The 2015 World Congress on
Advances in Structural Engineering and Mechanics (ASEM15)
Incheon, Korea, August 25-29, 2015

Lateral Load Testing of Two Existing Masonry Buildings

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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 and estimating the deformation capacities is important for seismic risk mitigation. In the process of developing new assessment guidelines for Turkey, a comprehensive research project was conducted in order to estimate the expected seismic performance of existing masonry buildings. Within the scope of this extensive research program, two existing two-story masonry buildings were tested under lateral cyclic loads on site. For one of the test buildings, a seven-meter-high steel reaction frame capable of resisting 3000kN base shear force was constructed next to the test building. The second building was saw-cut in the middle so that one side of the building was strengthened and acted as a reaction wall during the testing of the other side. The two structures were tested up to a lateral strength drop of approximately twenty percent from the ultimate load. The damage and deformation in the walls were closely monitored during the tests. This paper summarizes the key results and outcomes of this comprehensive research program.

Keywords: In-situ, Pushover Test, Brick Masonry

1. INTRODUCTION

Masonry has still been utilized for centuries in rural and even in urban regions of developing countries as it has the advantages over the other structural materials like widespread geographic availability in many forms, colors and textures, its economical nature for construction, fire resistance, thermal and sound insulation, durability, etc. Unfortunately, the strength and stability of masonry structures are critical in the case of cyclic lateral loads, i.e. under the effect of strong ground motions. Hence, the masonry

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buildings belong to the most vulnerable structural types as they have experienced heavy damage or even total collapse in previous earthquakes as documented from previous earthquakes like Elazığ-Turkey 2010, Bam-Iran 2003, Kashmir-Pakistan 2005, L'Aquila-Italy 2009. The design and modeling tools for these structures are rather primitive than their competitors for low rise building construction. 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 masonry. Therefore, the experimental techniques are widely utilized to understand the behavior of masonry structures and their components.

In the literature, there are some experimental efforts to determine the response of spandrels and piers. In those researches, the effects of aspect ratio, material type and boundary conditions were investigated in detail (Franklin et al. 2001, Paquette and Bruneau 2003, Beyer 2012a and Beyer and Dazio 2012b). The laboratory tests on small building models (Magenes et al. 1995, Magenes and Calvi 1997, Yi 2004, Shahzada et al. 2012) were also performed to examine the stiffness and strength characteristics of masonry assemblages and these experimental data were used to calibrate numerical models for design and assessment of masonry structures. 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 present 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.

In this study, in-situ pushover experiments were conducted on two two-story residential masonry buildings. The capacity curve of these buildings are determined by applying lateral loads compatible with the shape of the first fundamental modes at each structure. During each test, a manually-implemented displacement control is utilized. 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 BUILDINGS AND INSTRUMENTATIONS

2.1. Test Building 1

The first test building is located in the Northern part of Ankara, Turkey. The building is a two-story masonry structure. It has a floor plan of about 17 m x 10 m (Figure 1) and composed of hollow clay bricks. The building is comprised of masonry walls with reinforced concrete beams supporting slabs with thicknesses of 0.12 and 0.10 m in the first and second stories, respectively. Before the lateral load test of building, square wallettes with 0.90m x 0.90m dimensions were extracted underneath window openings to determine the material properties. These wallettes were tested in laboratory to determine the uniaxial compression, the diagonal tension, the shear strength under zero compressive stress along with the modulus of elasticity. Material testing apparatus and sample test results are exhibited in Figure 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.

The stiff reaction wall is required to apply lateral loads on the test structure. Therefore, the test building was sliced into two parts by using saw-cutters (Figure 3). In this way, the lateral loads could be applied by leaning on the west side of the building after strengthening four walls lying in that part of the building shown as hatched areas in Figure 1 with external mesh reinforced mortar.

