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Seismic vulnerability assessment of the Tughrul Tower

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Abstract. *The purpose of this study is to assess the vulnerability of an old masonry tower that located in the south of Tehran the capital of Iran against the near and far-field earthquakes. This construction is one of the important historical heritages in Iran which belongs to the Saljoghian(Seljuk) era about 800 years ago and in the old days this 20m height cylindrical tower was used as a tomb and indicating the way to the passengers by setting the fire. The tower is located on a soft soil in a high seismic zone in Iran and it could be at the risk of the earthquakes which may be generated by the active near and far faults. The only documented record shows that it has been retrofitted about 100 years ago.*

In general, the characteristics of the near and far field earthquakes are significantly different, and it is useful to understand the level of the vulnerability against the two kinds of the earthquakes and it helps to decide about the rehabilitation characteristics.

The first step for evaluating the tower consisted of site surveying. The authorities have not gave any permission yet for performing the required test such as the flat jack tests, shove tests, ambient vibration test and etc. which prepare sufficient information to develop analytical models, so in the absence of the exact mechanical properties of the materials, the authors decided to perform a series of the Interval Analysis which are able to consider the probable upper and lower limit of the material's properties. As the second step of the study the finite element models were developed and analysed. The Linear Incremental Dynamic Analysis (IDA) was conducted to determine the initiation level of the damages. The analyses revealed the insufficient strength of the brittle masonry construction. The differences of the upper limit and the lower limit material properties didn't influence the total behaviour of the tower. The structural response against the near-field and the far-field earthquakes showed some differences in the damage locations and the rate of its extent.

1 INTRODUCTION

1.1 Tughrul tower, is an old heritage

This tower is a 12th century monument in the vicinity of Ray, south of Tehran the capital of Iran and probably it is the tomb of Tughrul-I, one of the Seljuk governors. It is protected by Iran's Cultural Heritage Organization. This tower is in the form of a hollow cylinder with the height of 20m. In 1922 it had gone under retrofit and most of its architectural elements such as inscriptions and facade have been removed or destroyed. There is a stone slab on which the date of repair has been noted.

The gravity load bearing system of this structure is a brick curved wall. This tower has 20 meters height and the thickness of the crenate wall varies from 2.75 to 1.75. The outer and inner diameters are 16 and 11m respectively. The interior superficies of the wall is smooth but there are 24 jags along the perimeter of the exterior superficies, which have strengthened the wall and contribute in the stability of the structure against the lateral loads and tremors.

The aim of this article is to survey the amount of the vulnerability of this historical structure to render seismic rehabilitation methods or approaches.

Figure 1: Tughrul Tower

1.2 The behavior of Tughrul tower in the past earthquakes

Most parts of Iran located in the seismic zone and Tughrul Tower is in the hazardous area in Iran. The well-known reference[1] was studied to determine the historical earthquake events in the region, and all footprints of the seismic events were followed in the important historical documents such as the travelogues [2, 3] to find the seismic history of the Tower and the clues about the past damages and behavior of the Tower.

Figure 2, shows one of the paintings of the damaged Tower in 1840. Two destructed parts of the Tower are obvious in the picture. According to the Table 1 which has mentioned the historical earthquakes, it is discovered that the tower had been experienced several far-field earthquakes and the damages of the Tower could be attributed to those earthquakes. There was not any sever earthquake in the recent century and the evidences of the historical damages have been removed by sporadic retrofits.

Figure 2: Historical painting of the damaged tower (1840s) [2]

Table 1: Historical Earthquakes around Tughrul Tower [1]

Year	**Location**		**Magnitude**	**Distance(Km)**
	E°	**N°**		
1177	35.7	50.7	7.2	67
1485	36.7	50.2	7.2	165
1665	35.7	52.1	6.5	60
1687	36.3	52.6	6.5	130
1805	36.2	52.4	NA	109
1808	36.2	52.4	NA	109
1808	36.4	50.2	5.9	142
1809	36.3	52.5	6.5	123
1815	35.9	52.2	NA	76
1825	36.1	52.6	6.7	118
1830	35.7	52.5	7.1	96
1830	35.9	52.6	NA	110

