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SEISMIC VULNERABILITY ASSESSMENT AND REHABILITATION DESIGN OF THENAGHAREH-KHANEH EDIFICE

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ABSTRACT

The purpose of this study is to assess the vulnerability of an old edifice in the Imam Reza (PBUH) Holy Shrine Complex and that a decision to be made about the rehabilitation method. This complex is not only considered as the most important religious-cultural site and an important historical heritage in Iran, but since it is also utilized 24 hours a day, so all of its buildings must be rehabilitated to meet the high performance level.
The first step for evaluating the edifice consisted of site surveying, performing ambient vibration test, shove tests, single and double flat jack tests, georadar scanning and taking thermo graphical pictures which prepared sufficient information to develop analytical models. As the second step of the study, FE models were developed and analyzed. The analyses revealed a poor performance of the entire structure and insufficient strength of the edifice. Then some rehabilitation techniques such as reducing the mass, base isolating, post tensioning the walls by cables and etc., which were acceptable for their limited interventions were studied by computer simulation and FE models, but none of them met the desired performance in the selected hazard level. So it has shown that great interventions would be needed to upgrade the level of structural performance, for instance, embedding a mega-frame inside the historical building so as to meet the life safety performance level.

Keywords: *Naghareh-Khaneh, Seismic vulnerability, Performance level, Rehabilitation, Historical construction*

1. INTRODUCTION

1.1. Naghareh-Khaneh, an old building

The edifice of the Naghareh-Khaneh is one of the structures in the Imam Reza (PBUH) Holy Shrine Aggregate. This is one of the main entrances from where the pilgrims commute. This edifice comprises of masonry structures in five storeys, in addition to the Naghareh-Khaneh Tower (Fig. 1). To the eastern section of this structure is a vast premise, which is known as the "Sahn-e-Kohne or Ateegh" (Old Courtyard).
This Courtyard is a remnant of the Safavid period with a history of more than 3 centuries.
The gravity load bearing systems of this structure is generally of masonry elements consisting of thick walls and arches; and in some floors concrete frames intervene in the load bearing gravitation system. In order to resist the lateral loads, such as, earthquakes and wind, thick walls play the chief role and the participation of concrete frames is insignificant. The aims of this article are to survey the amount of vulnerability of this historical structure to render seismic rehabilitation methods or approaches. The rehabilitation goals of this survey is to gain the life safety performance level against the level 1 hazard (10% probability of exceedence in 50 years) and collapse prevention level of performance against the level 2 hazard (2% probability of exceedence in 50 years).

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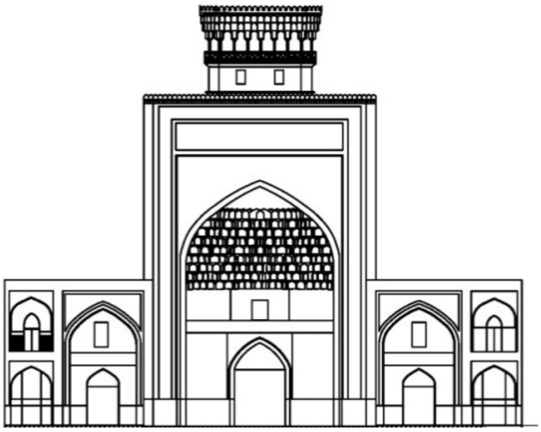


Fig. 1 Schematic view of the Naghareh-Khaneh

1.2. The behaviour of historical buildings in Iran in past earthquakes

Due to a historical richment and a civilization dating back to thousands years in Iran, various historical buildings dating to the past centuries have remained. Most of these structures that are remains are Islamic masonry structures, some of which also have a historical record of a millennium. The dispersion of these structures in various sections of the country, particularly in regions with seismic hazards, has caused the destruction of previous earthquakes to prevail in different parts of the country. A familiarity with the vulnerability aspects that have taken place in these kinds of structures can be a guidance or a lead to specify the weak points of the Naghareh-Khaneh.

Samples of these destructions are illustrated in Fig. 2. In the earthquake of Silakhur (2006), the Soltaniyeh Mosque endured structural and non-structural destruction. The minarets and the thick walls witnessed cracks at the joints of the arches and the anterior corners. Alongside the destruction of the main sections of the structure, masonry covering collapse is also observed. The destruction of this covering has no structural affect on the behavior of the building, but has an intense dangerous impact on the lives of the commuters.