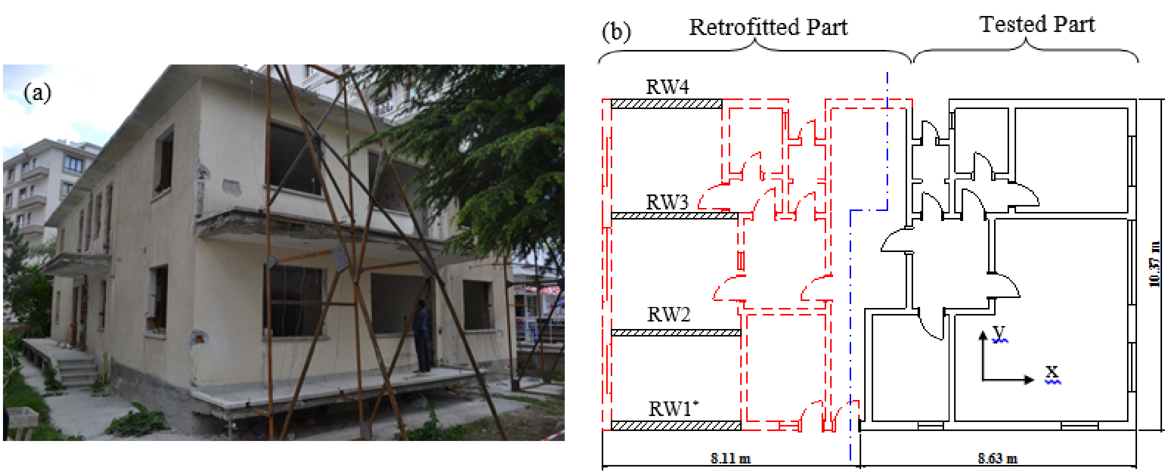


Figure 1. The First Test Structure: (a) Photo and (b) Plan View

Firstly, four hydraulic actuators were installed along the slice locations on both floor slabs (Figure 4). In the determination of hydraulic jack locations, it was aimed to have the resultant force at each floor coinciding with the mass centre of each floor. The connections of hydraulic actuators were secured by steel attachments to have a proper shear transfer without any local failures (Figure 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, which was exactly equal to the ratio of story displacements for the first fundamental mode shape estimated from finite element analysis.

