However even when latter weakness is removed, according to the results of the present study, moderate earthquakes can cause the collapse of such buildings, because of large in plane forces due to torsion.

The Nonlinear analysis depicts the actual behavior of the building and push over curves show a vivid illustration of the structural response during earthquake. The results of nonlinear analysis show that this type of building could not satisfy life safety performance level, even if the diaphragm and out-of-plan wall deficiencies do not exist.

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