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Seismic Vulnerability Evaluation of Historical Buildings by Performance Curves, Case Study for Ramsar Museum

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ABSTRACT: The purpose of this paper is to produce a systematic way for structural analysis of unreinforced masonry buildings. Because in analysis of historical masonry we face to lots of problems, such as: the geometry is usually missing, the constitution of the inner core of the structural elements is unknown, a complete mechanical characterization of the materials utilized is hardly possible, the sequence of construction is not documented and the building processes vary substantially from one period to another as well as from one site to another , finite element discretization, knowing mechanical properties of materials and simplification of finite element models. One of the questions researchers face in seismic analysis of historical building is that, how much of demand- capacity ratio can acceptable to assessment level of damage on the basis of linear- elastic analysis? By introducing such a limitation, useful simplification has been done which leads to save expense, time and easy interpretation of results. In this paper, by case study of a historical building of Ramsar museum, first site's seismic demand has been estimated for different hazard levels then the structure has been analyzed finally it has been controlled by suggestive criteria in research. At last it's cleared that limit criteria and introduced performance curves are practical and could be used as a suitable tool to develop planning purposes of researchers who work on analysis of historical buildings.
key words: Ansys, Finite Element Method, Historic Structures, MATLAB, Seismic Hazard Analysis, Time Response Analysis.

INTRODUCTION

Seismic vulnerability evaluation of historical masonry structures which are often unreinforced, is a hard work. Time to make mostly of these structures refers to the time before industrial revolution, steel and concrete hadn't been used. Lacking today's materials to establish historical building, mostly had been used materials such as brick and mortar in order to be load bearing compressive stresses also wood to tensile stresses. After several decades, wood have been destroyed by termite and worn out. By available way the architect makes possibility to reach to rotten wood situation and replacement them with new wooden lumbers , so the structure is retrofitted. In some cases reaching to forms such as arches, domes, vaults also basic static's principles like thrust lines had been tried to use maximum compressive capacity of materials and to make the least tensile stresses which is one of the weakness of unreinforced concrete and masonry structures materials, Brilliant succeeds has been obtained. These kinds of structures mostly have a good load bearing against gravity loads, moreover they usually haven't enough resistance against seismic loads (Hejazi, 1997; Sadeghi et al.,2010).

Considering told restricts also economic and computer ones, researchers who work on historical structures analysis, need to Simplifications in engineering in order to provide modeling possibility. For instance, Simplified Kinematic Limit Analysis way which is a simple one and its calculations can be done easily by use of basic principles of static's, is an answer to these group of researcher's requirement (Climent et al., 2010).

This method has been brought in Italian O.P.C.M 3431 ordinance in 2005 and considered it permissible for seismic Assessment and retrofitting plan of masonry structures(OPCM 3274/03,2005). Therefore, it seems that simplified methods which we produced in this field will be welcomed by researchers. In current search also has been tried to show such a way. The researchers who have had many obtaining in historical building structural analysis are :Oliveira, 2006; Ayala, 2002; Lurenco, 2001; Betti, 2012; Roca, 2010 and Mistler 2006.(Roca et al., 2010). Ghanaat proposed a methodology for damage estimation in concrete dams which can be found in USACE guideline(EM 1110-2-6053,2007). In this guideline a systematic method based on linear time history result in term of local and global performance indices was suggested. Empirical performance criteria are defined in term of these indices and they form a basis for qualitative estimation of damage. In Bayraktar et al(Bayraktar et al.,2010)studies, he and his co-workers evaluated seismic performance of concrete gravity, arch, RCC and CFRD dams using indices proposed by USACE subjecting to near and far fault ground motions. In addition, Ghanaat studied Morrow point and Pacoima arch dams performance under various earthquake using his proposed criteria. many study have been done on the seismic performance of concrete dams by many researchers (Sevim, 2011; Hariri Ardebili, 2011 & Esmaili, 2012) by utilize mentioned methodology.