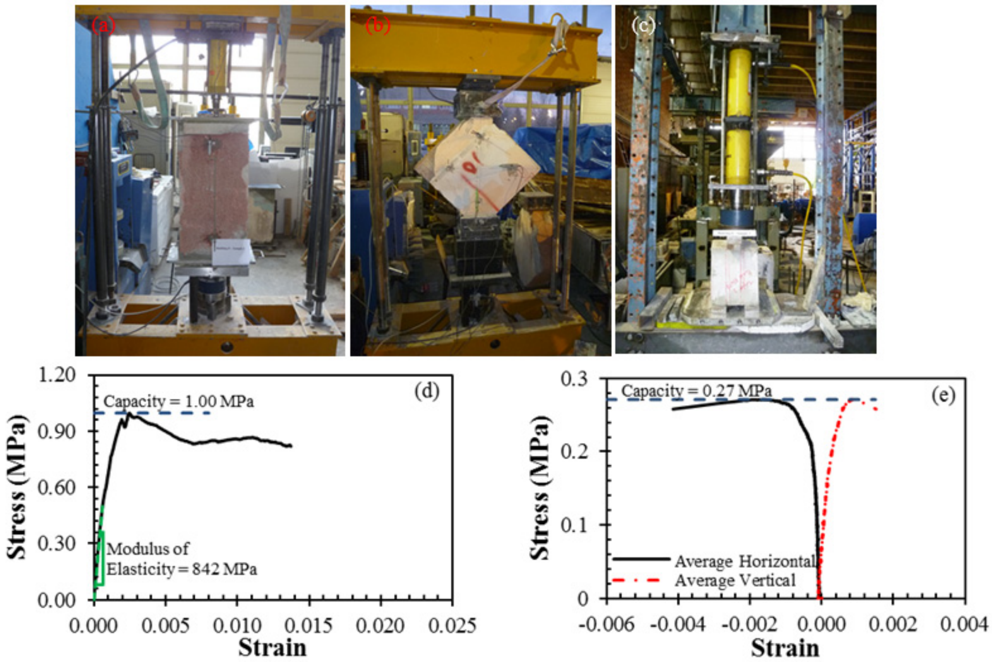
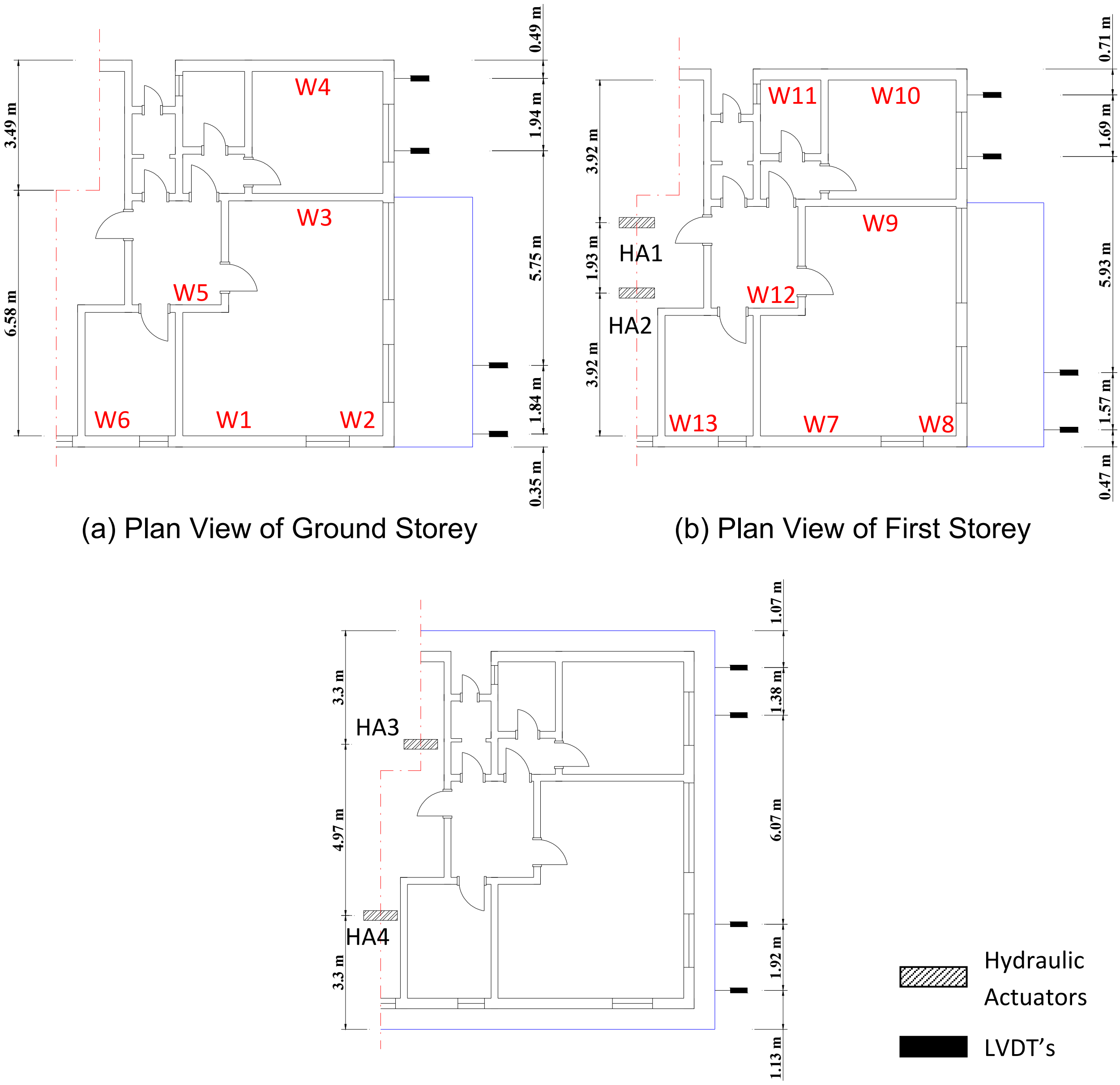


Figure 2. Material Test Setups and Test Results : (a) Compression Test Setup , (b) Diagonal Tension Test Setup, (c) Triplet Test Setup, (d) Compressive Stress-Strain Curve and (e) Diagonal Tension Stress-Strain Curves

Figure 3. Photos of Interface between Retrofitted and Tested Structures

The lateral displacements of every stories were recorded with Linear Variable Differential Transformers (LVDTs) installed at four different points. The locations of these LVDT's are also shown in Figures 4-6. In addition, the deformations of the walls were measured by using four LVDTs, two measuring vertical deformations, two



(a) Plan View of Ground Storey

(b) Plan View of First Storey

(c) Plan View of Roof

Figure 4. Locations of Hydraulic Actuators and LVDT's

recording diagonal deformations as shown in Figure 7.a for a representative wall. During the test, the control parameter was the interstory drift ratio of the first floor.

The test was conducted in a one way cyclic manner. The applied displacement protocol on the first floor and roof of the building are shown in Figure 7.b.

