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Lateral load on site testing of a two story masonry building up to near collapse

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ABSTRACT: Masonry structures constitute a significant portion of the building stock in many countries with high seismic regions. Understanding the load carrying mechanisms of such buildings as a system and estimating the deformation capacities still remains as an important task for seismic assessment and retrofit. In this research, one-way cyclic test of an existing masonry building was performed to determine its lateral load resisting behaviour. The test building was sliced approximately in the middle and one side was strengthened with the objective of obtaining a strong reaction wall. The other side of the structure was taken as the test structure with a floor plan of approximately 10 m x 8 m. Hydraulic actuators located in the two stories were employed to impose one-way cyclic displacement excursions. The structure was tested up to a lateral strength drop of approximately twenty percent from its ultimate load, which occurred at a drift ratio of about 1%. The failure of the walls in the building, which were mostly diagonal tension, was concentrated on the first story. Results provide important data on the performance of an actual masonry building and were employed for the applicability of various stiffness and strength models in the literature.

1 INTRODUCTION

Masonry has still been utilized for centuries in rural and even in urban regions of developing countries. Yet, the strength and stability of masonry structures are mostly insufficient under the effect of strong ground motions. Hence, the masonry buildings belong to the most vulnerable structural types as they have experienced heavy damage or even total collapse in previous earthquakes. In addition, there is no commonly accepted method in literature on how to determine the capacity as well as the performance of unreinforced masonry structures. This is mostly because of the anisotropic and heterogeneous nature of this construction material. In literature, there is a constant effort to understand the behaviour of masonry structures and their components by performing experimental researches. For example, the behavior of unreinforced masonry walls and spandrel beams have been investigated in detail by considering the aspect ratio, material type and boundary conditions as test variables in numerous studies (Franklin et al. 2001, Paquette and Bruneau 2003, Beyer and Dazio 2012 and Beyer 2012). In addition, laboratory tests on small building models (Magenes et al. 1995, Magenes and Calvi 1997, Yi 2004, Shahzada et al. 2012) are conducted to better understand the stiffness, strength, deformability and damage patterns of sub-assemblages, which are valuable for developing and calibrating numerical models for seismic assessment. Recent studies have focused on key factors such as behavior of spandrel beams, flange effects in walls or out-of-plane behavior of walls, which contribute to more refined seismic performance assessment procedures (Russell et al. 2014).

The knowledge obtained from both experimental and numerical researches enabled to put forth different rules to assess existing masonry structures, i.e. FEMA 356, ASCE/SEI 41-13, Eurocode 6, etc. In Turkish practice, the seismic assessment procedure provided by the Turkish Earthquake Code (TEC 2007) and Guidelines for the Assessment of Buildings under High Risk (GABHR 2013) is regulated. In these codes, the seismic assessment method for masonry buildings is carried out by utilizing inelastic spectrum obtained by dividing elastic design spectrum by response modification factors to compare the wall shear stresses under the effect of vertical and lateral loads with the strength limits. This procedure is no different than the procedure recommended to design new masonry structures.

The aforementioned seismic assessment method for masonry buildings has two major drawbacks: i- masonry material strength default values for different type of units suggested by TEC (2007) for seismic assessment calculations are used. These values, however, are not known with sufficient accuracy to be representative of the actual masonry strength in existing buildings, ii- The assessment method employed based on assuming a response modification factor (i.e. R=2) similar to the factor in new design lacks any theoretical and practical basis for an existing structure. These two important deficiencies of the existing techniques are sometimes found to render incorrect risk classification. To this end, in this study, an in-situ pushover experiment is implemented on a two-story residential masonry building which is composed of hollow clay bricks. The capacity curve of the aforementioned building is obtained by applying lateral loads compatible with the shape of the fundamental mode at each story. A displacement-controlled loading protocol is applied manually. At every aimed displacement, the structure is unloaded to its zero force position and re-loaded to the new target displacement in order to obtain the energy dissipation characteristics. The crack propagations on every wall in each story are also observed and the failure mechanisms of different walls are noted.

