Identifying masonry buildings that are under high seismic risk

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ABSTRACT: Masonry has been one of the most popular construction materials for centuries as it provided economical solutions for sheltering problems worldwide. Turkey, as a country with significant portion of its land in seismic zones, has a masonry building stock in the order of millions, most of which built in the twentieth century. In addition to the complications associated with anisotropic and composite nature of masonry buildings, the accurate estimation of seismic demands with simple yet accurate analytical tools is an extremely challenging task. A comprehensive research project was initiated at Middle East Technical University comprising of in-situ material strength determination from ten existing masonry buildings, two building tests including forced vibration and lateral load testing, and numerical simulations for calibration and modelling of existing masonry buildings. Making use of these experimental and simulation results, available seismic assessment techniques based on linear elastic analysis were refined. More specifically, lower bound material strength values, effective pier stiffness models and performance limit states for masonry walls were proposed. The overviews of the proposed revisions, which are currently under consideration by the committee, are presented in this paper.

1 INTRODUCTION

The behaviour of masonry is very complicated to be predicted due to its non-engineered construction, non-homogeneous and anisotropic material characteristics, its dependency on workmanship, etc. However, it has been widely used all over the world, especially in developing countries, for centuries. Therefore, its performance under the effect of strong ground motions has largely been tested by Mother Nature. Most of the time, this structural type distressingly fails to do its sheltering duty and caused death of its residents. This is mainly due to the inappropriate detailing rooted in the non-engineered construction of these structures, i.e. most of the masonry structures are built by utilizing traditional practices, which makes difficult to utilize simple analysis techniques to simulate this highly nonstandard type of construction. According to Abrams (2001), despite being the oldest construction material, masonry is still the least understood in terms of strength and deformation characteristics.

Apart from the above discussions, there are a large amount of masonry structures lying in earthquake-prone regions around the world that require assessment in terms of seismic safety considerations in order to mitigate financial and physical losses. Besides, most of the historical masonry structures are also composed of masonry and should be evaluated to prevent the loss of cultural heritage during future earthquakes. In literature, researchers aimed at proposing different techniques to assess and rank the seismic performance of existing masonry buildings (D’Ayala et al. 1997, D’Ayala 2005, Kappos et al. 2006, Park et al. 2009, Erberik 2010). In addition, various guidelines and standards are published to propose techniques for assessing existing masonry structures or for designing new ones (FEMA 356, Eurocode 6, TEC 2007).

Turkey has also suffered from the vulnerability of its building stock as it lies in one of the most active seismic zones. Consequently, seismic mitigation has gained importance as the estimated number of buildings requiring seismic risk assessment is in the order of 5 million. For this purpose, a revolutionary law, in comparison to the incompetent predecessor approaches, was passed in 2012, named herein as the Urban Renewal Law (URL). With this law, the need to identify the buildings under high seismic risk became extremely important in Turkey. The new law handed the power to abandon the sale or rent of a building with high seismic risk to government authorities, which brought the need to have sound techniques for risk assessment.

The seismic assessment procedure currently employed for masonry buildings during the execution of the law follows the rules provided by the Turkish Earthquake Code (TEC 2007) and Guidelines for the Assessment of Buildings under High Risk (GABHR 2012), where the latter document includes only minor changes compared to the former. In these documents, the seismic assessment method for reinforced concrete (RC) buildings are based on detailed studies calibrated according to component test results and building analyses (Binici et al. 2015). On the other hand, the seismic assessment method for masonry buildings is based on a procedure similar to that used in design where a response modification factor is assumed and computed wall shear stresses under the effect of vertical and lateral loads are compared with the strength limits. The aforementioned seismic assessment method for masonry buildings has two major drawbacks: i- Masonry material strength default values for different type of units (usually these default values are used in seismic assessment) suggested by TEC 2007 for seismic assessment calculations are not known with sufficient accuracy to be representative of the actual masonry strength in existing buildings, ii- The assessment method employed is based on assuming a response modification factor, i.e. R=2, similar to the factor in new design, hence lacks basis for an existing structure. These two important deficiencies of the existing techniques are sometimes found to render incorrect risk classification. A comprehensive research project was initiated at Middle East Technical University comprising of in-situ material strength determination from ten existing masonry buildings, two building tests including forced vibration and lateral load testing, and numerical simulations for calibration and modeling of existing masonry buildings. Making use of these experimental and simulation results, available seismic assessment techniques based on linear elastic analysis were refined. More specifically, lower bound material strength values, effective pier stiffness models and performance limit states for masonry walls were proposed. The overviews of the proposed revisions, which are currently under consideration by the committee, are presented in this paper.