(a) Hydraulic Actuators at First Story

(b) Hydraulic Actuators at the Roof

Figure 5. Photos of Hydraulic Actuators at Each Story

2.2. Test Building 2

The second test building is also located in the Northern part of Ankara, Turkey. This building is a two-story masonry structure with a floor plan of about 10 m × 9 m (Figure 8) made of solid clay bricks. Just like the first building, this building had RC bond beams underneath slabs with thicknesses of 0.13 and 0.10 m in the first and second stories, respectively. The material properties of the test building were determined before lateral load tests. To this end, square wallettes of 90cm × 90cm sizes were extracted under window openings. The wallette specimens were subjected to uniaxial compression, diagonal tension and triplet tests. The test results reveal that the average uniaxial compressive, diagonal tensile and shear strength were 2.14 MPa, 0.36 MPa and 0.17 MPa, respectively (Figure 9).

Figure 6. Photos of LVDT's installed at Each Story Level

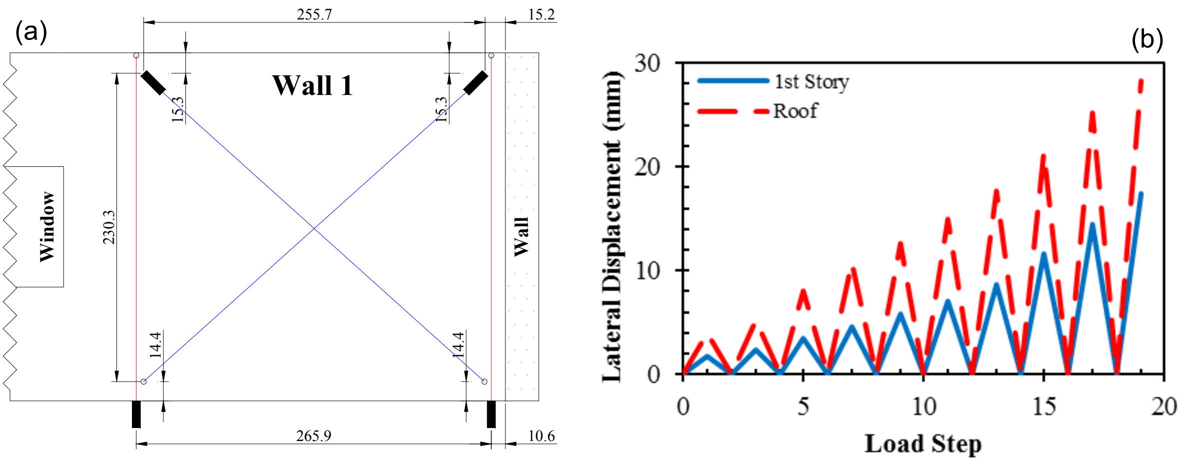


Figure 7. (a) The installed LVDT's for a Representative Wall and (b) Control Displacements applied to Each Story (All units are in cm.)

The lateral load testing of the existing building required a stiff reaction wall to apply lateral loads with the hydraulic actuators. For this purpose, a seven-meter-high steel reaction frame was built next to the test building (Figure 10). This reaction steel frame and its reinforced concrete foundation was designed to resist a maximum base shear force of nearly 3000 kN. Afterwards, hydraulic actuators were placed to the reaction frame and displacement excursions were imposed in a one-way cyclic manner at each story level. The shape of the lateral loading was obtained from the modal analysis of the building in computer environment. In the finite element model, approximately 66,600 eight-node shell elements were utilized. The nodes at the base of the model were assumed to be fixed. The first fundamental vibration mode along the testing direction had an effective modal mass of about 75% (Figure 11). This relatively high effective mass contribution for the first fundamental mode supported the idea of utilizing pushover test to obtain the capacity curve of this building. The ratio of the first and second story lateral forces was assumed to be similar to the first modal shape (1:1.78 ratio for the first and second story applied forces).

The test of the building was conducted by using four hydraulic actuators attached between the steel frame and the test building. The actuators were placed such that the resultant of the force at each floor would approximately coincide with the centre of mass of each floor. The lateral displacements of each story level were recorded with Linear Variable Differential Transformer (LVDTs) installed at four different locations at the ground, first and second stories. The locations of these LVDT's along with the hydraulic pistons are also presented in Figure 11.