2 INFORMATION ON TEST BUILDING

The test building is located in the Northern part of Ankara. The building is a two-story masonry structure with a floor plan area of about 17 m x 10 m made of hollow clay bricks. The floor plan of the test building is shown in Fig. 1. The building composed of masonry walls with reinforced concrete beams supporting slabs with thickness of 0.12 and 0.10 m in the first and second stories, respectively. In order to estimate the material properties prior to testing, wallettes having 0.90m x 0.90m dimensions were extracted underneath window openings and tested to determine uniaxial compression, diagonal tension, shear strength as well as the modulus of elasticity. Material testing apparatus and results are presented in Fig. 2. The average uniaxial compressive, diagonal tensile, shear strength and modulus of elasticity were determined as 1.0 MPa, 0.27 MPa, 0.17 MPa and 842 MPa, respectively.

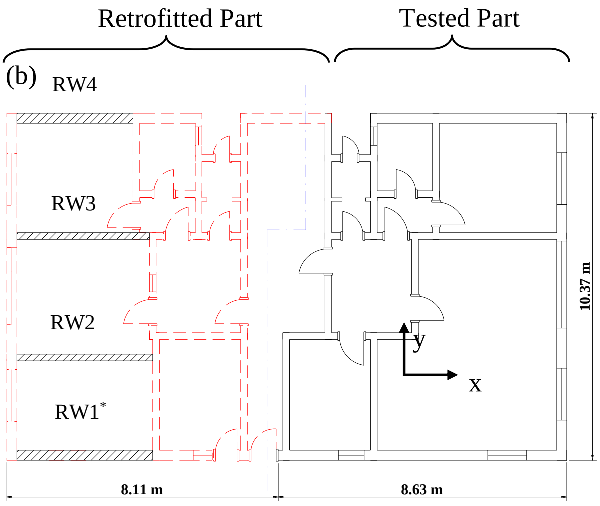


Fig. 1 – Test structure : (a) Photo and (b) Plan view of tested masonry structure (*: Retrofitted Wall)

A stiff reaction wall should be built to apply lateral loads by hydraulic actuators. To this end, the test building was sliced into two parts, having nearly equal sizes (Fig. 3). The west side of the building was then employed as a stiff reaction wall upon strengthening four walls shown as hatched areas in Fig.1 with external mesh reinforced mortar. In this way, reactions from the application of the load to the east side of the building could be sustained without any significant deflections and damage. Prior to lateral load testing of the building, a forced vibration test was performed in order to estimate the fundamental frequency of the building. The FFT of transfer function for recorded acceleration are shown in Fig. 4. The fundamental vibration period of the building was determined as 0.125 sec from forced vibration test. This result showed a good agreement with the analytical estimation of 0.128 sec obtained from finite element model employing a modulus of elasticity of 842 MPa (Fig 2.d). In the finite element model, approximately 55,700 eight-node shell elements are utilized by assuming the base nodes to be fixed for any translational and rotational effects. The numerical model also estimated the direction of the first fundamental mode correctly, i.e. coinciding with the testing direction.