NA: Not Available

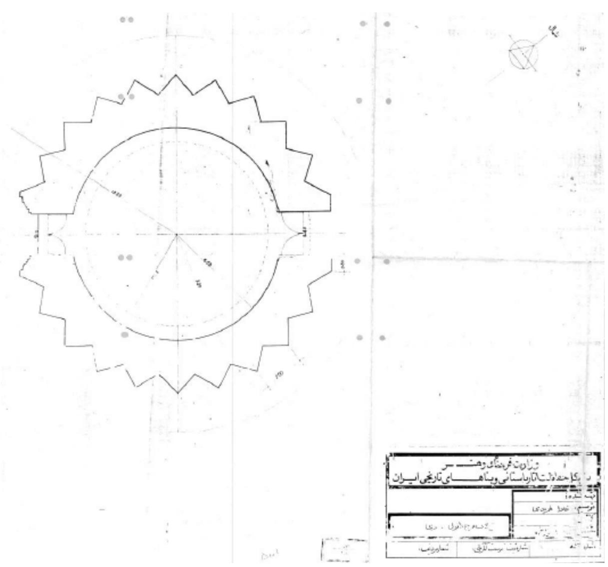


Figure 3: A sample of the existing documents of the Tower [3]

2 STRUCTURAL SURVEY

A sample of the existing documents for the geometry of the Tower has illustrated in Fig. (3), such an information were investigated and checked up by site visits. But there were no information about the material properties and the authorities codified no permission even for nondestructive material tests. While the material information are unknown, the authors were being faced with the significant challenge for the numerical studies on the structural model, so it was decided to compensate the lack of the information by using the Interval analysis which is explained later in the text.

3 SEISMIC HAZARD

Fortunately, the site soil properties, seismic geo-technique information and the report of the hazard analysis were in hand. Fig. 4, illustrates the site design spectrums. Two spectrums for 2 levels of probable hazards of 10% in 50 years (475 years) and 2% in 50 years (2475 years) events have shown in the figure. The maximum spectral acceleration of hazard (475 years and 2475 years) that came to hand was equivalent to 0.85g and 1.8g respectively. [3]

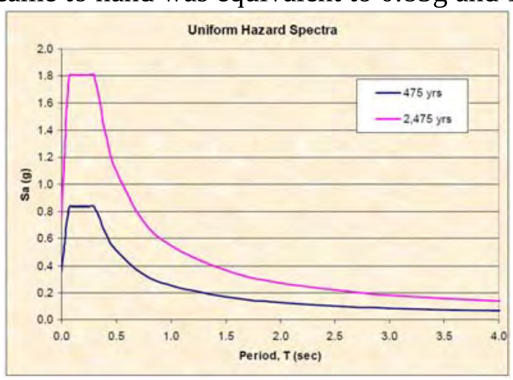


Figure 4: The design acceleration spectrum for two levels of hazards in the site [3]

4 MATHEMATICAL MODELLING

4.1 Material Property

As it mentioned before, in the absence of the exact mechanical properties of the materials, it was decided to perform a series of the Interval Analysis which are able the authors to consider the probable upper and lower limits of the material's properties. As shown in Table 2.

Table 2: Probable upper and lower limits of the mechanical properties of the materials [5]

Materials	Weight (KN/m^3)	Compressive strength (N/cm^2)	Tension strength (N/cm^2)	Elastic modulus (N/mm^2)	Shear strength (N/cm^2)	Shear modulus (N/mm^2)	Poisson ratio
$U_{Material}$	18	280	14	2400	9.2	400	0.2
$L_{Material}$	18	180	9	1800	6	300	0.2

4.2 Gravitational Analysis

Gravitational analysis were performed to verify the mathematical modelling. In Fig. 5, distribution of the gravitational stresses are rendered. The maximum gravitational stress in the level of the foundation of the Tower is approximately 5 to 7 kg/cm^2.

