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Performance-Based Evaluation of Historical Structures

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***Abstract*—Performance-based evaluation of historical buildings has been quite popular for the last few decades. The structural system of historical buildings, generally, consist of masonry walls or piers. The behavior of such walls are controlled by either deformation or force. This paper discusses the basic principles to be considered in performance-based seismic evaluation of historical structures. Proposed seismic hazard levels, evaluation of existing seismic hazard, selection of earthquake ground motions as well as site geology, geological and tectonic settings of the area, seismic activity of the region and local soil conditions are needed for a thorough evaluation. This study also presents a seismic performance evaluation of clock tower in eastern Turkey based on the proposed principles.**

***Keywords*—historical structures; masonry walls; performance-based design**

I. INTRODUCTION

Performance-based seismic evaluation of historical buildings with masonry walls has been the area of research all over the world for the last few decades [1, 2]. Evaluation of seismic performance for existing masonry buildings has been mentioned in many standards [3, 4, 5, 6]. These standards define four failure modes for unreinforced masonry (URM) walls in seismic evaluation; a) bed-joint sliding, b) rocking, c) diagonal tension, and d) toe crushing. Lateral deflections of walls and piers can get higher in bed-joint sliding and rocking as strengths remain close to constant. That is why these two failure modes are controlled by deformation. Diagonal tension and toe crushing, on the other hand, are controlled by force. These failure modes occur when a certain stress is reached and can cause sudden and substantial strength deterioration (Fig. 1). Stair-stepped diagonal cracking is also classified as a deformation-controlled failure mode. The behavior of the masonry wall or pier is governed by the failure mode with the lowest capacity. A certain ductile behavior is expected in a deformation-controlled failure mode, whereas the failure is due to brittle behavior in force-controlled failure mode. Expected lateral strength of walls and piers for deformation-controlled behavior and lower bound lateral strength for walls and piers for force-controlled behavior are provided in FEMA 356 [4], ASCE 41 [5] and Eurocode 8 [6] provide.

This paper briefly investigates the basic principles to be considered in performance-based seismic evaluation historical structures with masonry walls and piers. The seismic performance evaluation of a historical structure is also presented.

II. SEISMIC PERFORMANCE EVALUATION

Seismic evaluation of masonry buildings are usually carried out by linear or nonlinear procedures. For the analyses, the members should be classified as either having force-controlled or deformation controlled failure modes. If an element has a force-controlled failure, it should have enough capacity against the applied loads without yielding and plastic deformation, e.g. force demand should be less than or equal to the member capacity. If the element is designated as having a deformation-controlled failure mode, then the member is expected to experience some amount of ductility without significant loss of strength. Based on the performance level, the observed plastic deformation should be limited. The limitations for nonlinear static procedures are given in FEMA 356 [4] and ASCE 41 [5] (Table 1) (Fig. 2). Simplified force-deflection relations for URM in-plane walls and piers are given in Fig. 3.

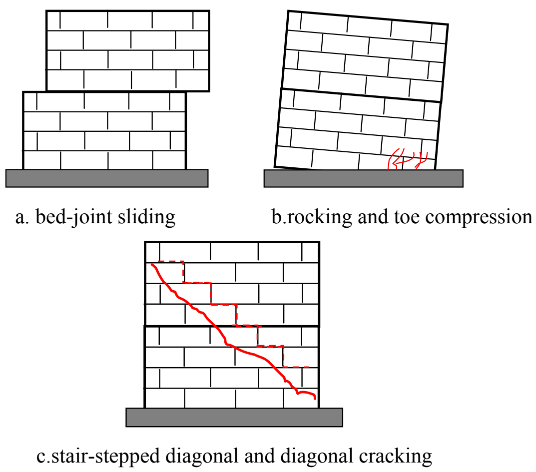


Fig. 1. Failure modes in unreinforced masonry walls

TABLE I. ACCEPTANCE CRITERIA FOR UNREINFORCED MASONRY IN-PLANE WALLS AND PIERS

	Limiting Behavioral Mode	Primary Members			Secondary Members	
		Immediate Occupancy (IO) (%)	**Life Safety (LS) (%)**	**Collapse Prevention (CP) (%)**	**Life Safety (LS) (%)**	**Collapse Prevention (CP) (%)**
FEMA 356 (2000) and FEMA 274	Bed-Joint Sliding	0.1	0.3	0.4	0.6	0.8
	Rocking	0.1	$0.3h_{eff}/L$	$0.4h_{eff}/L$	$0.6h_{eff}/L$	$0.8h_{eff}/L$
ASCE 41 (2006)	Rocking	0.1	$0.3h_{eff}/L$	$0.4h_{eff}/L$	$0.6h_{eff}/L$	$0.8h_{eff}/L$