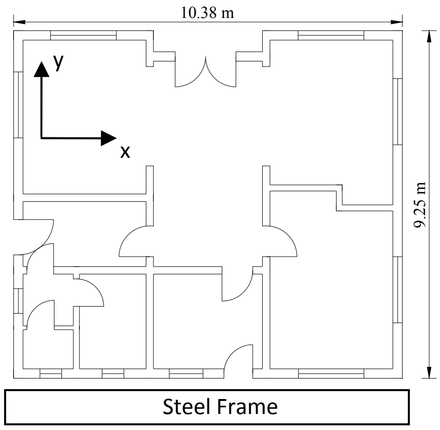
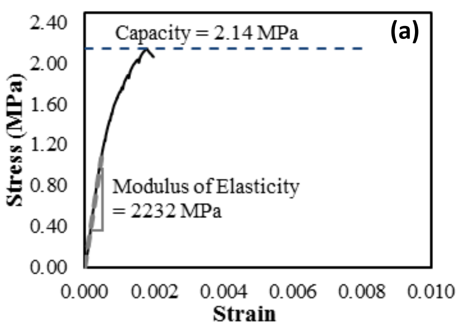


Figure 8. Photo and plan view of the second test structure



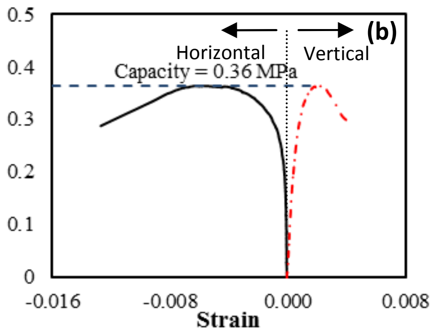


Figure 9. (a) Compressive Stress-Strain Curves and (b) Diagonal Tension Stress-Strain Curves

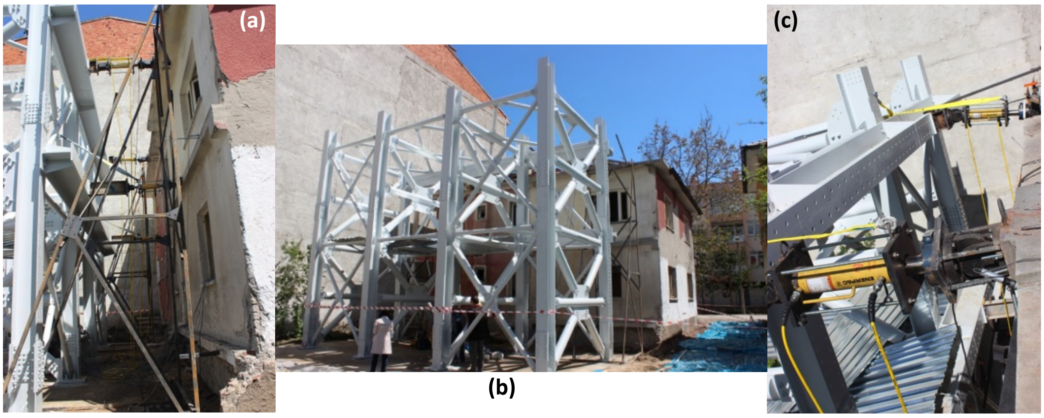


Figure 10. (a) Side view of LVDT's and hydraulic pistons, (b) photo of steel frame and (c) photos of hydraulic pistons at the second story