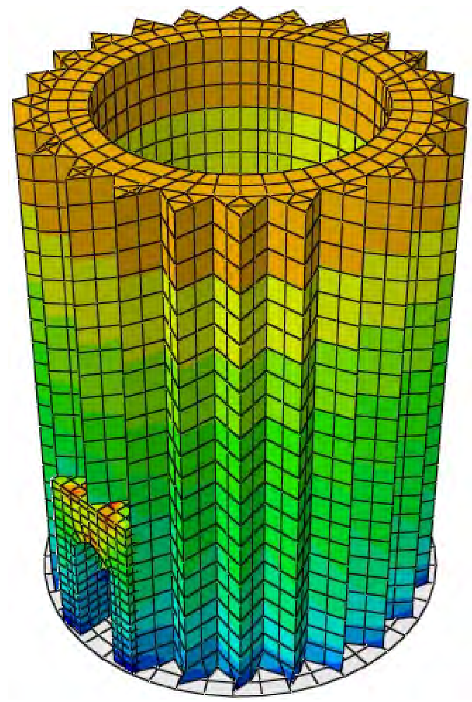


Figure 5: Gravity stress contours

5 SEISMIC VULNERABILITY ANALYSIS METHODOLOGY[6]

In order to analyze the vulnerability of historical structure under survey, tow methods of analysis was capable of being utilized. One method was the elastic linear method for predicting the crack initiation limit by controlling the fracture threshold criteria; and the other was the non-linear analysis by the modeling 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, whit the following assumptions, an acceptable engineering response can be attained:

- The ductility of the structures is negligible
- In the structure on the survey, nonlinear behavior leads to the extensive destruction in ornamental façade and finishing which are not acceptable
- Linear analysis is valid till a major cracks 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 [7] is an important reference for the controlling the vulnerability for the existing masonry buildings. This standard considers all failure modes of masonry walls but it is definitely recommended that the failure modes are considered as brittle once except for the rocking mode, as 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 criteria to acceptable behavior borderline and as a result, a base for the controls and designing. In the case the response in these limits is suitable, its behavior is not vulnerable and vice-versa, it is prone to vulnerability.

5.1 Earthquake Loading

Linear incremental dynamic analysis (IDA) was used to demonstrate the vastness of the high stressed zones and vulnerable parts of the Tower. Table 3, shows the six well known earthquake records for the analysis purposes of this study [8]. The records have categorized to the near and the far field earthquakes. In each increment of the analysis, all the records were being scaled to the same PGA and the damage criteria were being checked to determine the vulnerable parts of the structure.

Table 3: Near and Far Field Earthquakes [8]

EQ Record No	Record Name	Station	Dis. (Km)	Soil Type (USGS)	Magnitude	Duration(s)	Near/Far Field
R1	San Fernando	1015 Cholame-Shandon Array #8	223	C	6.6	23.5	Far
R2	Whittier Narrows 1987/10/01	90062 Mill Creek, Angeles Nat For	34.5	C	6	10.44	Far
R3	Chi-Chi	HSN	53.1	C	7.6	26.5	Far
R4	Loma Prieta	1601 Palo Alto - SLAC Lab	36.3	C	6.9	12.44	Near
R5	Loma Prieta	47125 Capitola	14.5	C	6.9	11.73	Near
R6	Loma Prieta	47381 Gilroy Array #3	14.4	C	6.9	11.35	Near

5.2 Linear and ultimate strength of the structure

ASCE41-06 [7] gives the approximate relations between the equivalent strength of a life safety level and the other conventional level of performance in a masonry wall (Table 4) comparing a linear elastic system with a nonlinear elasto-plastic system and assuming equal energy. The equivalent strength in the linear elastic system can be brought to hand as followed:

Immediate Occupancy strength:

$$F_{IO} = F_1 \tag{1}$$

Life Safety strength:

$$F_{LS} = 1.158\ F_1 \tag{2}$$

Collapse Prevention strength:

$$F_{CP} = 1.225\ F_1 \tag{3}$$

By the above mentioned formulas, the strength of the crack initiation in a linear model, could be utilized to estimate the other performance levels such as the collapse prevention level. Table 4 shows the deformation limits for various performance level of masonry structures.

Table 4: Deformation criteria for masonry structures

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

5.3 Damage control criteria

A suitable model for the masonry material should be taken into consideration to accurately specify the threshold limit of damage. Fig. 6, shows the model surmised in this article. This model is similar to Mohr-Coulomb model and has four limit lines, outside of the closed area is considered as the damaged zone and inside of the closed area is considered as the elastic behavior zone. To determine the damaged parts of the structure, the principal stresses have