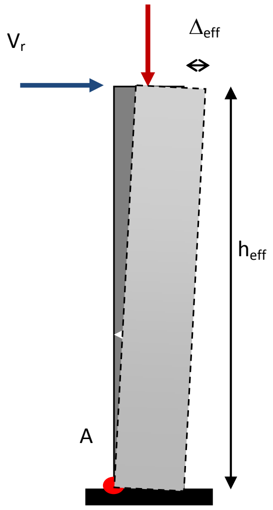
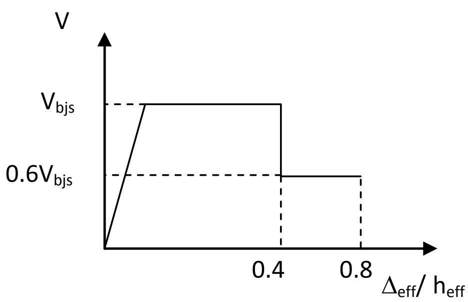
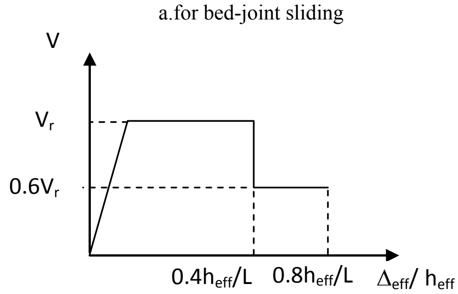


Fig. 2. Rocking forces



a.for bed-joint sliding



b.for rocking

Fig. 3. Simplified force-deflection relations for URM in-plane walls and piers [4, 5]

III. A METHODOLOGY FOR THE SEISMIC PERFORMANCE EVALUATION OF HISTORICAL STRUCTURES

A. Determination of the seismic hazard levels

Seismic hazard levels might be defined on either probabilistic or deterministic. Probabilistic hazards are generally defined in terms of probability for a specified period of time, whereas deterministic demands are defined within a level of confidence in terms of a specific magnitude event on particular faults. A three-level seismic hazard; namely, EQ1, EQ2, and EQ3 can be used for this purpose. EQ1 seismic hazard level corresponds to an earthquake ground motion with 50% probability of exceedance in 50 years, whereas EQ2 and EQ3 seismic hazard levels correspond to earthquake ground motions with 10% and 2% probability of exceedance in 50 years, respectively. The structures are expected to satisfy the immediate occupancy performance level under EQ1 seismic hazard level. This performance level corresponds to the structural damage where the risk of life-threatening injury as a result of structural damage is very-low and some minor structural repairs may be appropriate. For EQ2 and EQ3 seismic hazard levels, the structures are required to satisfy the life safety and collapse prevention performance levels. Life safety performance level defines structural damage in which significant damage to structure has occurred, but some margin against either partial or total structural collapse remains. Collapse prevention level defines the structural damage in which substantial damage to the structure has occurred and the repair of the structure may not be technically practical and the occupancy of the structure may not be safe.

B. Evaluation of the existing seismic hazard

Seismic Hazard is defined as the potential for earthquake-induced natural phenomena such as ground motion, fault rupture, soil liquefaction, landslides and tsunami with adverse consequences to life and built environment at a specific site [8]. Seismic risk, on the other hand is defined as the expected losses due to the consequences of the earthquake induced phenomenon.

