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Seismic Assessment of the Superstructure of the Naghareh Khaneh Edifice after Base Isolation by Simplified Kinematic Limit Analysis

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Abstract The limitation of intervention in historical buildings is one of the basic challenges for choosing a strengthening method. Base isolation is one of the best methods which can satisfy such limitations. Although there is a probability to reduce base shear and have negligible drift by base isolating, but it is very important to make sure that the integrity of superstructure reliably stands against the induced acceleration.

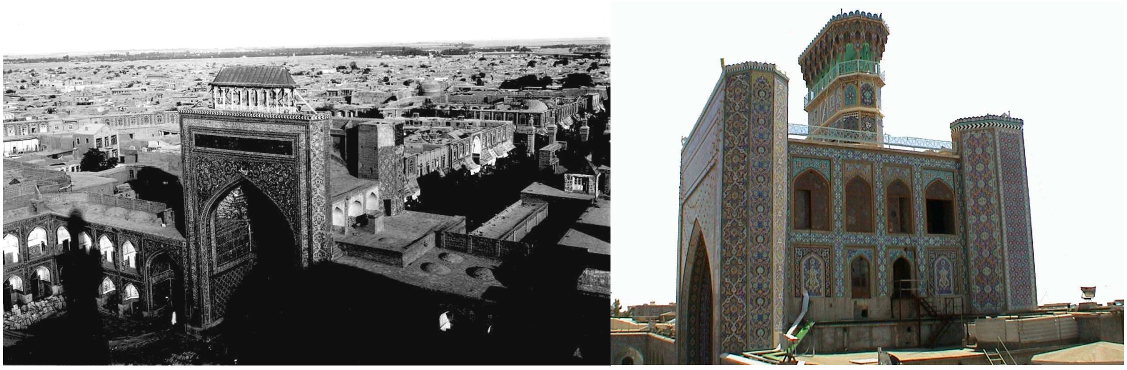
Lack of integrity leads to local failure mechanisms rather than global failure mechanisms in masonry buildings. For this reason, simplified kinematic limit analysis has been selected as a method in earthquake safety assessment.

This paper presents an investigation about the capabilities of simplified kinematic limit analysis as a complementary method, for the seismic safety evaluation of the superstructure of a base isolated masonry historical construction. This method has been applied on the “Naghareh Khaneh” edifice. The Naghareh Khaneh is a masonry historical building in Iran which was constructed around 400 years ago. The main body of this study focuses on the vulnerability assessment of the superstructure after using isolators on the base and the efficiency of simplified kinematic limit analysis will be discussed which shall be based on the results obtained from the out of plane and in plane behaviour of walls after installing isolators and dampers on the base of the Naghareh Khaneh structure.

Keywords: Seismic safety, historical building, kinematic limit analysis, Naghareh Khaneh

Introduction

Naghareh Khaneh edifice is one of the main buildings of the holy shrine of Imam Reza (PBUH) in Mashhad, the second largest city in Islamic Republic of Iran. Imam Reza (PBUH), the eighth Shi'ite Imam, was buried in this place after his martyrdom on the 5 September 818 A.D. The mausoleum of Imam Reza (PBUH) is the most important site of pilgrimage in Iran and many courtyards, buildings and bazaars have been constructed and developed around it during its history. This edifice is one of the main entrance gates of the oldest courtyard adjacent to the Imam Reza’s tomb and it has been built around 400 years ago (aqrazavi.org 2010). There is a tower on the top of the Naghareh Khaneh which is used to play a special music at the sunrise and the sunset time to announce the prey time to the pilgrims. Fig.1 shows the changes which has made during 1960’s in this building. As it is shown, the old music tower was a light wooden tower placed on the parapet of the edifice but the new concrete tower placed on the new concrete floor and columns on the roof of the old edifice. The dimensions of the plan of this building are about 17m×32m and its height is 32m. Many studies have been done during the recent years to find the best method to increase the seismic performance of this building while keeping it operational and minimizing the required intervention in old facade. Base isolation method seems to be one of the practical methods which could be used in this building but the question is how much intervention in superstructure should be done after base isolation? In other word, is it possible to design the base isolation system in a manner which no intervention in facades become required?



(a) (b)

Figure 1: View of Naghareh Khane edifice, (a) old wooden tower, (b) new concrete tower and storey

Base Isolation Design

Limitation in interventions in historical constructions from the aesthetics point of view is an important matter. Seismic isolation requires some interventions in the base level of the structure and it might have minimum external representation, whereas it reduces earthquake forces by the lengthening the fundamental period of structure.