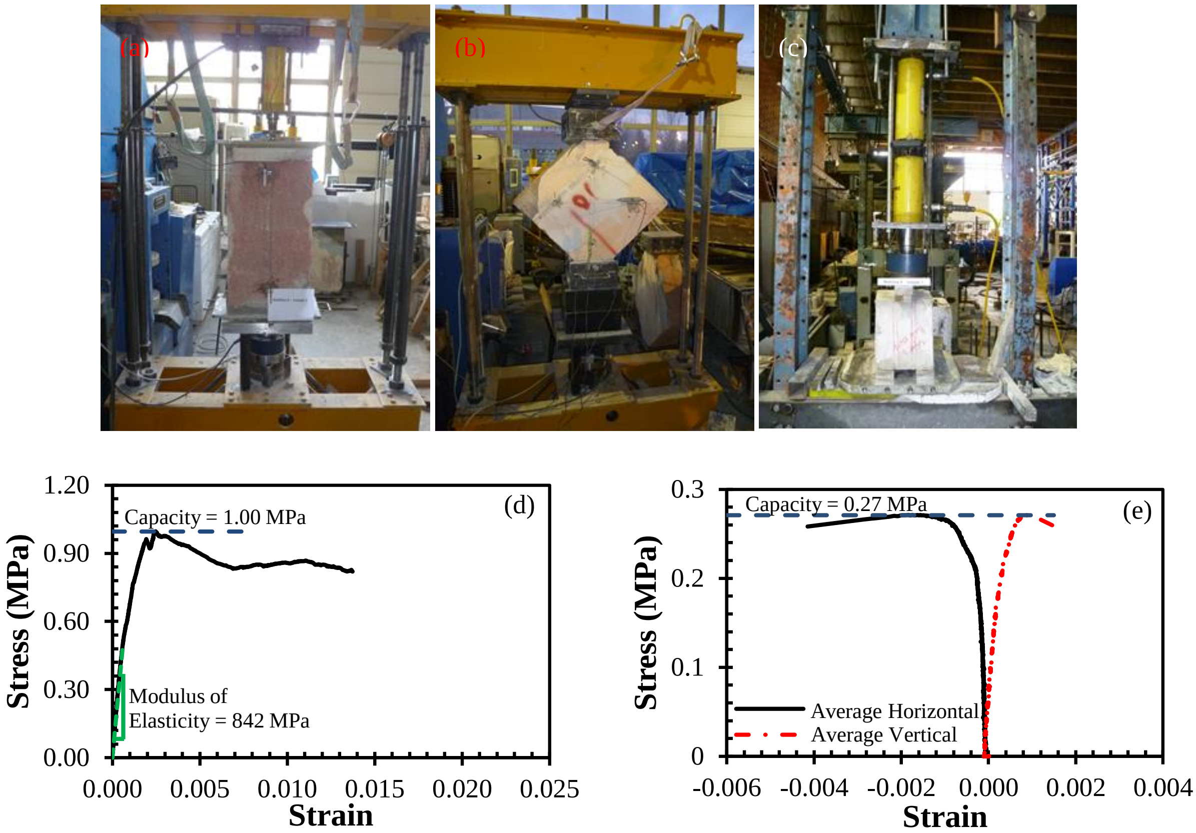


Fig. 2 – Material test setups and test results : (a) Compression test setup , (b) Diagonal tension test setup, (c) Triplet test setup, (d) Compressive stress-strain curve and (e) Diagonal tension stress-strain curves

3 INSTRUMENTATION AND TESTING

Firstly, four hydraulic actuators were attached along the slice locations on both floor slabs. At this point, the resultant of the force at each floor caused by hydraulic actuators was enforced to coincide with the center of mass of each floor. The hydraulic actuators were connected by steel attachments that ensured proper transfer of shear forces without any local failures (Fig. 5). The loading was adjusted by an electric controlled oil pump in order to apply the imposed displacements. The lateral load ratio of the second floor to that of the first floor was 1.7. This ratio was determined from the estimation of the shape of the first fundamental mode obtained from finite element analysis. The lateral displacements of each story level are recorded with Linear Variable Differential Transformer (LVDTs) installed at four different points.

The locations of these LVDT's are also presented in Fig. 6. In addition, the deformations of the walls were measured by using four LVDTs, two measuring vertical deformations, two recording diagonal deformations as shown in Fig. 7.a for a representative wall. The testing was conducted by controlling the interstory drift ratio of the first floor in a one way cyclic manner. The applied displacement protocol on the first floor and roof of the building are shown in Fig. 7.b**.**