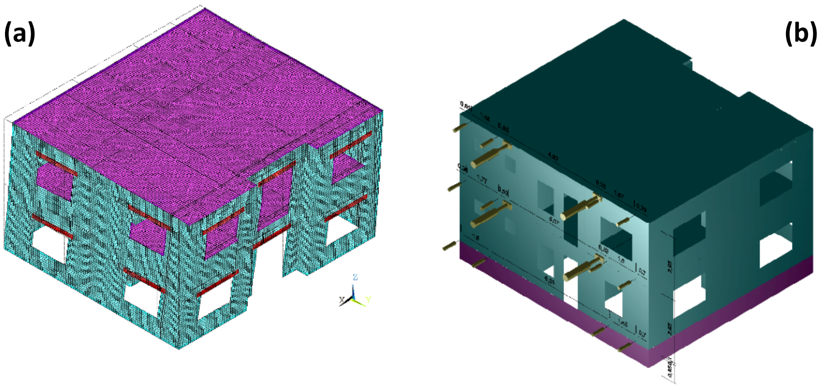


Figure 11. (a) Finite element model and (b) first mode shape

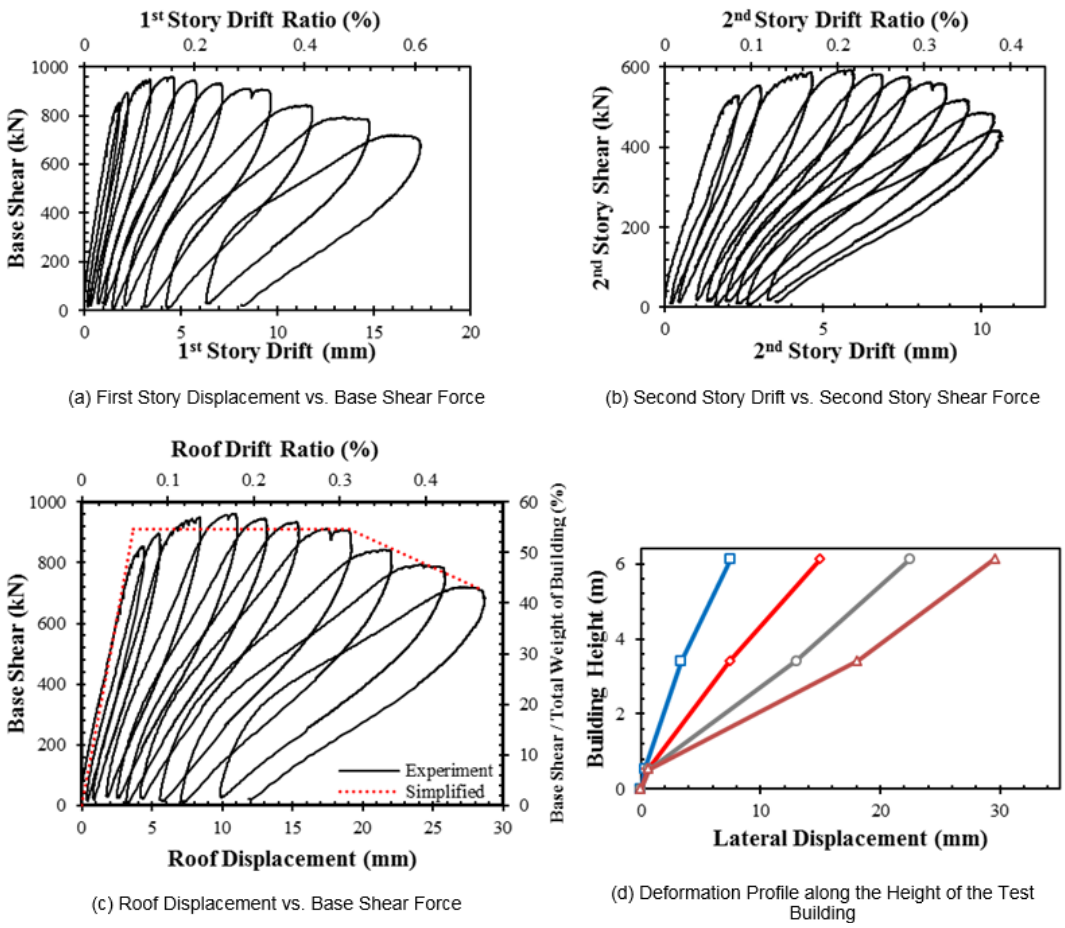
3. TEST RESULTS

3.1. Test Building 1

Measured load deformation response of the building is presented in Figure 12.a-12.c. In these figures, the deformation measurements at the mass centre are exhibited determined by dividing the story displacement with the story height in those graphs. The profile of story displacements and the plan deflections for the different roof displacement demands are shown in Figures 12.d and 13, respectively. The damage pictures of the selected walls are also presented in Figure 15.

The building was observed to stay in its elastic limit up to a total base shear of approximately 200 kN. After this load, cracks started to initiate causing a reduction of tangent stiffness. The drift profiles was nearly uniform throughout the experiment except for the last drift demand. In the last cycle, a relatively larger stiffness loss in the first story was detected stemmed from the enhanced damage accumulation on the walls in that story. 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 showed that the ductility capacity of the building was about 5.2 assuming the yield point obtained from the idealized bilinear response curve shown in Figure 12.c. Story displacement profiles shown in Figure 13 exhibit that the building experienced slight torsional rotations in the counter clockwise direction at both story levels with increasing nonlinearity due to asymmetric wall damage.