According to the general considerations (Mayes and Naeim 2001) this kind of structure is suitable for seismic isolation because 1)the site soil does not produce a predominance of long period based on geotechnical report of site, 2)the structures has more than two stories and it is so heavy, too, 3)the adjacent structures permits more than 8 inch horizontal displacement although it may be need to do some special details, 4)structures is fairly squat and 5)wind lateral loads is less than 10% of weight of structures. So can say it is a suitable method for rehabilitation of historical constructions like the edifice under question. In this section the calculation of imposed acceleration to the superstructure after using base isolators and dampers on the base level has been presented briefly. The general information and assumptions of the analysis and design are that the lateral loads resisting system of this building consists of unreinforced masonry structures(R=1.25) (ISRM-85 2006), the center of mass and center of rigidity is considered coinciding, the project is located on a site with soil class III according to Iranian code (Standard No.2800-84 2005) corresponds to the Class E. of IBC-2000, the anticipated damping is about 30% of critical damping and a margin of $\pm 10\,\%$ variation in stiffness from the mean stiffness value of the isolators is considered acceptable. The earthquake deign spectrum for Naghareh Khane site has been presented in Fig. 2.

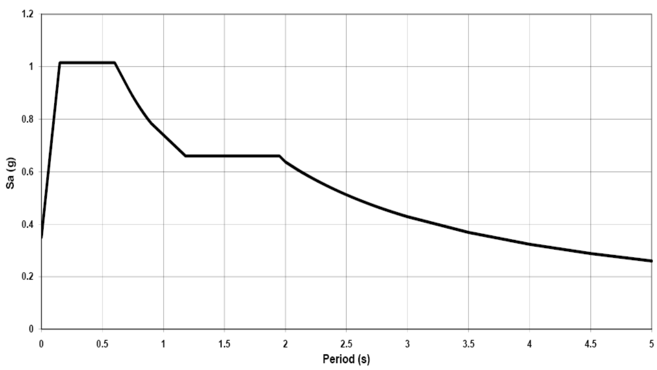


Figure 2: 5% damping Design Spectrum for Naghareh khaneh Site

Structure above the isolation system should withstand the shear force, Vs, (Mayes and Naeim 2001):

$$V_S = \frac{K_{D\max} D_D}{R_I} \tag{1}$$

In above equation DD is the design displacement of the center of the rigidity of the isolation system at the design base earthquake (DBE), KDmax is the maximum effective stiffness of the isolation system at DBE in horizontal direction and RI is a reduction factor analogous to the R factor of superstructure. After substituting the parameters, Eq. 1 becomes:

$$V_S = \frac{1.22 S_a}{B_D} W \tag{2}$$

In above equation Sa is spectral acceleration at the effective isolated period (TD), W is the total mass of the building and BD is the damping reduction factor for DBE which is calculated by Eq. 3. (Naeim and Kelly 1999)

$$B_D = \frac{1}{0.25(1-\ln \beta)} = \frac{1}{0.25(1-\ln 0.3)} = 1.81 \tag{3}$$

By consideration of the fix base shear force according to the Iranian code and RI=1 it is concluded that the superstructure should endure against the acceleration of:

$$a_{TD} = 0.67 S_a \tag{4}$$

On the basis of the above explanation the value of DD, KDmin and aTD for different TD has been given In Table 1.

Table 1: D_D, K_{Dmin} and a_{TD} base on different T_D after isolation

T_D(s)	D_D(cm)	K_{Dmin}(N/mm2)	a_{TD}/g
1.5	20	156	0.44
2	35	88	0.43
2.5	44	56	0.34
3.5	62	29	0.25
4.5	80	17	0.19
6	99	10	0.13

In Table 1, it has been shown that considering the target TD lengthening from 1.5sec to 6sec will be caused a reduction in imposed acceleration aTD from 0.44g to 0.13g. Although the smaller aTD is seems better but it is related to the greater DD. Actually the lateral movement of the structure should be reduced to prevent the pounding to the adjacent buildings. The limit of 40cm is acceptable for the displacement of this structure, but the related aTD in Table 1 and the shape of the spectrum in Fig 2 show that the results are the same from TD=1.5 to 2sec and noticeable acceleration imposed to the superstructure. It means that a special damping system is necessary to achieve desirable displacement with smaller stiffness and imposed acceleration. To evaluate the variety of design targets TD, the integrity of the super structure related to each targets TD were investigated through the next section.