In this research, for the first time, a systematic way has been produced to time response analysis of unreinforced historical structures which its innovation of this research is performance curves. A designer can answer the following question which is the idea of this study. how much of demand- capacity ratio can acceptable to assessment level of damage on the basis of linear- elastic analysis? There are three kinds of responses including: yes, by a good approximation; yes, by an acceptable approximation and no. If the designer faces the first two answer, innovation of this research have many advantages like saving expense, time and modeling. we can seek the present innovation concepts in FEMA 273 and EM 1110-2-6053 from U.S.A military. The structure of current research consists of four general phases. First seismic hazard analysis has been done. In the second phase mechanical properties of materials have been determined by some destructive and nondestructive testing of materials; The third phase consists of discretization of model and structural analysis, finally a systematic way has been suggested to determine the correctness of time response linear analysis results in order to evaluate damage level of structure. Moreover, after probabilistic seismic hazard analysis (PSHA), by written code in MATLAB software, making parametric model with APDL, the model has been analyzed under different loads combination.

Site Specific Seismic Hazard Analysis (PSHA)

Ramsar, one of the northern cities in Iran, has 50.646 longitude and 36.903 latitude. Figure 1 has shown satellite image of the city. The study area has been considered a circle with 200 km in radius to determine seismicity and seismic sources. Seismic hazard analysis commonly is done by two ways; deterministic and probabilistic. In this case study, used of 177 earthquake records in bank information for seismic hazard analysis consists. In figure 1, the study area of seismicity trend has been shown for event with the magnitude greater than 1 (left Figure), greater than 4.5 (middle) and greater than 6 (right Figure). By Studying all active faults in this circle area , finally adopt two line source and three Area ones. In this research spectral attenuation law has been used. Mentioned spectral attenuation law and strong ground motion has been obtained near source events. the database, used as the input for spectral attenuation law, consists of 89 three components accelerograms, recorded between 1975 and Dec. 2003 by National Iranian Strong Motion Network. (Equation 1).site soil type has been considered IV(Zare et al.,2008 ; Kramer,1996).
b2, b3 are fixed dynamics and b1-1 till b1-5 are related dynamics to the site.

Equation 1:
$$\ln Sa(T) = b_1(T).S_i + b_2(T)(M-6) + b_3(T)(M-6)^2 + b_5(T).\ln R + \pm S_a(T).P$$

The summary of seismic hazard analysis results has been shown in Table 1 forming that peak ground acceleration with the probability of occurrence for site which has been used in loads combination of current study. In Deterministic Seismic Hazard Analysis (DSHA), the probability of occurrence given mean and standard deviation estimated 0.16g and 0.25g respectively. PSHA for two hazard levels I and II by earthquake return period of 475 and 975 also estimated 0.34g and 0.65g respectively. The results of PSHA have a greater amount which is used as seismic demand in this case study.

Figure 1.satellite image of Ramsar – by software (Google earth)

Figure 2 .trend of seismicity study area.

Table 1 .summary of seismic hazard analysis

Seismic Hazard Analysis Method	Seismic Hazard levels Probability of Occurrence	PGA
DSHA	Mean	0.16g
	Mean + σ	0.25g
PSHA	10% probability of exceedance in 50 years	0.34g
	5% probability of exceedance in 50 years	0.65g

Seismic Vulnerability Evaluation

Ramsar museum is the most beautiful historical buildings in Iran. The structure refers to the two centuries ago. Its architecture had been designed one floored underground to residential use. Its view made by marble onyx with the approximate thickness of 30 cm, four stone piers with the approximate height of 10 m, is the only sample of beautiful elements in this palace. (Figure 3) Drucker-Prager yielding criteria and William-Warneke criteria is Two criteria that utilized in this study for nonlinear time response analysis.(table 2)

DETERMINATION MECHANICAL PROPERTIES OF MATERIALS

Destructive and nondestructive tests have been done to get to the physical properties materials of the building including: schmidt hammer test, mortar shear strength test , brick and stone tensile and compressive strength test which can be seen in figure 4. Moreover , Italian O.P.C.M. 3431 ordinance's guides had been used to choose minimum and maximum amount of elasticity modulus and mass per volume of bricks with lime mortar and regular cut stones(OPCM 3274/03,2005). In Table 4 material physical properties and used criterions parameters can be seen .It can be said the elasticity modulus of stone piers had been considered 20Gpa. William- Warneke and Drucker- Prager criterion for modeling had been used by nonlinear material behavior(Sadeghi et al.,2010).