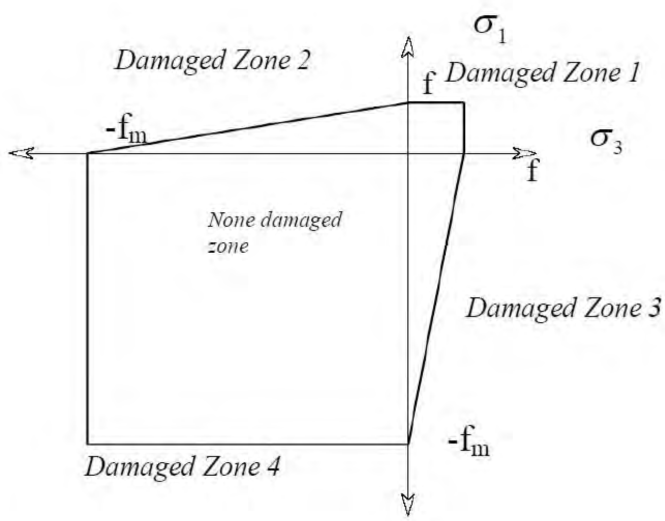


Figure 6: Idealized model for cracking in masonry

been extracted, then the location of the combined stresses of (σ_1, σ_2), (σ_1, σ_3) and (σ_2, σ_3) in the damaged or nine damaged areas have been as shown in Fig. 6. In this order, vulnerable points, in relative to cracks and erosion have been drawn graphically.

6 INTERVAL ANALSIS RESULTS

To assess the vulnerability of the Tughrul tower in the absence of information about the exact material properties, two extremes of the material properties named ($U_{Material}$) and ($L_{Material}$) from Table 2 have been chosen for numerical analysis. Analyzes have been repeated for the two material type models against the six earthquake of the Table 3. Sample results have been shown in Table 4.

For example, according to the Table 2, if the tension stress passes the limit of 14 and 9 N/cm^2 for $U_{Material}$ and $L_{Material}$ respectively, crack is expected in the material.

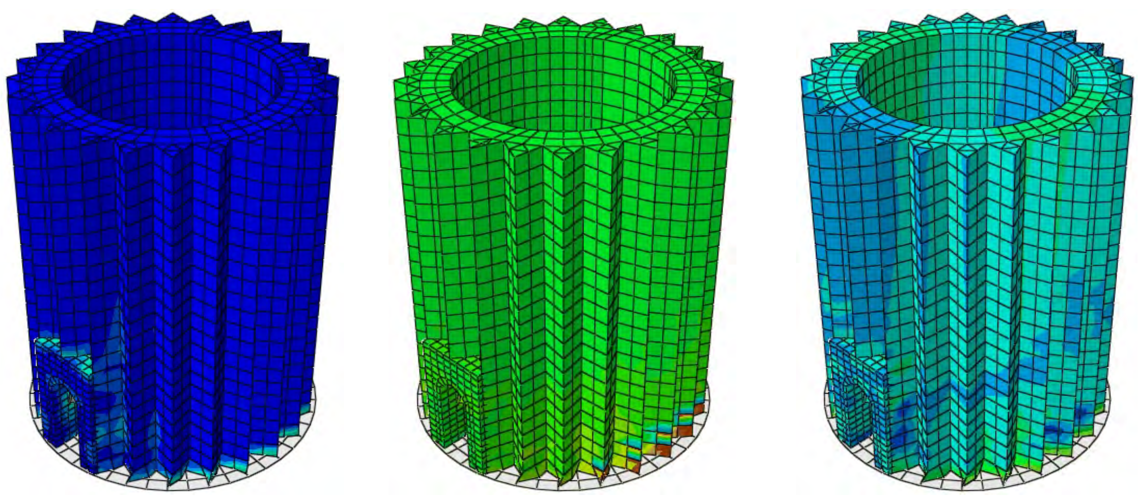
Finite Element Analysis have been shown that the major cracks on the structure would be appeared against the far field earthquakes at the PGA of 0.12g in $U_{Material}$ model and the PGA of 0.08g in $L_{Material}$ model. Brittle behavior of the masonry, rapidly leads the structure to the collapse stage and as an estimation it could be say that the Tower will collapse in the earth-quakes with the PGA greater than 0.15g.

In the near field earthquakes, no difference between the results of the $U_{Material}$ and $L_{Material}$ models have been observed and serious damages would be happened at the PGA of 0.02g which is meant that the near field earthquakes are thoroughly catastrophic for the Tower.