In structures that have undergone a more intensive vulnerability, destructions have been vastly displayed and noted in the base and center of thick walls at the location of the joint of the main arch of the structure. This feature is observed in the edifice of the Kabud Mosque of Tabriz, which tolerated intense vulnerability in the earthquake of 1721 AH (Fig. 2b). Parts that have been repaired are shown in contrast in white. In addition to the thick walls, cracks in the crown of the arch have also been noticed. Similarly, major sections of the walls above the arches which were similar to parapets have collapsed.

By observing these destructions and other structures that have been surveyed, it is specifie that historical structures are extremely vulnerable and most of its components, such as, walls, minarets and ornamental façade and finishing may experience severe damges in moderate to strong earthquakes.

(a) Soltaniyeh Mosque of Brojerd damaged by the Silakhur earthquake (2006)

(b) Kaboud Mosque of Tabriz Damaged by the Tabriz earthquake (1721)

Fig. 2 Sample responses of masonry edifices to earthquakes

2. STRUCTURAL SURVEY

Structural specifications comprise of an extraction of as built information and a collection of information of the components of the structure, such as the location, geometrical dimensions, specification of constructional materials, general structural specialties and probable vulnerabilities.
As built information of the Naghareh-Khaneh structure has been developed and prepared. The Naghareh-Khaneh structure has five floors. The Tower on the top of the Naghareh-Khaneh, itself has three storeys. To complete the structural information visual inspection and observations have been utilized. In order to collect information from the internal sections of the thick walls, deep perforation with a lengthy drill in the thick brick walls was utilized. This information consists of the thickness of the internal layers, their types, probable crevices in the route of holes and probable vulnerability. Due to the fact that the studies took place in an analyzed form, this necessitated that the constructional materials were specified and determined appropriately. In this relative, in-situ tests for the masonry and concrete materials were utilized. The last step of inspections was the performance of ambient vibration tests to determine the natural frequencies. This test took place to control the authenticity of the software model and the comparison of the modal analysis with the test results.

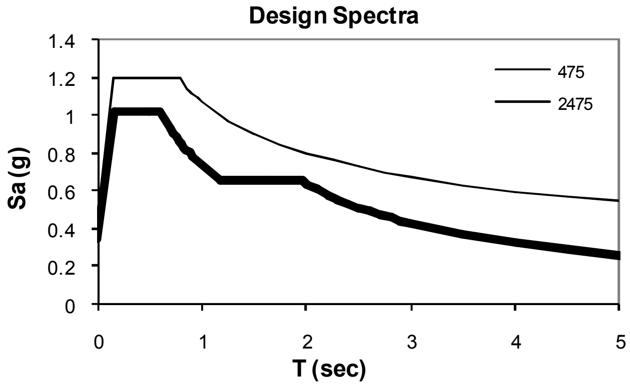


Fig. 3 The Design acceleration spectrum for two level of hazards in the site

3. SEISMIC HAZARD

Site soil properties and seismic geo-technique information were gathered, hazard analysis was performed, site design spectrums were developed and consistent artificial accelerations according to the site condition were generated. Site design spectrums have been illustrated in Fig. 3. Studies for 2 levels of probable hazards of 10% in 50 years (hazard level 1) and 2% in 50 years (hazard level 2) took place and two spectrum were extracted. The maximum spectral acceleration of hazard levels 1 and 2 that came to hand were equivalent to 1g and 1.2 g respectively.