Fig. 3 – Photos of interface between retrofitted and tested structures

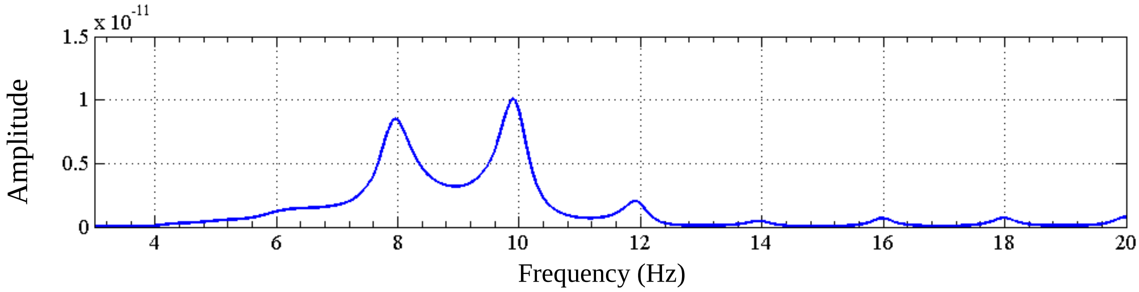


Fig. 4 – FFT of transfer function for recorded accelerations

4 TEST RESULTS

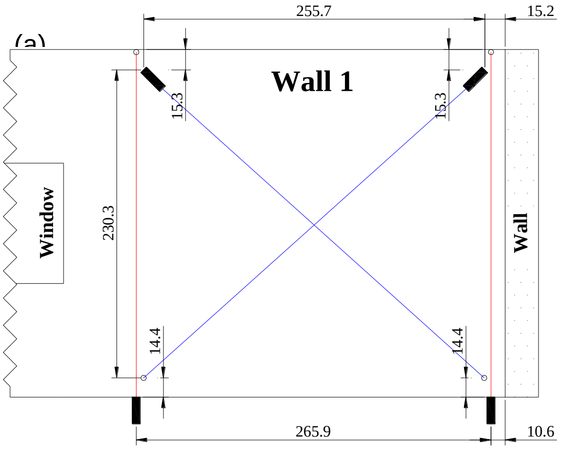
Measured load deformation response of the building is shown in Fig. 8.a-8.c in terms of first drift - base shear force, second story drift - second story shear force and roof displacement – base shear force. The deformation measurements at the center of mass are also presented as interstory drift ratios which were found by dividing the story displacement with the story height in those graphs. The deformation profile along the height of the building is presented in Fig. 8.d. The damage pictures of the selected walls are shown in Fig. 9. The building behaved in an elastic manner up to a total base shear of about 200 kN beyond which crack initiation started resulting in a reduction of tangent stiffness. The interstory drift profile along the height of the building was nearly uniform throughout the testing except for the last drift profile which clearly demonstrates the relatively larger stiffness loss in the first story due to the enhanced damage accumulation on first story walls. Significant nonlinear response initiated at a base shear force of 800kN. The ultimate base shear force recorded during the test was 950kN at overall building drift ratio of about 0.18%. Force-displacement relationship of both floors exhibited softening beyond this drift ratio showing that the damage occurred on the walls of both stories with increasing displacements. The interstory drift ratio at 20% strength drop for the first and second stories occurred at about 0.5% and 0.38% interstory drift ratios. This result shows that the ductility capacity of the building was about 5.2 assuming the yield point obtained from the idealized bilinear response curve shown in Fig. 8.c. Cracking patterns observed in walls 1, 3, 7 and 9 are shown in Fig. 9. It can be observed that horizontal cracks occurred between the horizontal beams and walls followed by diagonal cracks. It is interesting to note that even for walls with relatively large aspect ratio (H/L), diagonal cracking was the dominant failure mode as opposed to expected rocking failures. This observations support the recent findings of Russell et al. (2014) who point out the importance of flange effects for accurate failure mode and strength estimation of masonry walls.

Fig. 5 – Photos of hydraulic actuators at each story : (a) First story and (b) Roof

Fig. 6 – Photos of LVDT's installed at each story level

5 CONCLUSIONS

The capacity curve of a two-story brick masonry building was determined from a one-way cyclic experiment conducted on-site. The fundamental frequency of the test building was estimated with an error of less than 5% by finite element model by utilizing the modulus of elasticity obtained from wallette compression tests. The interface cracks between horizontal beams and the masonry walls were initially observed. After that, these interface cracks proceeded through the masonry walls to form diagonal tension cracks, which predominantly caused the capacity loss of nearly all of the walls. Interestingly, this observation was valid for each wall independent from the aspect ratio (height to depth ratio). Therefore, the flange effect should be taken into account for predicting the correct failure mode and capacity. The ratio of base shear capacity to total weight of building was determined as 0.6, which validated that the assumption of R = 2 was a conservative approach for masonry buildings. Moreover, the displacement ductility of the test building was obtained as 5.2. The test building showed a significant stiffness loss after a drift ratio of 0.1-0.2% and it started to considerable strength degradation at a drift ratio of 0.5%. This important observation gives a clue on the displacement capacity of unreinforced masonry structures, at least an order of magnitude.



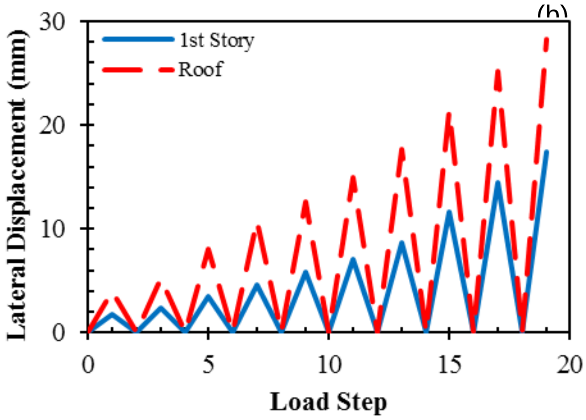


Fig. 7 – (a) The installed LVDT's for a representative wall and (b) Control displacements applied to each story *All units are in cm.