Table 4: Results of the interval analysis for two limits of the material properties against the earthquakes

EQ Record No	Material	PGA (g)	σ_1 (N/m^2)		σ_2 (N/m^2)		σ_3 (N/m^2)	
			compression	tension	compression	tension	compression	tension
R1	U	0.15	-2.18E+03	2.57E+05	-6.72E+05	6.83E+05	-4.64E+04	1.45E+05
	L	0.15	-2.53E+03	2.85E+05	-7.41E+05	7.53E+05	-5.11E+04	1.60E+05
R2	U	0.15	-1.68E+03	2.12E+05	-5.57E+05	5.66E+05	-7.23E+04	1.15E+05
	L	0.15	-1.60E+03	2.06E+05	-5.31E+05	4.92E+05	-3.60E+04	1.14E+05
R3	U	0.15	-2.57E+03	4.35E+05	-9.81E+05	1.03E+06	-1.01E+05	2.57E+05
	L	0.15	-2.97E+03	4.79E+05	-1.08E+06	1.13E+06	-1.12E+05	2.83E+05
R4	U	0.15	-5.44E+04	4.11E+06	-8.95E+06	9.12E+06	-1.06E+06	2.27E+06
	L	0.15	-4.15E+04	3.14E+06	-7.15E+06	7.44E+06	-7.56E+05	1.86E+06
R5	U	0.15	-2.00E+03	2.90E+05	-7.50E+05	7.64E+05	-5.32E+04	1.62E+05
	L	0.15	-2.55E+03	2.85E+05	-7.38E+05	7.40E+05	-8.81E+04	1.58E+05
R6	U	0.15	-1.05E+04	9.47E+05	-2.12E+06	2.22E+06	-2.18E+05	5.58E+05
	L	0.15	-9.46E+03	8.54E+05	-1.94E+06	2.02E+06	-2.04E+05	5.05E+05

Fig. 7, shows the general pattern of the stress distributions against a far field earthquakes for three principal stresses, σ1, σ2 and σ3.



(a) σ_1 (b) σ_2 (c) σ_3

Figure 7: Pattern of the stress distributions

7 CONCLUSION

Tughrul Tower is one of the Iranian heritages that located in a high seismic zone. To assess the seismic behavior of the Tower, the linear IDA analyzing method has been utilized to demonstrate the vastness of the high stressed zones and vulnerable parts of the Tower.

There were not any information about the mechanical properties of the structural materials, so it was decided to use the upper ($U_{Material}$) and lower ($L_{Material}$) limits of the mechanical properties of the conventional masonry material for the mathematical modeling and analyzing of the Tower. Another part of the study was focused on the roles of the near field and far field earthquakes on the extent of the damages in the tower.

A series of the finite element analysis were performed and the results showed that in the the far field earthquakes the serious cracks on the structure would be observed at the PGA of 0.12g in $U_{Material}$ model and the PGA of 0.08g in $L_{Material}$ model.

The extent of the damages lead the structure to the local or global collapse, and as an estimation it could be say that the Tower will collapse in the earthquakes with the PGA greater than 0.15g.

In the near field earthquakes, the first cracks on the structure would be observed at the PGA of 0.02g and no difference between the results of the $U_{Material}$ and $L_{Material}$ models were observed. It means that the material properties does not dominate the behavior of this structure against the near field earthquakes.

Finally, it is obvious that the seismic demands of the Tower are considerably greater than its seismic capacity. Especially the hazard of the near fault earthquakes are very serious and if the Rey fault (the nearest fault) severely shakes, the tower will suffer the extended damages or collapse, so rehabilitation and strengthening of the Tower against the earthquakes strongly recommended.

REFERENCES

[1] N.N.Ambraseys, C.P.Melville, *A History Of Persian Earthquakes,* Cambridge University, Britain, 1982

[2] Eugène Flandin, *Voyage en Perse, avec Flandin*, éd. Gide et Baudry, 1851

[3] Iranian Culture and Heritage Conservation, *Tughrul Tower documents Archive*, 1960

[4] http://iranhazard.mporg.ir/PageItem.aspx?p=Reports, *Probabilistic Seismic Hazard Analysis Of I.R.IRAN Phase 1-Greater Tehran Region,*2006

[5] OPCM 3431, Building Seismic Code of Ordinance PCM 3274 and Modifications, English Version Of Masonry Sections of Annex 2, dated 03/05/05 ,2005

[6] P.Homami, M.Golabchi, M.J.Jabbarzadeh, Seismic Vulnerability Assessment And Rehabilitation Design Of The Naghareh-Khaneh Edifice, *Proceeding of the 8th conference on Structural analysis of Historical Constructions (SAHC-2012), pp1573-1581, Wroclaw*, Poland ,2012

[7] American Society of Civil Engineers, *Seismic Rehabilitation of existing building,* ASCE Standard, SEI 41-06, 2006

[8] http://peer.berkeley.edu/smcat/search.html, *Peer Strong motion Data Base*, 2014