2 PROPOSED LOWER BOUND MASONRY STRENGTH VALUES

In the first part of this study, ten pre-evacuated unreinforced masonry buildings were selected for material strength characterization. Nine of the buildings were located in Ankara (capital of Turkey) and the other one was located in the city of Kırşehir (a city located to the south of Ankara). No prior damage was observed in the walls of selected buildings. The general information regarding the selected buildings is summarized in Table 1. Before 1970's, solid clay bricks were the primary construction material utilized in the construction industry. After that time, factory-produced hollow clay bricks had started to be widely used in masonry construction practice. In addition, adobe and, to some extent, concrete blocks were commonly used in especially rural areas. For this reason, the buildings in this study were selected to represent the masonry construction characteristics in Turkey.

Also, the number of stories of each selected building ranged between 2 and 3 in parallel with the limitation on the number of stories given in TEC 2007. In addition, there was no meaningful correlation of total wall area with the seismic zone and/or the number of stories and, also, there was a large variability in plaster thickness, which indicated non-engineered and non-standardized construction practice. Nearly all of the selected buildings have regular plan geometries. For each selected building, 70 cm x 70 cm square wallettes were extracted from the ground story. The wallettes were extracted by utilizing hydraulic saws to prevent any damage and to ensure the integrity of wallettes (ASTM C1532). In some buildings, fewer specimens were extracted from some of the selected buildings due to the insufficient wall areas (Table 1).

The material tests were performed by utilizing a displacement-controlled testing machine. In this study, compression, diagonal tension and sliding shear tests under zero pressure were completed by implementing ASTM standards (ASTM C1314, E519/E519M, C1532 and C1552) and European norms (EN 1052). The test results presented in Table 2 could be compared with the recommended capacities of TEC 2007. The obtained strength values and the TEC 2007 recommended strength values are compared in Table 3 for each material type. In Table 3, it is clear that TEC 2007 compressive strength values are almost identical to the test results. Therefore, the code proposed material strengths lack of the factor of safety as they are nearly equal to the experimentally determined ones. Also, it is apparent that the compressive strength differs depending on the unit type of the masonry. This conclusion is also consistent with the TEC 2007 recommendations. However, no correlation was observed if shear and diagonal tension capacities were investigated (Fig. 1). This is because, the mortar quality played more important role for shear and diagonal tension strengths as the failure of nearly all of the specimens was caused by cracks following the mortar layers. The proposed lower bound strength values are presented in Table 4.

Table 1. General information about the selected buildings

Building ID	1	2	3	4	5	6	7	8	9	10
Location	Ankara	Ankara	Ankara	Ankara	Ankara	Kırşehir	Ankara	Ankara	Ankara	Ankara
Construction year	NN*	1990	1950	1960	1970	1977	NN	NN	NN	NN
Type of masonry unit	Hollow clay brick	Hollow clay brick	Solid clay brick	Solid clay brick	Cellular concrete block and adobe	Solid concrete block	Solid clay brick	Hollow clay brick	Hollow clay brick	Solid clay brick
Earthquake zone	3	4	4	4	4	1	3	3	4	4
Number of stories	2	2	3	3	1	2	2	2	3	3
Building dimensions (m x m)	8.9 x 14.0	7.3 x 12.5	10.5 x 18.0	14.5 x 16.2	9.0 x 9.1	9.0 x 10.7	10.1 x 12.1	10.5 x 16.9	9.2 x 12.3	11.9 x 21.7
Wall ratio x-dir. (%)**	29.4	34.5	25.5	22.1	24.3	22.1	22.9	25.4	31.1	26.3
Wall ratio y-dir. (%)**	24.5	21.0	23.0	32.5	30.7	31.0	23.9	27.3	19.0	17.0
Average plaster thickness* (cm)**	3.5	4.6	6.0	4.6	3.5	7.4	4.8	3.1	4.2	5.4
Number of Wallettes	6	8	8	8	6	6	6	6	6	8

*: Not known, **: The ratio of the length of walls in one direction to the floor plan area, ***: Total plaster thicknesses on both sides of each specimen is measured and the average thickness for the corresponding building is reported.