Both Probabilistic Seismic Hazard Assessment (PSHA) and/or Deterministic Seismic Hazard Assessment (DSHA) may be employed for the performance-based evaluation of historical structures. The core of PSHA lies in the integration of individual influences of potential earthquake sources (considering both size and distance) into the probability distribution of maximum annual ground motion parameter from which the return period follows. Ground motion descriptors such as peak ground acceleration (PGA) are calculated from ground motion prediction equations (or attenuation relationships) and the mean rate of exceedance of specified ground motion amplitude, which is the hazard, is determined [9]. The results of a PSHA can be represented in several ways. A common approach involves the development of seismic hazard curves, which indicate the annual probability of exceeding different values of a selected ground motion parameter. The seismic hazard curves can then be used to compute the probability of exceeding the selected parameter in a specified period of time.

C. *Selection of earthquake ground motions*

At least, 7-pairs of horizontal ground motion time history components (two horizontal components for each ground motion record orthogonal to each other) should be selected and scaled either in frequency or time domain complying with the following criteria [7]:

a. Real EQGM records are suggested for selecting each ground motion compatible with the scenario earthquake parameters.
b. The average 0-sec spectral acceleration determined from the scaled EQGMs do not fall below the spectral acceleration at 0-sec period ($0.4S_{MS}$) of the design spectrum.
c. The ground motion duration in accelerograms between the points where the acceleration exceeds ±5%g for the first and last times equals, at least, to 15 sec or five times the fundamental period of vibration of the structure (T).
d. A 5% damped combined response spectrum is obtained for each scaled ground motion considering the horizontal orthogonal components of that ground motion by taking the square root of sum of squares (SRSS) of each component.
a. The ground motions are scaled such that the average value of the SRSS spectra does not fall below 1.3 times the 5% damped spectrum of the design earthquake between 0.2T and 1.2T.
b. The fundamental period of the structure should be estimated. The response spectral acceleration of the combined spectrum for EQ1, EQ2, EQ3 seismic hazard levels should be scaled accordingly based on this estimation.

D. *A Simple Procedure for Seismic Evaluation*

The main steps of a simple approach for seismic evaluation of historical structures may be followed as below:

a. Analyze the structure under at least two-level seismic hazard and the corresponding response spectra and ground motions,
b. Determine and check the lateral displacements,
c. Check if there are any tension forces develop in the masonry piers,
d. If there is no tension in the pier, no rocking response will develop.

IV. CASE STUDY: ERZURUM CLOCK TOWER

A. *Historical Information*

Erzurum is one of the most important historical centers in eastern Anatolia, Turkey. It is considered as a valuable ancient city not only in Turkey, but also in the world due to its natural, historical, and archeological assets. The city has been located on the historical Silk Road and conquered by many ancient civilizations throughout its history. According to the historical records, it has been an important center for Roman Empire, Byzantine, Persia, Armenia, and Turkish civilizations.

Erzurum has a historical and cultural background since almost 7,000 years. Over the centuries, the ancient city has imposed many different cultural effects and architectural styles. One of the valuable historical structures in the city is the Erzurum Clock Tower. This fascinating tower is also known as the Tepsi Minare (Tray Minaret) due to the former structural shape. The tower is situated on the southwest side of the Erzurum Castle and has a slender structural form. It was built originally as a minaret by Muzaffer Gazi Bin Ebu'l Kasım in the 12th Century [10]. This minaret had a balcony, which was placed on top of the minaret; however, this balcony disappeared for unknown reasons. After the disappearance of the balcony, the minaret served as a watchtower. Moreover, a clock mechanism, cupola (small dome) and bell of the clock were installed in the 19th century and the watchtower turned into a clock tower [10]. As of today, the clock mechanism of the body and bell of the clock included in the cupola do not work properly and the tower is visited as a historical heritage site (Fig. 4).

B. *Geometrical Description*

The clock tower has three major parts; tower base, tower body, and small dome (Fig. 5). The tower base consists of cut-stones and has plan dimensions of 6.8 m x 6.8 m. It is the lowest section of the structure with a height of 9.0 m. It is also the most rigid part of the structure. The second part is the tower body. The cylindrical tower body is made of stones and bricks, and has a diameter of 6.6 m. The cylindrical body is 15.0 m in height and the highest part of the tower. The ancient clock is located on the tower body. The last part is the small dome (cupola). It consists of a timber shell and is anchored to the tower body with the timber frames. The height of the dome is 5.5 m and covered with lead roofing. The bell of the clock is placed in the cupola. The stairs within the tower are configured in a spiral way rising along the tower height in a counter clockwise direction. The staircases consist of stone steps and surround the internal section of the cylindrical structure.