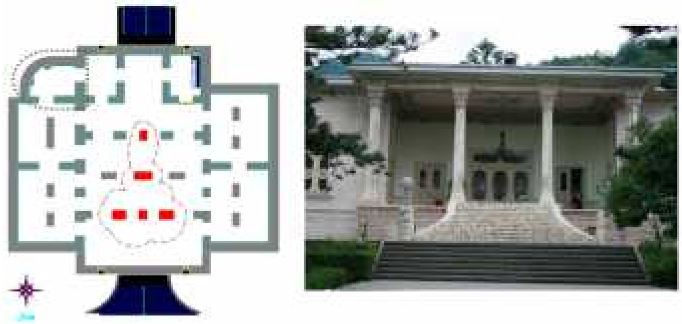


Figure 3. Northern view and underground plan of the Ramsar museum

Figure 4. Destructive and nondestructive tests

Table 2.Mechanical and physical property of material

material physical properties	Drucker-Prager yielding criteria	William-Warneke criteria
$1.8\,{}^{N}\!/\!_{mm^2} < E < 2.2\,{}^{N}\!/\!_{mm^2}$	$_t = 0.05$	$C = 200000Pa$
$°_{full\ brick\ masonry} = 1800\,{}^{KN}\!/\!_{mm^3}$	$_c = 0.95$	$¨ = 30^\circ$
$°_{cut\ stone\ masonry} = 2100\,{}^{KN}\!/\!_{mm^3}$	$±_t = 0.35\,\mathrm{MPa}$	$« = 15^\circ$
$¬ = 0.2\,;\,£ = 0.002$	$±_c = 3.5\,\mathrm{MPa}$	

DISCRETIZATION BY FINITE ELEMENTS (BY MACRO MODELING METHOD)

in macro modeling the units are neglected and the body of the masonry structure is modeled as a homogeneous material. Therefore, regarding physical characters of used materials, three dimensional modeling in Ansys software and different analysis on the model is done. used elements in three dimensional modeling are isotropic with 8 nodes per element. FEA model can be seen in figure 5. Model contains 10027 elements and 3120 nodes.

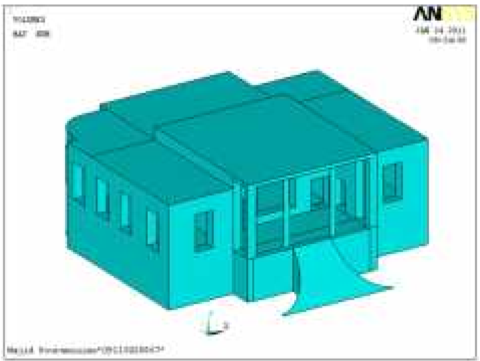


Figure 5 .numerical model of structure

The Analysis Of The Structure Under Self Weight

Entire weight of the building has been counted 527 ton. According to 2800 Iranian code, equivalent static shear force for the structure has been obtained 208 ton. according to Iranian 2800 code shows that the relative wall in two directions has been twice as much as suggested with 2800 code(Standard 2800, 2005).

Relative wall in north – south side for underground and the ground floor has been calculated 14.2 and 10.9 respectively; for the east-west direction has been calculated 12.8 and 9.5 respectively. However it also contradicts standard rules in some cases. Greatness of sums of openings length in comparison with half of the walls length , the distance of the first opening from the external side of the building being more than 0.67 of height of the opening, and the height of ground floor being more than the permissible height of the building are of these controversies. After analysis of structure under its self weight, cleared that the amount of compressive stress in five piers is more than the other underground piers. These piers can be seen by rounded cloudy dotted line shape in figure 3. After lateral seismic loading of model, maximum tensile and compressive stresses in the mentioned piers will be raised, but it has never farther up allowable tensile and compressive stresses. five loads combination, including one gravitational load combination and four extreme ones for common study had introduced which has been shown in Equation 2. The results of building analysis under first load combination, has been shown in Table 3. Maximum stress and strain in element nodes is less than allowable stresses. First principle stress remarks maximum tensile stress and third one remarks maximum compressive stress shown orderly by positive and negative sign.

In this study cleared that the load combination consist of vertical seismic component is not dominant and its results has not been told in this text. Light roof and not having a vertical dominant mode can be one of the such a claim. Moreover, in time response loading has been done in positive and negative direction of global axes of building which leads to same results.