3 PROPOSED STIFFNESS MODEL

While determining the stiffness of masonry walls, the computation of effective height is one of the most complicated tasks. Thus, in this study, the most suitable approach to calculate the effective pier height of a masonry wall is also investigated. To this end, the selected buildings were modeled on computer environment by using finite element method. In each model, eight-node shell elements were utilized and all models were three dimensional. Also, frame models were formed by using TEC 2007, Dolce (1989), Moon (2004) and Full Height approximations for the calculations of effective heights. Then, the initial stiffnesses of each building were compared and the best approximation for determining the effective height was examined (Table 5).

From Table 5, it can be inferred that the full height approach is generally better to simulate the lateral stiffness of the selected buildings as both it has minimum average and standard deviation. This conclusion was also confirmed by the in-situ test results. As explained in the introduction part, two site experiments were also conducted in the scope of this study. It is clear from Fig. 2 that the secant slope of the first experiment lies between the Dolce (1989) [percentage error of 11.9%] and Full Height model [percentage error of -23.3%]. On the other hand, the secant slope of the second experiment falls between Moon (2004) [percentage error of 6.2%] and Full Height model [percentage error of -5.1%]. The percentage errors of other effective height models are also summarized in Table 6, which supports the conclusion that Full Height model is better than the other techniques. Hence use of full height as the effective height for walls between openings is proposed for the revised provisions.

4 PROPOSED ASSESSMENT METHOD

The assessment method employed in the current version of the aforementioned codes for masonry is based on assuming a response modification factor, i.e. R=2, similar to the factor in new design, and checking the shear stresses of walls based on bed joint sliding capacity. This method lacks basis for the assessment of existing structures, where the ductility level is not known as a priori. The assessment for RC buildings is conducted based on R=1 analysis followed by a member by member evaluation, which is more sound compared to the aforementioned approach. The proposed revision for the risk assessment of masonry building is conducted using the following steps:

- Calculate the elastic base shear demand according to TEC 2007 provision. In calculations, S(T) is assumed as 2.5 regardless of the first fundamental period of the structure in concern.

The assessment should be conducted for both earthquake directions separately and the building is classified as "seismically under high risk (HR)" if the assessment result for any direction fails to comply the limits.

- Full Height assumption is utilized to determine the stiffnesses of the walls and the end condition of walls are assumed to be fixed. While calculating the stiffnesses, both flexural and shear contributions are taken into account. Depending on the relative shear stiffness of each wall segment, the shear force acting on each wall is computed taking additional demands from possible torsional effects into account. The shear stresses for each wall are computed.
- Walls are classified as "slender" or "non-slender" according to its slenderness ratio. If the slenderness ratio is greater than 2, the wall is classified as "slender". In other cases, it is non-slender. For slender walls, there exist three different failure modes (sliding, diagonal tension and rocking) whereas only two failure modes, i.e. sliding and diagonal tension, are taken into account for non-slender walls. The minimum of the capacities for different possible failure modes is taken as the capacity of the wall (Table 7).
- Determine demand – capacity ratio (m) by dividing elastic shear demand on the selected wall by the wall capacity. Compare the calculated demand – capacity ratios with the limit values in Table 7. If m exceeds the m_{lim}, the wall is classified as vulnerable.
- Total shear force acting on the vulnerable walls divided by the total story shear force is compared with the limit of 35%. If the limit is exceeded, the building is classified as being under high seismic risk (HR).