Fig. 4. Erzurum clock tower

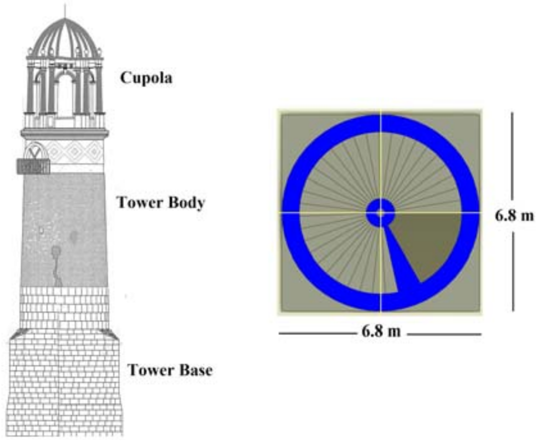


Fig. 5. Sketch of the clock tower

C. On-site Investigation and Structural Damage

Many historical structures in Erzurum have had some damages and the risk of being razed due to their structural defects over the time. The clock tower, especially, has been substantially deformed and lost its structural integrity and stability. The main problems on the structure are the damage on the structural elements, the loss of material, and the decrease in the structural strength. The mortar on one of the facades of the piers has been partly demolished and many irregular micro cracks have been observed on the tower.

The tower was constructed with rectangular cut-stone blocks, handmade bricks and mortars. During the life of the tower, construction materials have been deteriorated and have lost their qualities due to many reasons such as environmental conditions and earth disasters. These have caused irreversible negative effects on the tower. Observed structural failures have been generally occurred due to the material degradations which have been localized above the tower base and on the tower body facade (Fig. 6). Some damages have occurred especially on the tower body as cracks and the separation of the bricks. Various separations have also been appeared between the stones and bricks. In addition, the tower has been damaged several times due to the natural disasters or human interventions over the past few decades. Therefore, it has not been technically repaired, but has been renovated several times by using different materials such as stones and mortar. However, these unconscious repairs have caused more damages to the structure (Fig. 7). The binding material between the masonry units was partially eroded. These abrasions may have caused the structural resistance to get weaker. In some cases, abrasion and degradations are also observed in the stone units of the tower base.

The deteriorations of the structural elements and the decay of the structural materials have caused damage on the structural elements at the internal sections. Spalling and flaking have been also visible to the eye in the internal walls. Moreover, structural cracks and fractures have been observed at the internal facades and they are repaired non-technically. All observed damages are considered to be very dangerous for the tower since they may cause destructive crashes and fractures. Therefore, these damages should be considered seriously and some precautions should be taken to avoid or to abate their effects.

D. Seismic Hazard

Design spectra for EQ1, EQ2, and EQ3 seismic hazard levels are determined using the 5% damped spectral response accelerations at short period (S_S) and at a period of one-second (S_1) are determined for the reference soil type B. The 5% damped spectral response accelerations for short periods (S_S) and at one-second (S_1) for B Class Soil are determined using the site coefficients F_a and F_v. S_S and S_1 are assumed to be 0.30, 0.59, 0.95 and 0.11, 0.23, 0.37 for EQ1, EQ2, and EQ3 seismic hazard levels, respectively. F_a and F_v are taken to be 1.0 for EQ1, EQ2, and EQ3 seismic hazard levels, respectively. Fig. 8 shows the design response spectrum corresponding to EQ3 seismic hazard level.