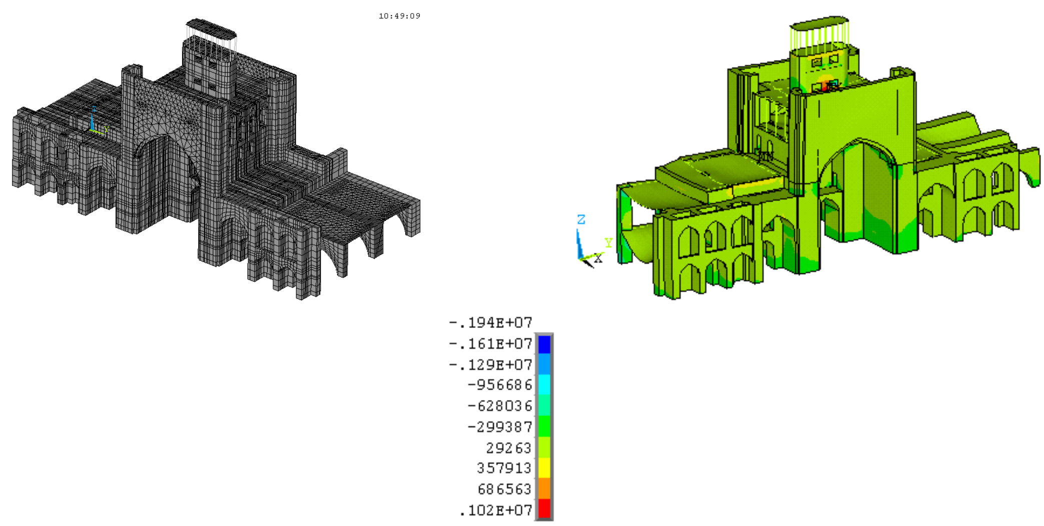
4. MATHEMATICAL MODELLING

4.1. Material Property

Specifications of the masonry construction materials are based on the amount of the average results of tests taken into consideration in the two group in two parts of the structure. The first group location consists of the space between the basement till the first roof terrace and the second part comprises of the space of the first roof terrace till the highest point of the structure. Elastic models for the lower sections and the upper sections have been determined as being equivalent to 28154 kg/cm^2 and 11399 kg/cm^2 respectively. The density of the material equates to 1800 kg/m^3 and the poisson's ratio ratio taken under consideration equalizes to 0.25.

4.2. Gravitational Analyses

Gravitational analyses by enforcing the gravity acceleration of 9.81 m/s^2 took place in a vertical direction. The analyses was assumed and performed on the base of the behavior of the elastic material. In Fig. 6 changes of the gravitational stresses are rendered. On this basis, it can be considered that the maximum gravitational stress dominant in the main walls of the Naghareh-Khaneh structure (dark green) is approximately 4 to 6 kg/cm^2.



(a) FE model of the Entire structure (b) Gravity stress contours(KN/M2)

Fig. 4 General view of the numerical model

4.3. Modal Analyses

In order to verify the FE model, the amount of the natural frequencies that have come to hand from the ambient vibration tests and the analytical model results were compared. In Table 1 the amount of natural frequencies of two experimantal and analytical methods has been compared. For each of the two directions (x direction, east-west and y direction, north-south) these comparisons have been made. In Table 1,the amount of the first mode frequency of the Tower has labled by the number of 1in each direction and the amount of the first mode frequency of the main structure of the Naghare-Khaneh Edifice has labled by the number of 2 in each direction.

By comparing the results of the periods experienced and analyzed in the main modes of vibrations under consideration, the amount that came to hand for the period in the first mode in the x direction differed by 9% and in the y direction differed by 30%. This difference, particularly, in the direction of width of the steucture(x direction) is neglegible..

Table 1 Comparison of the experimental and the analytical frequencies

mode	**Direction**	**Frequency (Hz)**		**Period (sec)**	
		Test	**Analysis**	**Test**	**Analysis**
1	Longitudinal (E-W)	2.80	2.84	0.36	0.35
2		4.30	3.40	0.23	0.29
3		10.00	6.70	0.10	0.15
4		13.00	9.48	0.08	0.11
1	Transversal (N-S)	3.30	3.09	0.30	0.32
2		4.90	4.57	0.20	0.22
3		9.10	9.00	0.11	0.11
4		13.00	12.39	0.08	0.08

5. SEISMIC VULNEREBILITY ANALYSIS

In order to analyze the vulnerability of the historical structure under survey, two methods of analyses was capable of being utilized. One method was the elastic linear method for predicting the crack initionation limit by controlling the fracture treshold criteria; and the other was the non-linear analyses by modeling the nonlinear behavior of material and using the fracture models. The first method is more reliable and much simpler but gives less information because it is valid in linear domain and only covers the linear behavior of the structure. On the other hand, complexities and insufficient information for a nonlinear analysis make it inaccurate and unrealistic. In fact the accuracy in information should be adaptable with the analytical model. Through, linear elastic analysis, this adaptability is acceptable, such that, with the following assumptions, an acceptable engineering response can be attained:

- The ductility of the structure is negligible
- In the structure under survey, non-linear behavior leads to the extensive destruction in ornamental façade and finishing which are not acceptable
- Linear analysis is valid till a major crack appears, thereby with its help the related seismic strength for the IO limit state of each part or in the entire structure can be estimated
- The standard ASCE 41-06 [2] is an important reference for the controlling the vulnerability for the existing masonry buildings. This standard considers all failure modes of masonary walls but it is definitely recommended that the failiure modes are considered as brittle ones except for the rocking mode, as the the rocking mode occurrence in the structure under survey is not expected. Hence, the control of masonry components is only determined with the evaluation of strength within the linear elastic limits.

Due to the abovementioned reasons, a surpassing elastic behavior can be taken to be as a criteria to acceptable behavior borderline and as a result, a base for the controls and designing. In the case that the response in these limits is suitable, its behavior is not vulnerable and vice-versa, it is prone to vulnerability.

5.1. Evaluation of the structural behavior control

Comparing a linear elastic system with a nonlinear elasto-plastic system and assuming equal energy as shown in Fig. 5, the amount of related strength for a specified non-linear displacement is capable of being calculated. If the drift indicated in ASCE 41-06 (Table 2) in the walls for the functioning level and the collapse limit state respectively equates to U_e and U_u, then the equivalent strength in the linear elastic system can be brought to hand. So, the equivalent strength of a life safety level is also capable of being extracted:

Immidiate Occupancy $$F_{IO} = F_1 \quad (1)$$

Life Safety $$F_{LS} = 1.158\ F_1 \quad (2)$$

Collapse Prevension $$F_{CP} = 1.225\ F_1 \quad (3)$$

In this order, if the related strength of the crack initiation is extracted in a linear model, the related strength can be estimated by the coefficient of the life safety performance level and the collapse prevention level.

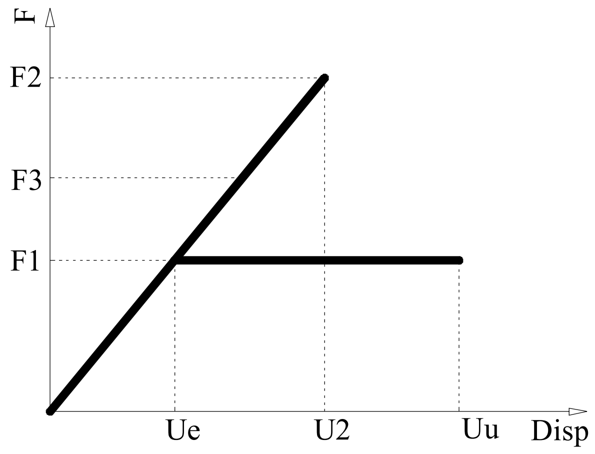


Fig. 5 Idealized model for elasto-plastic and its equivalent elastic behavior

Table 2 Acceptance criteria for masonry

Masonry Performance	IO	LS	CP
Drift (%)	0.1	0.3	0.4

5.2. Seismic Loading

A suitable method for analyzing such a structure is the modal analysis. But it has been seen that in order to attain a participatory level of 90% of mass of the structure, the number of the required modes is 800. Due to computer and software limitations, the performance of such analysis was not possible, so static simulation was utilized. The base shear according to the Iranian Standard [3] by neglecting the ductility of the structure (R=1) is calculated as hereunder:

$$V = \frac{ABI}{R \approx 1}.W = ABI.W \tag{4}$$

AB equates to an amount of design spectra, whereas, on one side, according to the importance of the structure, two seismic spectra, one for its period of return of 475 years or hazard level (1) and the other for a return period of 2,475 or hazard level (2) has been determind for the site. Therefore, the affect of importance factor **I** is automatically considered in the spectrum. In this way, instead of **ABI** an amount of **Sa** from the designing spectrum are applied.

The base shear V with a triangular distributing [3] is enforced on the structure (Fig. 6). It is better to apply a distribution of acceleration instead of load over the height of the structure because a rigid horizontal diaphragm is not there in any story.

Due to irregularities in height and plan, the seismic load taken into consideration is in a combined form in the directions of x and y. For loading the structure, eight combinations of gravitational loading with lateral loads in the form of ±100Ey ±30Ex and ±100Ex ±30Ey have been considered.