Equation 2:

$$
\begin{aligned}
LC_1 &= DL + 0.2\,LL \\
LC_2 &= DL + 0.2\,LL + EQ_X \\
LC_3 &= DL + 0.2\,LL + EQ_Y \\
LC_4 &= DL + 0.2\,LL + EQ_X + 0.3\,EQ_Y \\
LC_5 &= DL + 0.2\,LL + EQ_Y + 0.3\,EQ_X
\end{aligned}
$$

Modal Analysis

After modal analysis cleared that the distance of mass and rigidity center of building for eastern–western seismic loading is more than northern– southern direction. since first mode shape has a torsional mode while the second one is shear. In figure 3 parts of the building which makes plan irregularity, has been shown by limited dotted line. Period of first mode obtained 0.1 second in Y direction.

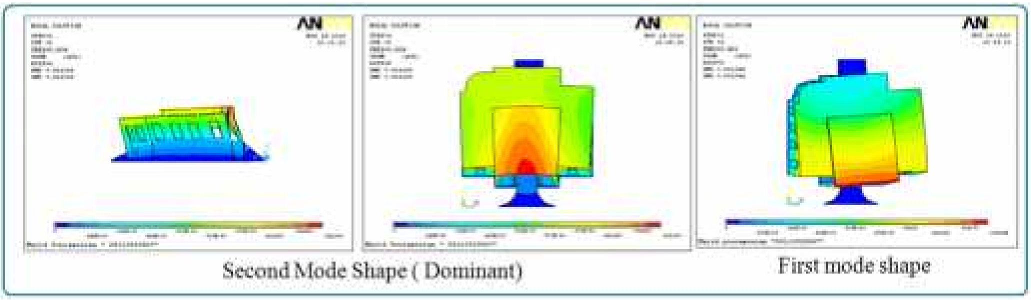


Figure 6 . Dominant mode shapes

Time Response Analysis

By choosing seven proper accelerograms including chi-chi, Kobe ,Tabas ,Manjil ,duzce, Erzincan and Elcentro step up to numerical model seismic analysis.

In this study, Manjil's earthquake record has chosen between mentioned records and is done on the model. Maximum amount of time response analysis considering linear material behavior and seismic hazard level I has been shown in Table 3. The amount of stress and strain is less than their Allowable ones and through different loads combination , LC_4 leads to more stresses and strains ; thus just related result to this load combination have been shown. In figure 7 the maximum principal tensile stress has been shown in the whole nodes of model during earthquake loading. As you see for all extreme loads combination with 475 years return period also for loads combination 2 and 3 with 975 years return period, maximum tensile stress in loading period, never reaches to allowable amounts of stresses and strains, so we can make sure of seismic performance of the structure linearly. maximum principal compressive stress during loading is always less than allowable compressive stress amount. In combination of fourth and fifth loads from hazard level loading II, tensile stresses have raised more than allowable tensile stresses. Furthermore, comparing loads combination results cleared that structure is more vulnerable in eastern – western excitation. Because of this, the percentage of relative wall in this direction is more than the others in each two floors, but it seems that plan irregularity is more effective than much amount of relative wall. The most critical seismic loading is only through made maximum tensile stress in structure related to LC_4 hazard level II . As it clears , Passing of tensile stress amount just for a while and just for some cycles passing cannot be remarked necessarily structure nonlinear performance. According to rapid loading and strain rates in dynamic loading, also being apparent tensile strength twice as much as its static strength, structural real behavior from linear analysis results can be concluded by an acceptable approximation in some cases .

To reach to a criteria to consider the correctness of linear analysis results has been referred to the performance curves of gravity dam produced by U.S. Army Corps of Engineers (figure 8)(EM 1110-2-6053,2007).Cumulative inelastic duration of stress shows a duration of time loading in which demand- capacity ratio would be more than that amounts. Regarding complexities of each structures, dam and historical unreinforced masonry buildings, also similarity of used materials, use of such a criteria and performance curves in historical structural analysis has many advantages. According to high effect of: history of dynamic loading and strain rate on structural performance and high sensitivity of tensile strength of masonry materials to rapid strain rate, it would be used this kind of performance curves for unreinforced masonry structures by required changes. In graph 1, the effect of strain rate has been shown on unreinforced masonry materials dynamical increase. Tensile dynamic increase factor (TDIF) has been introduced tensile dynamic in comparison with static; compression dynamic increase factor (CDIF) also has been introduced for compression similarly. Increasing strain rate, tensile relation will be increased rapidly. The more increase load frequency and seismic strain, the more dynamic tensile in loading ; and able to tolerate more tensile stress(Lodygowski et al., 2011).According to authors idea, considering orthotropic characteristics of masonry materials due to the present of mortar, and deterioration of masonry materials, cumulative inelastic duration of stresses should be reduced in order to use in unreinforced masonry structures. In figure 9 , you can see suggested curves for Ramsar museum.