Table 2. Summary of material test results

Building ID		**1**	**2**	**3**	**4**	**5**		**6**	**7**	**8**	**9**	**10**
Modulus of Elasticity*, **	1	2282	3236	4010	2725	838	582	2006	2232	842	1775	2729
	2	4928	3783	3714	1657	968	1000	1381	-	2060	1574	-
Compressive Strength*	1	2.15	1.74	2.65	1.55	0.57	0.52	1.51	2.14	1.00	2.08	2.00
	2	1.93	1.60	3.98	1.85	0.49	0.47	1.25	-	1.80	2.30	-
Diagonal Tension Strength*	1	0.31	0.24	0.08	0.15	0.14	0.12	0.29	0.36	0.27	0.17	0.20
	2	0.24	0.27	0.12	-	-	-	0.22	-	-	0.19	-
Shear Strength*	1	0.09	0.14	0.20	0.06	-	-	0.19	0.09	0.19	0.09	0.21
	2	0.23	0.21	0.35	0.09	-	-	0.17	0.18	0.15	0.11	0.19

*: Units are in MPa. , ** Modulus of Elasticity is calculated from slope of secant line at 50% capacity

Table 3. Comparison of test results with the strength values recommended in TEC 2007

Properties	**Masonry Unit Type**					
	Hollow Clay Brick		**Solid Clay Brick**		**Cellular and Solid Concrete Brick**	
	Test	**TEC 2007**	**Test**	**TEC 2007**	**Test**	**TEC 2007***
Compressive Strength (MPa)	1.00-2.30	1.00-2.00	1.25-3.98	1.60	0.49-1.51	1.60
Shear Strength (MPa)	0.09-0.23	0.24-0.50	0.06-0.35	0.30	0.17-0.19	0.40
Diagonal Tension Strength (MPa)	0.17-0.31	NA	0.08-0.36	NA	0.14-0.29	NA

*: TEC 2007 provides capacities only for solid concrete brick

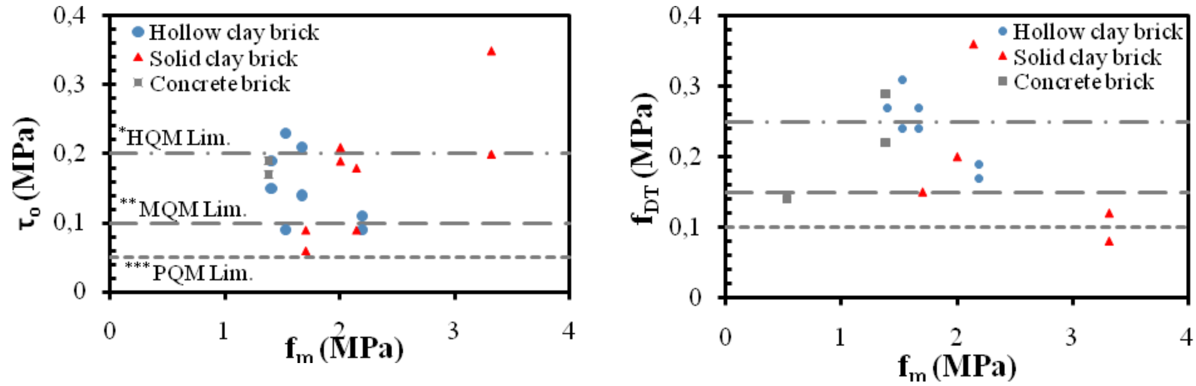

Fig. 1 – Relationships between compression strength with (a) shear strength and (b) diagonal tension strength (*: High Quality Mortar Limit, **: Medium Quality Mortar Limit and ***: Poor Quality Mortar Limit)

Table 4. Recommended strength values based on masonry unit types

Masonry Unit Type	Compressive Strength (MPa)	Shear Strength (MPa)			Diagonal Tension Strength (MPa)		
		Observed mortar quality			Observed mortar quality		
		Poor	Medium	High	Poor	Medium	High
Hollow clay brick	0.90	0.05	0.1	0.2	0.1	0.15	0.25
Solid clay brick	1.00						
Cellular concrete	0.40						
Solid concrete brick	0.85						