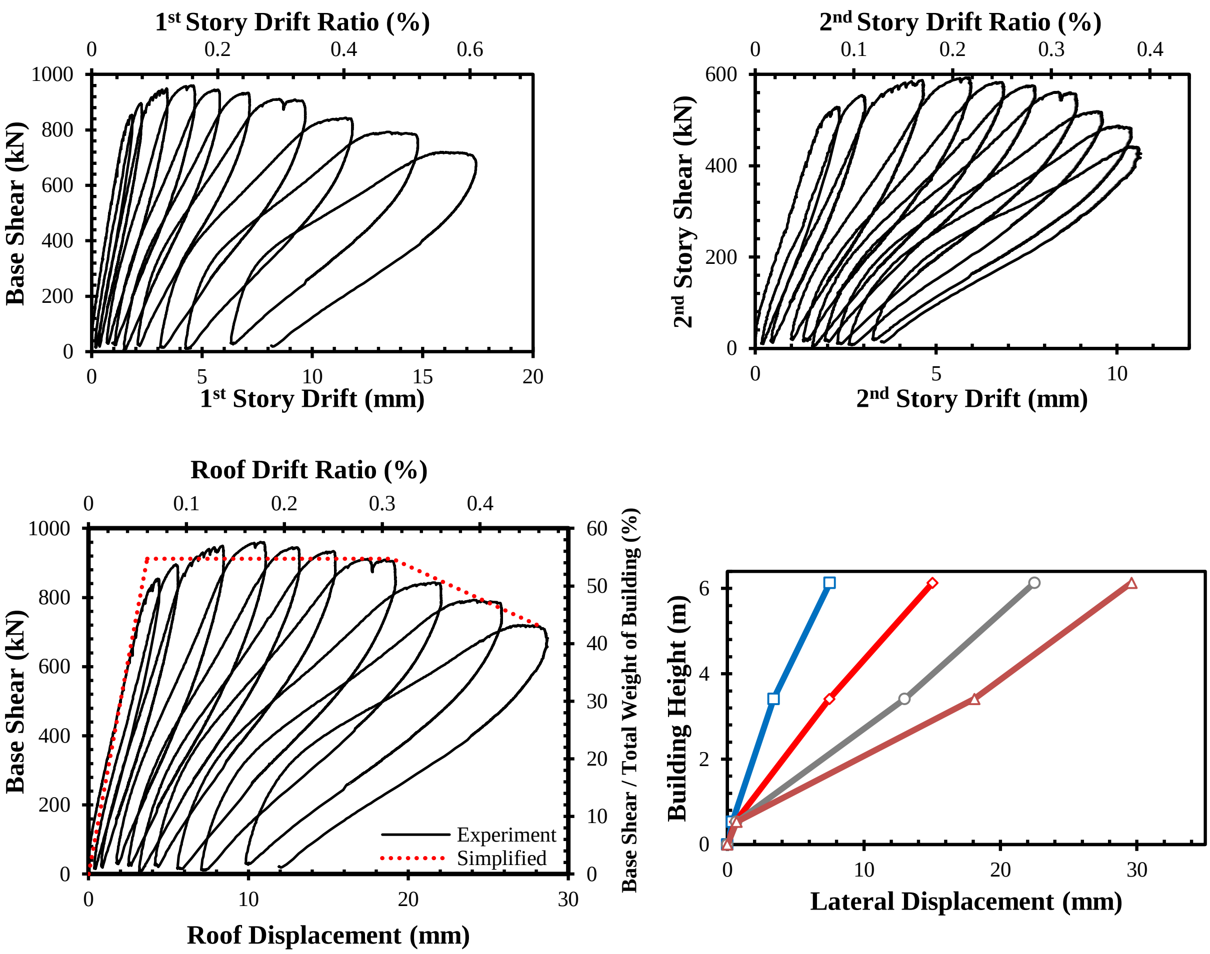


Fig. 8 – Recorded load – deformation responses

Fig. 9 – Observed wall damages

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