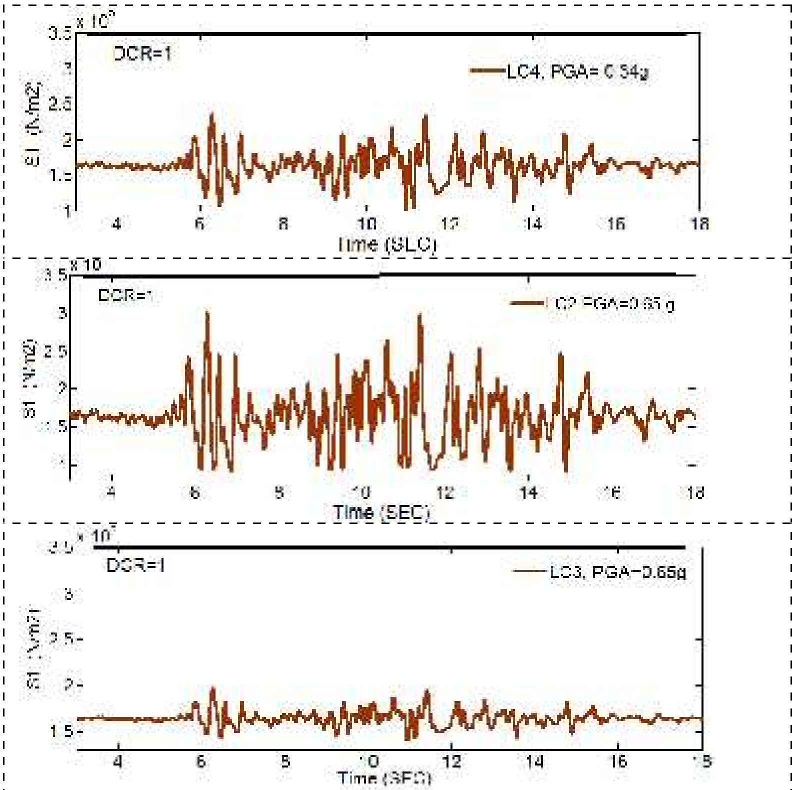
As you see, results analysis of loads combination obtained through previous section, is located that it is placed under performance curves or on its line. If related line to a load combination passes performance curve limitation, it means that to structural behavior assessment may require nonlinear time response analysis to estimate damage. Thus, through all loads combination, LC_4 and LC_5 (with hazard level II) should be assessed by nonlinear analysis. In current study, a parameter called A^{i}_{level} has been produced which remarks the area under curves of load combination i and top of the performance criteria's line. It's for LC_4 (hazard level II), is 0.06 and in LC_5(hazard level II), is 2.36. LC_5 and LC_4 are loads combination located top of performance curves and nonlinear analysis has been done for them. Structural Vulnerability is low and structure is stable. For LC_5(hazard level II) the structure has become unstable and collapsed after 7 second of earthquake loading.

Table 3. Maximum response of self weight analysis and linear time response(hazard level I)

Component\ load case	LC_1	LC_4
S1 (Tensile Stress, Mpa)	0.13	0.24
S3(compressive Stress, Mpa)	-0.59	-0.61
Von Mises Strain	0.0003	0.0003
Roof Displacement (mm)	0.16	0.8

Table 4. Maximum response of linear time response (hazard level II)

Component\ load case	LC_2	LC_3	LC_4	LC_5
S1 (Tensile Stress, Mpa)	0.3	0.2	0.58	0.48
, S3(Mpa)compressive Stress	-0.67	-0.65	-1.19	-0.73
Von Mises Strain	0.0003	0.0003	0.0007	0.0004
Roof Displacement (mm)	1.39	1.17	1.7	1.17