Table 5. Stiffness comparison with finite element model for selected buildings

Building ID	1	Moon (2004)	TEC 2007 [Flexural + Shear Stiffness]	Dolce (1989)	Full Height	TEC 2007 [Shear Stiffness only]
1	x	45%	43%	21%	12%	175%
	y	8%	130%	2%	4%	208%
2	x	10%	9%	5%	15%	37%
	y	1%	6%	9%	40%	126%
3	x	104%	126%	47%	8%	226%
	y	5%	2%	9%	8%	93%
4	x	75%	56%	11%	24%	167%
	y	42%	20%	26%	7%	98%
5	x	121%	183%	13%	25%	318%
	y	157%	159%	64%	27%	308%
6	x	15%	10%	3%	41%	153%
	y	54%	91%	32%	17%	201%
7	x	11%	13%	3%	30%	211%
	y	44%	56%	31%	29%	175%
8	x	12%	16%	28%	0%	111%
	y	43%	43%	12%	12%	149%
9	x	24%	53%	4%	22%	69%
	y	37%	114%	57%	19%	245%
10	x	28%	14%	19%	45%	94%
	y	21%	2%	7%	21%	68%
	Ave.	***43%***	***57%***	***20%***	***20%***	***162%***
	St. Dev.	***42%***	***57%***	***18%***	***13%***	***77%***

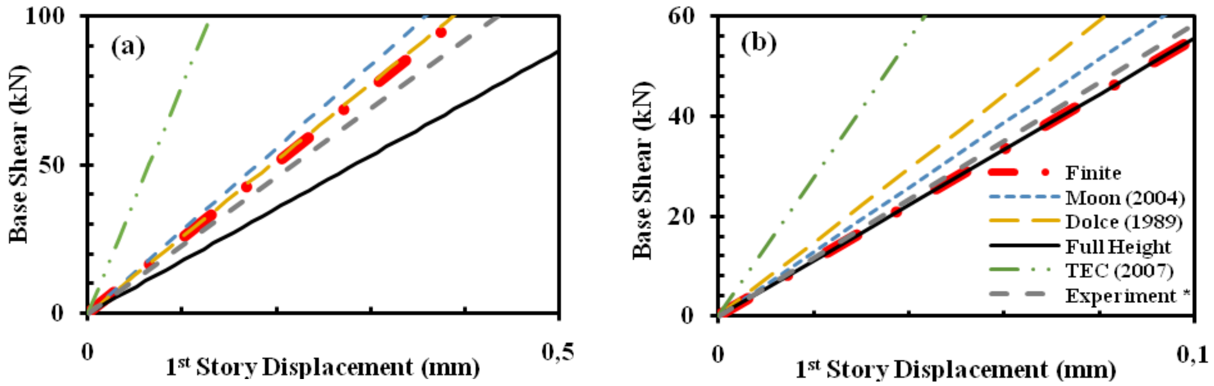

Fig. 2 – Stiffness comparison from in-situ experiments: (a) B7 and (b) B8 *Secant stiffness is used for the experiment.

Table 6. Stiffness comparison with site experiments

Method	Percentage Error (%)	
	B7	**B8**
Finite Element Model	9.4	-5.2
Moon (2004)	20.9	6.2
TEC 2007 [Flexural + Shear Stiffness]	22.5	9.7
Dolce (1989)	***11.9***	21.4
Full Height	-23.3	***-5.1***
TEC 2007 [Shear Stiffness only]	232.9	100.5

While determining the acceptance limit of 35%, the pushover analysis results of the selected buildings are utilized. For each direction of the selected buildings, the capacity curves are obtained. Therefore, the analysis technique is verified firstly by comparing the results with the pushover curves obtained from the site tests. For example, the capacity curve estimation for B8 is compared with the experimentally obtained capacity curve in Fig. 3. It is clear from Fig. 3 that the ultimate base shear strength and second story shear force could be estimated with an accuracy of within 5% by using the analysis technique. Then, the elastic base shear demand is determined by utilizing equal energy rule (Fig. 4). After that, the structure is assessed under the effect of this elastic base shear demand. The acceptance limit is determined in order to classify all of the buildings as HR. This value comes out to be 35%. Therefore, this method makes possible to assess unreinforced masonry structures correctly even if they are expected to take severe damage as the method bases on experimental data obtained from site tests on masonry building near collapse.