Cracking patterns observed in walls 1, 3, 7 and 9 are shown in Figure 11. Horizontal cracks occurred between the horizontal bond beams and walls. Afterwards, diagonal cracks started to initiate. 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.



(a) First Story Displacement vs. Base Shear Force

(b) Second Story Drift vs. Second Story Shear Force

(c) Roof Displacement vs. Base Shear Force

(d) Deformation Profile along the Height of the Test Building

Figure 12. Recorded Load – Deformation Responses

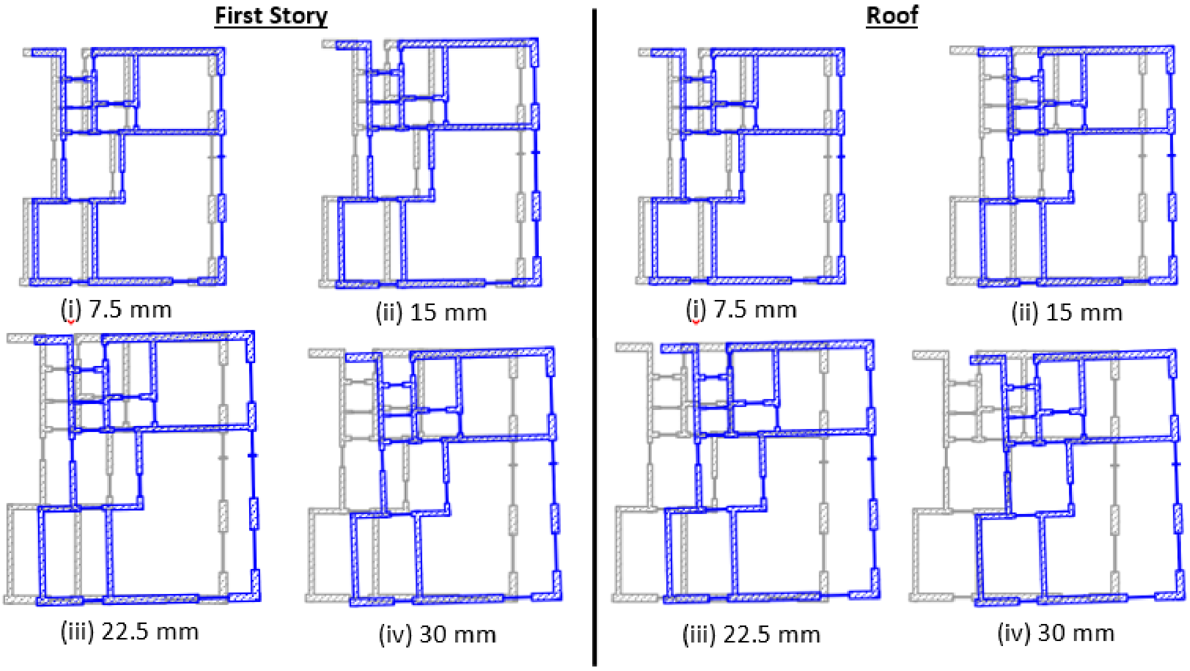


Figure 13. Plan Deflections at Different Roof Displacements for Each Story Level

3.2. Test Building 2

Measured load deformation response of the building is shown in Figure 14 for the centre of mass location. In addition, the deformation profile along the height of the building and the plan deflections at different roof displacements are presented in Figure 16. The damage pictures of the test building are shown in Figure 17.