Simplified Kinematic Limit Analysis

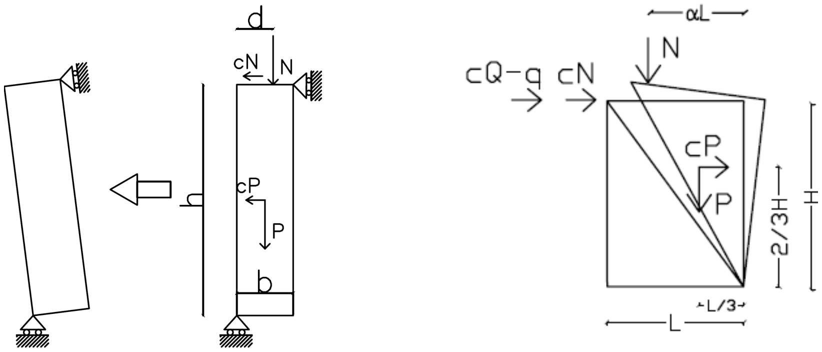
Limit analysis method is useful for analyzing and designing the strengthening of masonry construction and historical buildings due to the simplification of assumption, reducing number of necessary mechanical properties and giving an ultimate condition of structure in critical section. It must be stressed that even though advanced modeling is a proper solution to make deep understanding of historical construction but it is less effective for designing strengthening but simplified modeling such as kinematic limit analysis is a great means for masonry construction in any case(Lourenco 2002). Evaluation of the ultimate load bearing capacity of entire masonry buildings subjected to horizontal loads is a fundamental task for the design of masonry buildings in

some code, for instance the recent Italian O.P.C.M. 3431 paid attention to simplified kinematic limit analysis. (Giuffrè 1993, O.P.C.M. 3431 2005)

The limit analysis has good accuracy in ULS and can be a proper method to estimate capacity of load bearing of masonry structures. (Lourenco 2001).

Because of disintegrated behavior of masonry buildings, generally local failure mechanism caused to the collapse of these construction. It means it is possible to prevent collapses of masonry buildings by preventing of local failure mechanisms. It must be pointed out that the failure mechanism of a building is not unique and if a critical failure mechanism is not be studied the safety factor which is found by another failure mechanisms might be higher than the real safety factor.

Kinematics models analysis calculates the collapse coefficient c=a/g, which corresponds to the seismic mass multipliers for the considered mechanism. It is possible to compare collapse coefficient (c) and seismic base shear coefficient according to the local standards. Fig. 3 illustrates two kinematics models and related c coefficients. (Binda et al. 2006).



$$C = \frac{P \times \frac{b}{2} + N \times d}{P \times \frac{h}{2} + N \times h}$$

(a)

$$C = \frac{N \times \alpha L + P \times \frac{L}{3} + q \times H}{N \times H + Q \times H + P \times \frac{2}{3} \times H}$$

(b)

Figure 3: Kinematics models: a) out-of-plane, b) in-plane overturning, of a solid wall simply supported to roof (Cardani 2004)

To study these kinematic models it has been assumed that: 1) Masonry has no tensile strength due to the weakness of mortar and bond behavior between mortar and brick, 2) Masonry has an infinite compressive strength. Specially In ancient masonry structures the compressive stresses are usually small compared with the corresponding strength, so, the probability of crushing failure is less although in some new research this assumption was discarded (Bustamante 2003), 3) Failure occurs under small displacements and also volumetric weight of masonry material is considered 1850 kg/m^3 according to the (INBC-06 1996).

On the basis of the above explanation 21 failure mechanisms has been selected for safety assessment of Naghareh Khaneh edifice, 12 wall overturning, 7 in plane and 2 sliding failure mechanisms have been studied and These are shown in Fig. 4. It must be notify that failure mechanisms of massive masonry piers are not presented here because of their stable performance in preliminary studies.

The collapse coefficient (c) of each mechanism has been calculated and the results are presented in Table 2.

There is a direct relationship between the collapse coefficient (c) in Table 2 and the acceleration (aTD) in Table 1. In fact in each case of the Table 1, it is expected that the superstructure experiences the acceleration of (aTD) and in this situation the safety of those failure mechanism (in Table 2) could be guaranteed that in which the (c) coefficient be greater than (aTD). As it is obvious, even by considering TD =6 sec and DD =99cm it is not possible to get rid of all of the failure mechanisms and it means that some rehabilitation efforts on superstructure is inevitable. On the other hand, according to the allowable horizontal displacement (40 cm) the maximum TD could be selected 2s, so after base

isolation an imposed acceleration of 0.43g to the superstructure is expected and consequently 40% of failure mechanisms (M5, M7, M12, M14, M16, M18, M19, and M20) are safe and strengthening against the rest of failure mechanisms of superstructure is necessary.