Table 7. Demand capacity ratio limits

Failure Mode	Capacity	m_{lim}
Sliding	$(\tau_o + \mu \times \sigma) \times L \times t$	1.5
Diagonal Tension	$0.75 \times f_{dt} \times L \times t \times (1+\sigma/f_{dt})^{0.5}$	1
Rocking	$N \times L/H \times (1-(N/0.8 \times fm \times L \times t))$	2.5

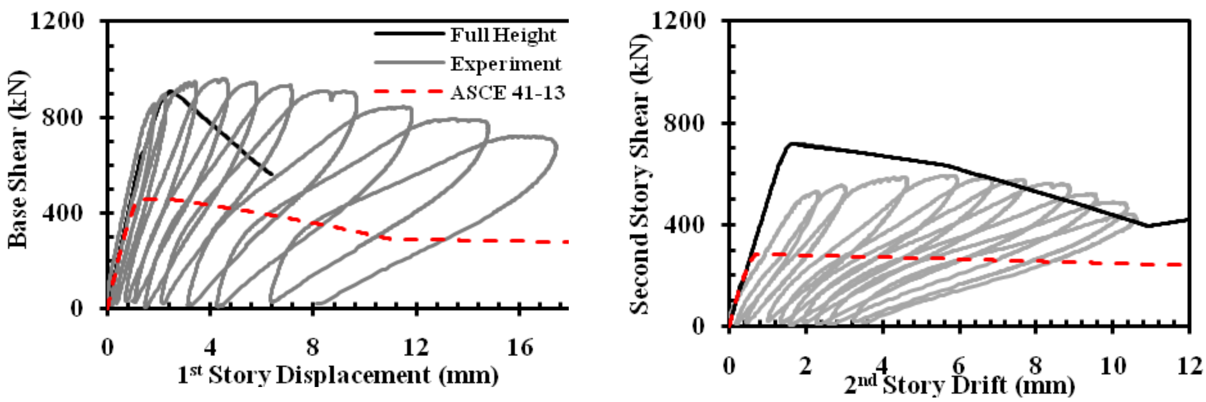

Fig. 3 – Analysis results for test building B8

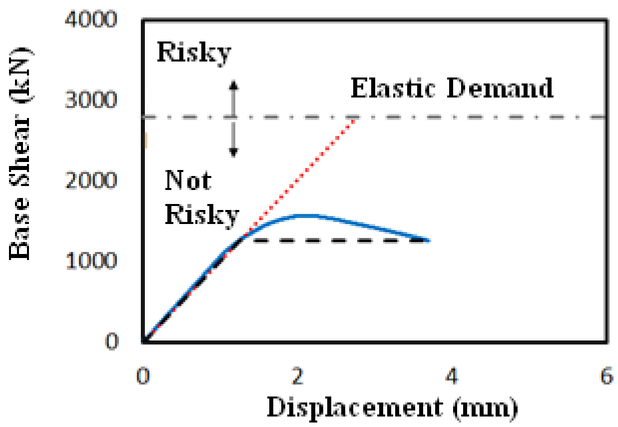

Fig. 4 – Elastic demand determination for B4

5 CONCLUSIONS

This study presents a brief summary of an extensive research program composed of in-situ material strength determination from ten existing masonry buildings, two building tests including forced vibration and lateral load testing, and numerical simulations for calibration and modelling of existing masonry buildings. The material properties of different types of masonry walls extracted from existing buildings were determined in laboratory environment. The obtained strengths were compared with the code recommended values and the impact of different strength values on the assessment results was investigated. The following conclusions can be drawn on the basis of the conducted research:

- Full height approach for the effective height calculations of piers resulted in the best stiffness match by using the finite element model and the simplified pier model in both directions of test building.
- According to the laboratory tests, the compressive strength of masonry walls depended on the masonry unit type as expected and stated by TEC 2007. Test results showed very weak correlation between diagonal tension and sliding shear strengths. Instead, those material properties were more related to the quality of mortar.
- The proposed method mainly depends on the site experiments of different masonry structures whose material strengths are available. In other words, the assessment procedure and its acceptance limit are derived from an extensive laboratory and numerical research. Therefore, this method is calibrated to result in safe and correct evaluation results.

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