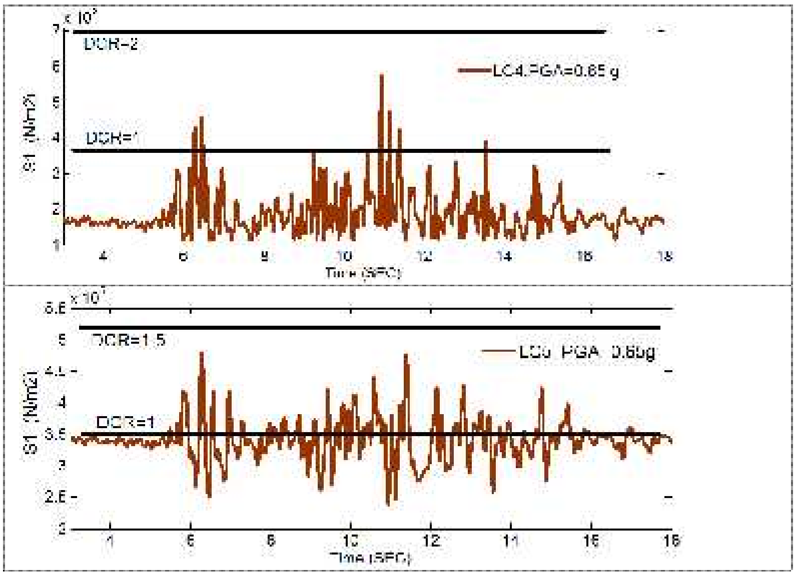
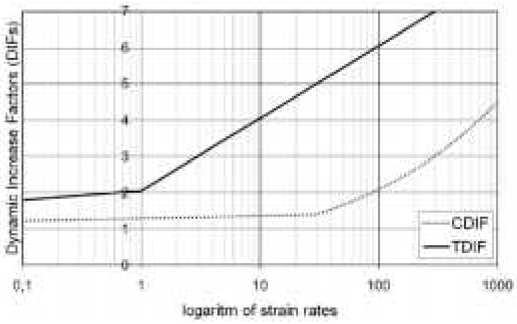


Figure 7 . History of maximum principal tensile stress in element nodes

Graph 1. Dynamic Increase Factor versus logarithm of strain rates for the tensile and compressive cases (Lodygowski et al.,2011).



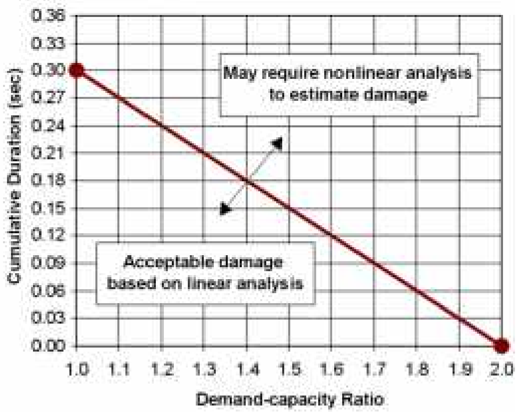


Figure 8.performance curves for Gravity dams

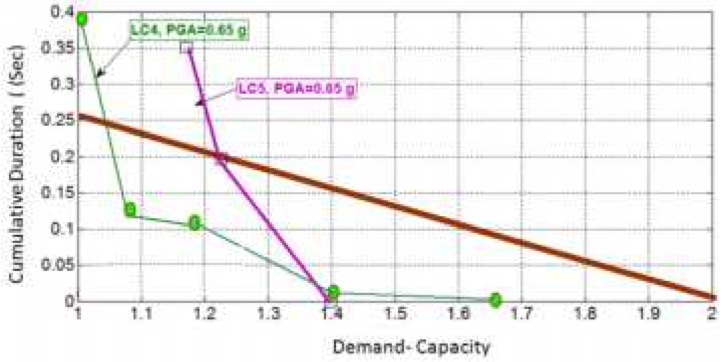


Figure 9. Suggested performance curves of historical masonry buildings

CONCLUSION

While the gravity and seismic loading with hazard level I capacity of the structure is high, its lateral resistance to seismic loading with hazard level II is not enough. Dynamic bearing capacity of the structure in y direction, is less than x direction. One reason for this may be the plan irregularity of this structure. Regarding to results analysis of nonlinear time response and great amount of parameter A^{LC5} (hazard level II) cleared that the structure in northern – southern direction has much vulnerable. Furthermore, considering little amount of A^{LC4} (hazard level II) and done nonlinear analysis results we can use materials behavior two or more linear curves by an acceptable approximation, instead of considering exact behavior of nonlinear materials. Introduced performance curves in Ramsar museum had used as a proper tool to evaluation of vulnerability and it reduced Run-time analysis to one ninth of required time for nonlinear time response analysis.

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