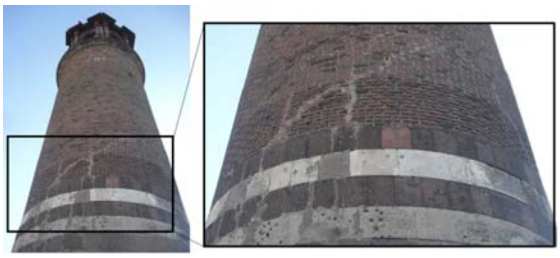


Fig. 6. Material degradation and macro cracks

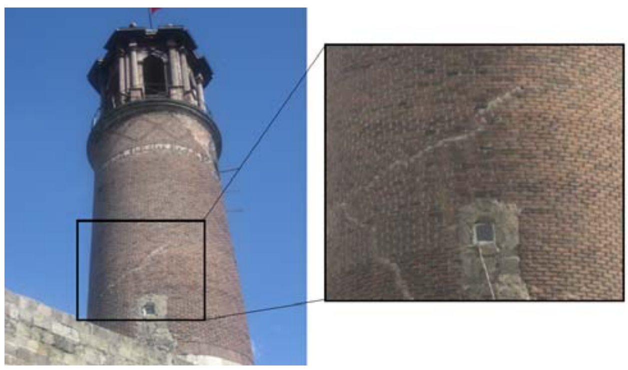


Fig. 7. Unconscious repairs on the tower body

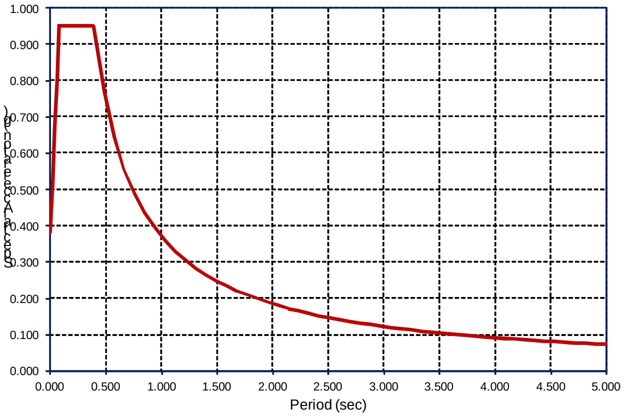


Fig. 8. Response spectrum

V. ANALYSES RESULTS

Three-dimensional (3-D) finite element (FE) models have been developed based on the structural state and the geometrical constraints of the tower. FE analyses program, ANSYS Workbench, has been used to analyze the tower with SOLID186 elements, which has 20 nodes and three degrees of freedom per node. In the numerical model, the tower was discretized with 19890 solids, corresponding to 101892 nodes (Fig. 9).

Modal analyses indicate that the mode shapes result in pairs of orthogonal modes in X-Y plane, i.e. first and second modes are orthogonal with T=0.31 sec, third and fourh modes are orthogonal with T=0.093 sec. The torsional mode generally occurs in higher frequencies in such tall slender structures. For the tower, the torsional mode occurs in sixth mode with T=0.062 sec. For the first six modes, the mode shapes are presented in Fig. 10.

Response spectrum analysis for the tower has been carried out to investigate the behavior of the structure. The maximum normal stress was found to be 10.98 MPa around the cupola of the tower (Fig. 11a), whereas the normal stress was about 1.04 MPa around the transection zone between the tower base and tower body (Fig. 11b). The maximum lateral displacement was observed at the top of the tower about 46.855 mm corresponding to %0.16 drift ratio (Fig. 12). Table 1 indicates that at this drift ratio level, the tower is expected to have a structural responsee between immediate occupancy and life safety.