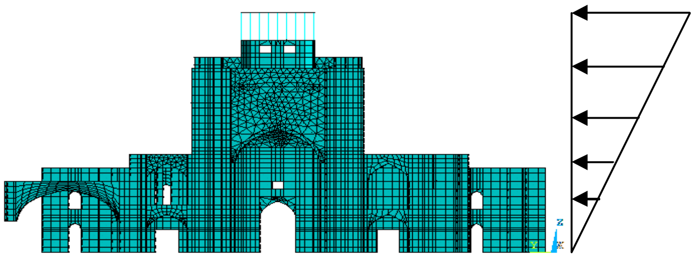


Fig. 6 Lateral loading pattern

5.3. Assessment of the crack limit stage of the structure

In order to specify the treshold limit of damage, a suitable model for the masonry material should be taken into consideration. The model surmised in this article is similar to Mohr-Coulomb Model. This model has four limits, outside of which is the damaged zone and the inside of which the considered elastic behavior zone. These four limits are considered and are according to Fig. (9). To control the structure after the performance of each analysis for a combination of loads with a specific Sa, the principal stresses are extracted and combined in pairs such as (σ_1 , σ_2), (σ_1 , σ_3) and (σ_3 , σ_2) according to the mentioned pattern. In this order by overlapping their results, vulnerable points, in relative to cracks and erosion has been drawn graphically.

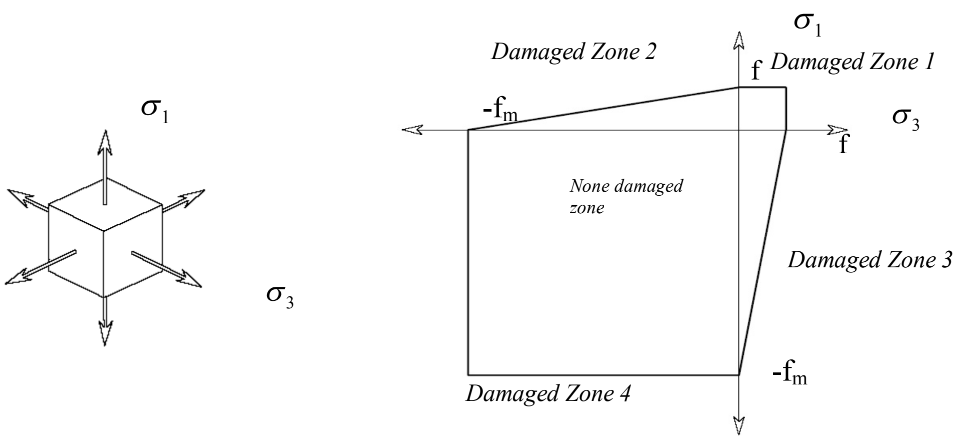


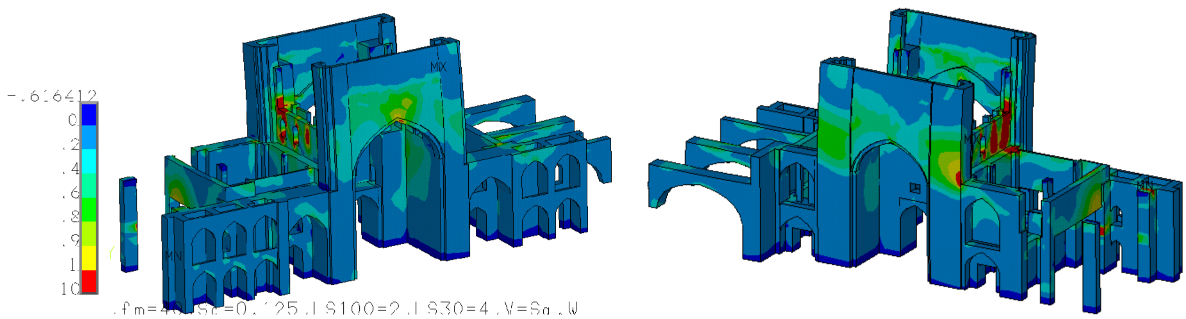
Fig. 7 Idealized model for cracking in masonry