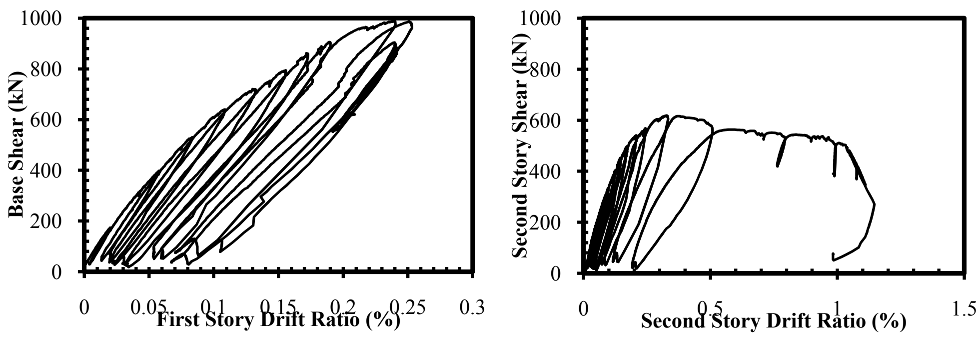


Figure 14. (a) Finite element model and (b) first mode shape

Figure 15. Observed Wall Damages

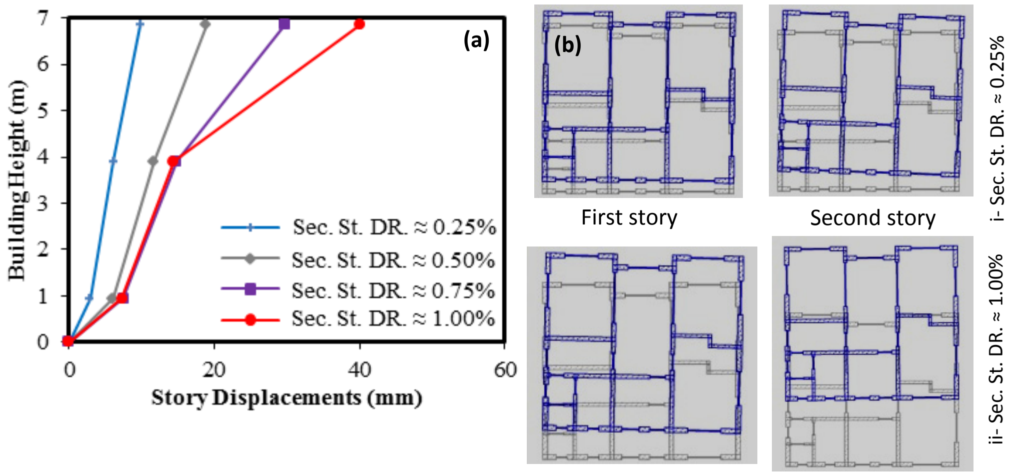


Figure 16. (a) Deformation profile at different drift ratio levels of the second story and (b) plan deflection of first and second stories at a drift ratio of (row i) 0.25% and (row ii) 1.00%

Figure 17. Different cracks observed during one-way cyclic test

The building remained nearly elastic up to a base shear force of about 400 kN. After this point, the lateral stiffness of the test building started to decrease due to the crack formation. The drift profile along the height of the building was detected to be approximately uniform until an interstory drift ratio of 0.50% at the second story (Figure 16.a). Afterwards, the lateral stiffness loss at the second story was relatively larger due to the enhanced damage accumulation both on the second story walls and at the interface between the slab and the second story walls. In fact, at a base shear of nearly 900 kN, the slab at the second story started slide over the second story walls (Figure 14). This damage caused a ductile response in the load-deformation response of the second story.

The ultimate base shear force recorded during the test was 990kN at overall building drift ratio of about 0.35%, corresponding to a nearly 0.25% of interstory drift ratio at each story level. The test was stopped when the second story drift ratio reached about 1.2% since the test second story lost 20% of its ultimate capacity. According to the plan deflection sketches of the test building (Figure 16.b), the building experienced slight torsional rotations in the clockwise direction at both story levels. The damage of the first story walls were relatively minor during the one-way cyclic displacement excursions. However, the second story walls were damaged heavily (Figure 17).

4. CONCLUSIONS

The capacity curve of two two-story brick masonry buildings were determined from one-way cyclic experiments conducted on-site. The following conclusions can be drawn on the basis of the observed response of the first test building:

- 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, accordingly the 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.

The following conclusions can be drawn on the basis of the observed response of the second test building:

- The structure was pushed until 20% of its ultimate lateral load carrying capacity occurred. This corresponded to a drift ratio of about 1.2% at the second story level. In the light of experimental findings, the lateral load carrying capacity of the building is detected to be approximately 40% of its weight.
- From the capacity curve, the displacement ductility of the test building was determined as about 4.
- The observed wall cracks were mostly diagonal tension. Furthermore, sliding shear failure between the second floor slab and walls were detected.
- The results presented herein provide invaluable information on the expected performance of an actual building and will be used in the calibration of numerical models and assessment techniques.

5. ACKNOWLEDGEMENTS

Authors acknowledge the financial support provided by Çevre Şehircilik Kentsel Dönüşüm ve Altyapı for conducting this study. The contributions of Baran Çobanoğlu, Salim Azak and METU Structural Mechanics Laboratory workers Osman Keskin, Murat Demirel and Barış Esen during the site and laboratory work are also gratefully acknowledged.

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