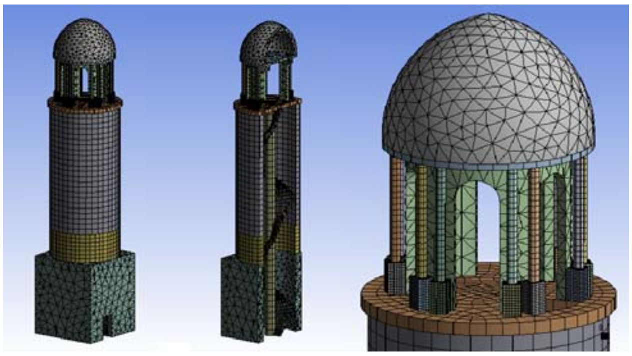


Fig. 9. Numerical model of the tower

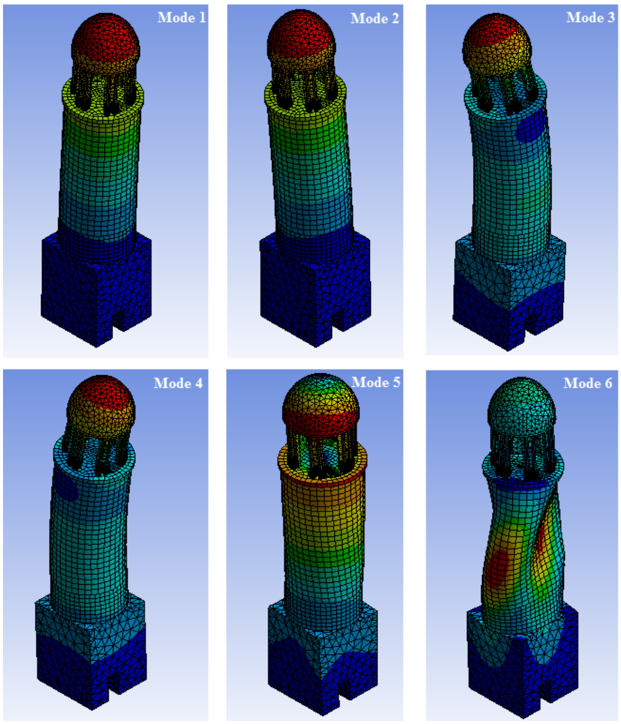


Fig. 10. Mode shapes of the first six modes

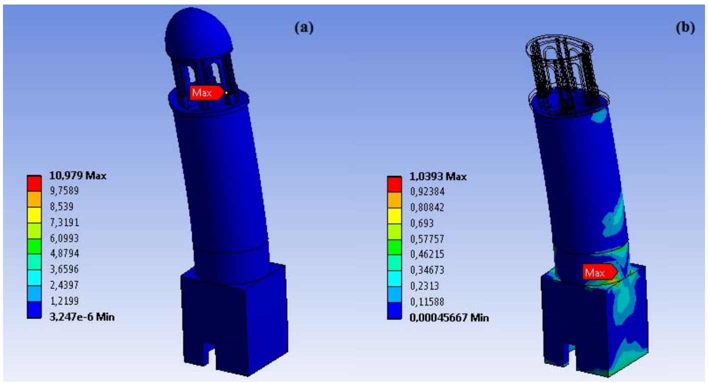


Fig. 11. Normal stresses (MPa) under EQ3 (a) Cupola, (b) Tower Body

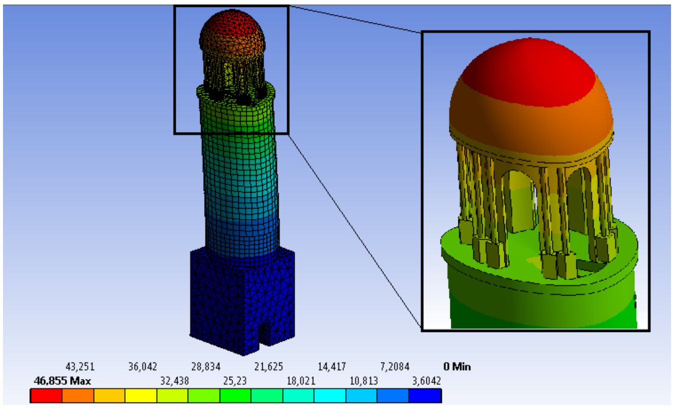


Fig. 12. Deformed shape and maximum displacement (mm) under EQ3

VI. CONCLUSIONS

The vulnerability of the masonry structures to earthquakes and seismic effects has been among the most common reasons of the collapse of masonry structures. Therefore, it is crucial to determine performance-based seismic performance of masonry structures located in active seismic zones.

This paper briefly investigated the basic principles to be considered in performance-based seismic evaluation historical structures with masonry walls and piers. The seismic performance evaluation of a historical clock tower is also presented. Seismic hazard levels, evaluation of existing seismic hazard, selection of earthquake ground motions as well as site geology, geological and tectonic settings of the area, seismic activity of the region and local soil conditions are needed for a thorough evaluation.

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