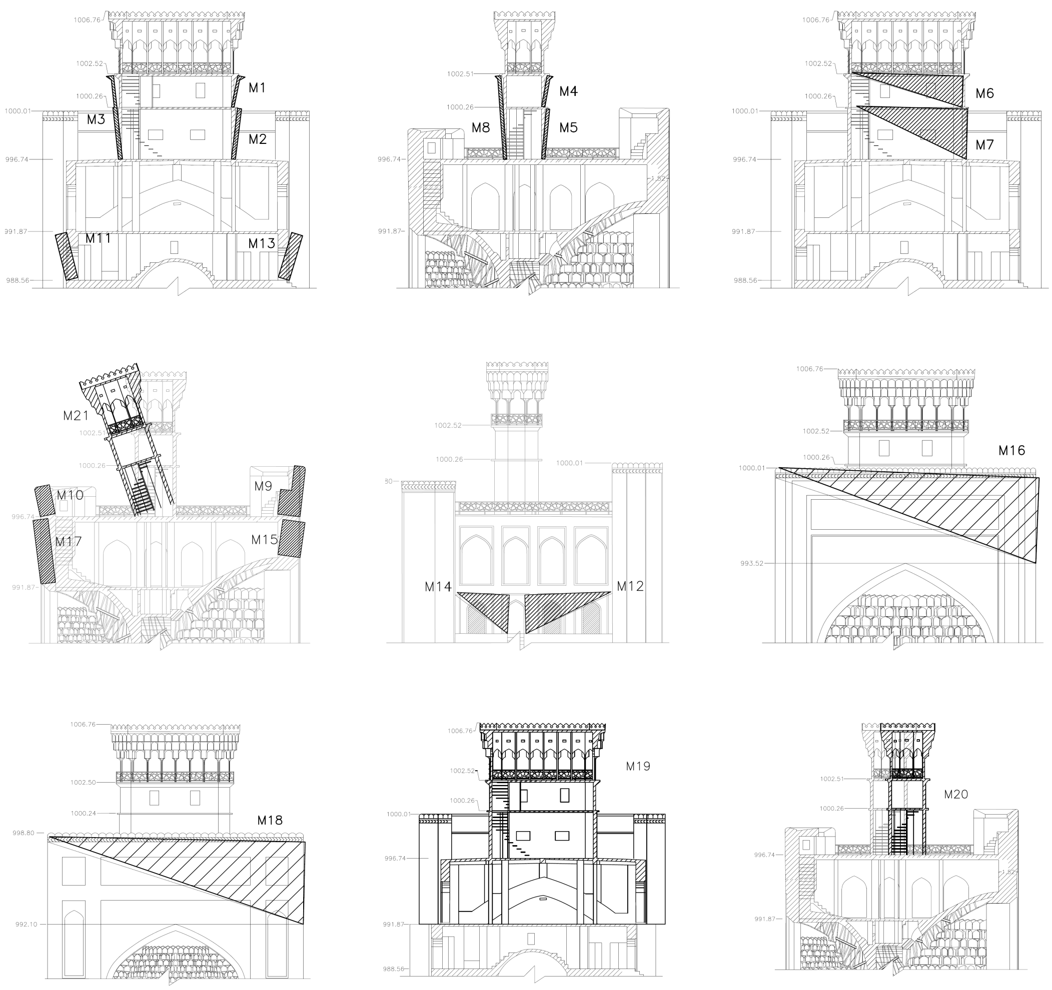


Figure 4: 20 Failure Mechanism considered for Naghareh Khane edifice

Table 2: Safety factors for mechanisms

Mechanism No.	*Collapse Coefficient(c)*	*Mechanism No.*	*Collapse Coefficient(c)*	*Mechanism No.*	*Collapse Coefficient(c)*
M1	0.056	M8	0.03	M15	0.27
M2	0.06	M9	0.21	M16	1.95
M3	0.05	M10	0.35	M17	0.15
M4	0.07	M11	0.16	M18	2.26
M5	2.19	M12	1.23	M19	4.3
M6	0.04	M13	0.16	M20	8.3
M7	1.34	M14	0.73	M21	0.3

If it is desirable to minimize the intervention of the superstructure it is evidence that the allowable displacement of the superstructure must be increased or especial dampers should be designed to reduce the displacements of the superstructure to 40cm.

Conclusion

In this paper, seismic safety assessment of Naghareh Khane edifice after base isolating has been investigated by using simplified kinematic limit analysis method.

In one hand 21 Failure Mechanisms have been considered for Naghareh Khane edifice and collapse coefficient (c) was calculated base on kinematic limit analysis and in the other hand imposing acceleration(aTD) correspond to the maximum allowable horizontal displacement has been calculated after using base isolators.

Regarding to the results, only 40% of failure mechanisms are found to be safe so strengthening of the superstructure would be necessary after using base isolators.

It has been shown that, according to the desirable level of intervention in the superstructure, considering the results of the kinematic limit analysis would lead to find the target desirable period (TD) for designing the base isolator system and also it is indirectly useful for defining the target damper system characterization which is needed for reducing the response of the structure to what would be acceptable.

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