6. VULNEREBELITY ANALYSIS OF THE EXISTING STRUCTURE

To determine the vulnerability of the structure, the amount of Sa has been gradually increased and the structure is analyzed. Then the amount of vulnerability of the structure and occurrence of damages are surveyed; and the amount of related Sa with the damage appearance in any type of members and in the entire structure are extracted. These operations take place by utilizing a macro programming. The components surveyed comprise of the main walls, main arches (two of the entrance arches), subsidiary walls and arches, including the thicker walls. The criteria of damages commence with cracks with an increased lateral acceleration on the structure. In every member an acceleration based on the amount of the damage limit come to hand by the analysis. To estimate the acceleration of the damage limit of the entire structure, the minimum acceleration of the main members is considered. This acceleration is shown by A_{IO} which is related to the acceleration of the immediate occupancy performance level. So the acceleration of the life safety performance level A_{LS} is equal to $1.158A_{IO}$ and the acceleration of the collapse prevention performance level A_{CP} is equal to $1.225A_{IO.}$ Since the cracks appear much sooner than the crushed material, the criteria of cracks dominates the resulting aspects of the structures and damages of the members. On these bases the acceleration which is tolerated by the structure in the LS level is 0.145 g and for the CP level is 0.153 g which is a very slight amount for the seismic tolerance load. In Table 3, the amounts of the colored contours in Fig. 8 are demonstrated for the two conditions of cracks in the main arch (8-a) and in the main thicker walls (8-b). The red zones demonstrate the cracked parts in which the amount of the criteria of the cracked limit surpasses 1.

Table 3 Relation between Sa and Performance Level

	Sa (g)					
	Main walls	Side walls	Wall of upper storey	Arch of the main entrance	Arch of the side entrances	Entire structure
Immidiate Occupancy	0.125	0.175	0.075	0.125	0.100	0.125
Life Safety	0.145	0.203	0.087	0.145	0.116	0.145
Collapse Prevension	0.153	0.214	0.092	0.153	0.123	0.153



(a) Damage initiation in main arch (b) Damage initiation in main wall

Fig. 8 Pattern of the initiation of damage

The numerical studies show that related accelerations of IO performance level of the Naghare Khane Edifice equates 0.125 g; LS performance level equates 0.145 g and CP performance equates 0.153 g. While the hazard level 1 is the event with the return period of 475 years and the hazard level 2 is the event with the return period of 2,475 years and according to the Fig. 3 the related spectral accelerations of the structure with the dominant periods of 0.2 and 0.23 seconds is 1g and 1.2 g in these two hazard levels, it is obvious that the difference between the demands and the capacities are considerable.

So the various methods of rehabilitation were investigated to find the most convenient method for upgrading the performance level of the structure.

7. REHABILITATION PLAN FOR THE EXISTING STRUCTURE

In order to suitably modify the structural and non-structural behavior and to prevent risks for the pilgrims during earthquakes, this existing structure should be rehabilitated by appropriate methods. So in this context, methods capable of being implemented for seismic rehabilitation for the Naghare Khane Edifice were surveyed and were primarily proposed. These are utilizing shear concrete walls in the proximities of the brick walls, reducing the masses, strengthening by post tensioning the walls internally by cables, base isolating [4], rehabilitating with embedding a mega-frame inside the main thick walls.

For this analytical modeling has been utilized; and according to the method used for the existing structure, the maximum spectral acceleration capable of being tolerated in the three levels of performance, was extracted. In Fig. 9 acceptable accelerations for the immediate occupancy of the life safety level of the building for the structure in its current condition; and various rehabilitation methods on the bases of the studies performed were rendered. Horizontal lines express the spectral requirement in hazard levels 1and 2. The amount of this difference shows that the only rehabilitation method is the concrete mega-frame that can secure the seismic requirements of structures against danger levels 1 and 2.

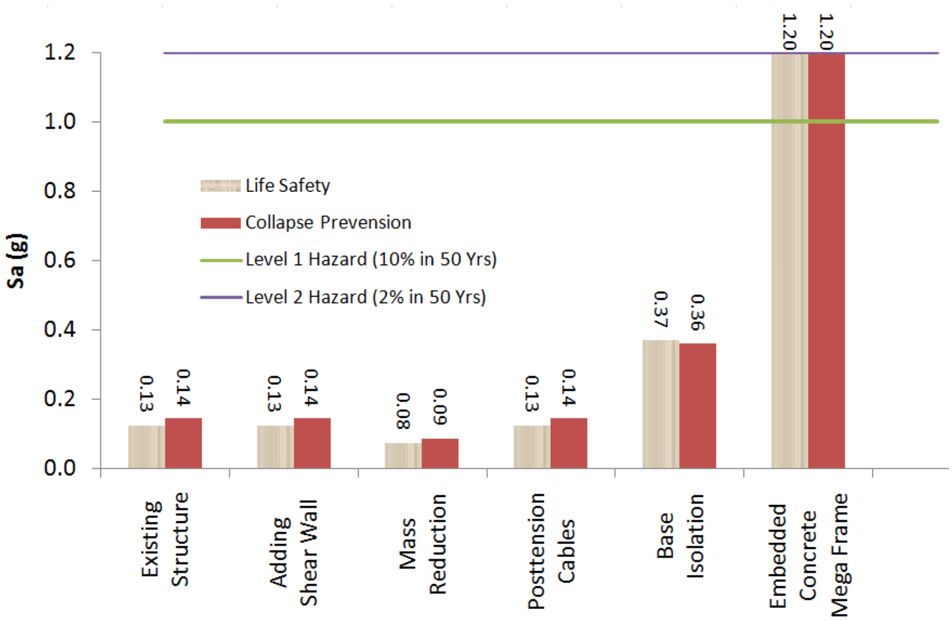
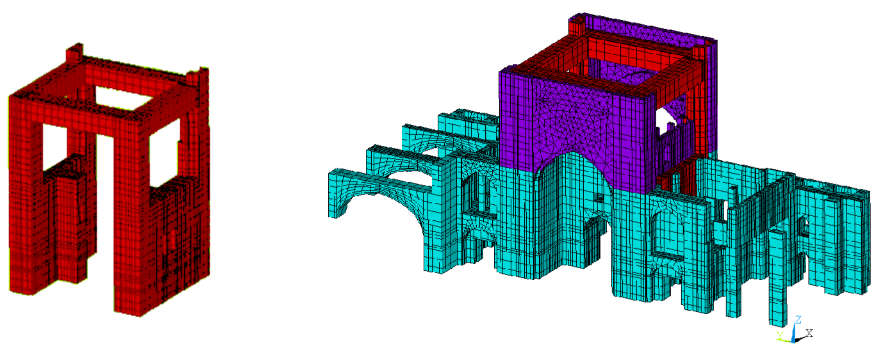


Fig. 9 Acceleration capacity of the investigated methods

Using embedded mega-frame increases the strength capacity of the structure and stiffness of the structure. It can protect the entire structure by supporting the acceleration sensitive and deformations sensitive components of the structure. On the other hand it does not disturb the architectural aspects such as forms and finishings and is consistant with the previous stabilization technique which was used for the edifice [5]. But it needs intervetion in old thick walls and in replacing the original material by reinforced concrete. It seems that outer wall around the embedded frame acts like a secondary element and is suspended to the inner frame.



(a) Mega-frame detailed model (b) Mega-frame embedded in the thick walls

Fig. 10 Embedded concrete mega-frame FE model

8. CONCLUSION

The Naghareh-Khaneh is one of the main entrance gates in the Imam Reza (PBUH) Holy Shrine. The Vulnerability assessment of the Nahhareh-Khaneh edifice showed that the structure has a very low performance level and it is necessary to be rehabilitate it so as to improve its performance level and decrease the risk of the performance failures. Many mathematical models utilized to assess the efficiency of various methods for improving the performance level of the existing structure of the Naghareh-Khaneh Edifice, such as utilizing shear concrete walls in the proximities of the brick walls, reducing the masses, strengthening by post tensioning the walls internally by cables, base isolating which were examined but none of them were as efficient as desired. So rehabilitating with an embedding mega-frame inside the thicker walls were studied. The mega-frame can support both of the acceleration sensitive and deformations sensitive components and increases the strength and stiffness of the structure, as well as to meet the high performance level criterias. The use of the embedded mega-frame dose not disturb the architectural aspects. But it requires to be involved in old thick walls.Totally, it is clear that, the high performance level for the historic structures cannot be achieved, unless the authorities accept the great interventions in some parts of the structure.

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