Experimental Assessment of the Seismic Behavior of Load-Bearing Masonry Walls Loaded Out-of-Plane

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Abstract

The typical housing type of the rural population of Turkey is load-bearing masonry units. Masonry buildings are as vulnerable to seismic failure as reinforced concrete buildings. However, the majority of research efforts are directed towards reinforced concrete buildings.

Masonry walls that constitute masonry structures are subject to in-plane and out-of-plane seismic forces during an earthquake. It is shown in this paper that out-of-plane acceleration can exceed in-plane accelerations. Therefore, it is very possible that masonry buildings begin to fail by collapse of upper story walls subject to out-of-plane accelerations.

My masonry wall loaded out-of-plane fails by forming fracture lines similar to the yield lines of a 2 way reinforced concrete slab. Of course, the failure of the masonry wall is brittle. The fracture lines of a masonry wall loaded out-of-plane form rather quickly and the applied load is reduced. As such, the masonry wall loaded out-of-plane does not seem to have enough ductility to justify the use of a seismic force reduction factor of $R_a(T_1) = 2.5$ as specified by the Turkish Earthquake Code.

Key words: Masonry structure, Out of plane loading, Seismic accelerations.

Introduction

The urban and rural population of Turkey, its economy and its future are under severe earthquake threat.

A vast majority of research efforts to develop structures that are earthquake safe is concentrated on reinforced concrete and steel structures. This is, of course, understandable because such structures house considerable economic and business activity as well as urban populations. However, in rural areas, clay and brick masonry make up the traditional housing type. Such masonry structures are vulnerable to earthquake damage just as much.

The common effort to learn and understand state-of-the-art earthquake engineering focuses on reinforced concrete and steel structures. Consequently, the engineer who is knowledgeable about the seismic behavior and design concepts of concrete and steel structures can hardly comment on the seismic behavior of masonry structures.

In a masonry structure subject to seismic action, the load-bearing masonry walls are subject to in-plane and out-of-plane inertial forces. Therefore, in order to engineer earthquake-resistant masonry structures, it becomes mandatory to understand the response of a masonry wall to seismic action.

Behavior of masonry buildings during earthquakes

In a masonry building subject to seismic action, the load-bearing walls are subject to in-plane and out-of-plane accelerations. Consider the plan of a masonry building, as given in the Turkish Earthquake Code.
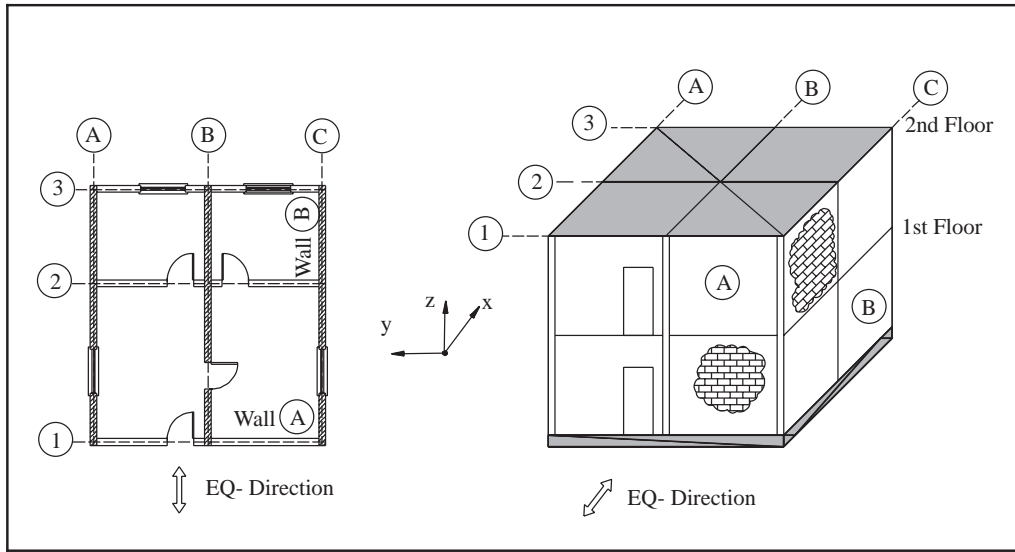
Figure 1. Two-story masonry building located in earthquake zone I.

The fundamental period in the x-direction of the building shown in Figure 1 is mainly controlled by the sway stiffness of the walls that lie on axes A, B, and C. These walls parallel to axes A, B, and C are subject to in-plane accelerations as dictated by the Acceleration Response Spectrum (Turkish Earthquake Code, 1997).

Generally, it can be safely assumed that the initial sway stiffness of masonry buildings is large, as mainly determined by the stiffnesses of walls parallel to the direction of earthquake attack. Consequently, the fundamental period of the masonry building considered will be smaller than T_B of the response spectrum. It can thus be concluded that the ground acceleration (a_g) will be subject to the greatest spectrum coefficient, $S_a(T) = 2.5$ (Figure 2), as required by the Turkish Earthquake Code 10.2.

As seen in Figure 2, for a ground acceleration of 0.4(g), the magnified maximum acceleration becomes 1.0(g) at the roof level of the masonry building considered.

Figure 3 shows the sway profile of the masonry building from base to roof level, y(t). Considering the fact that the seismic forces generated are mainly dependent on the fundamental mode, both the ac-
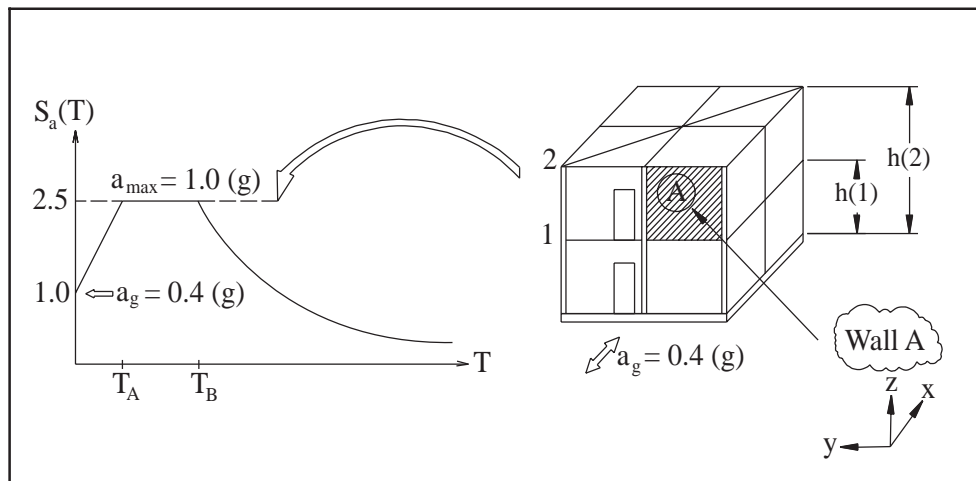

Figure 2. The response of the masonry building in compliance with the acceleration response spectrum of the Turkish earthquake code.

celeration and seismic force distribution along the height of the building resemble the sway profile (Figure 3).

$y(t)$ = sway profile
$d^2y(t)/dt^2$ = acceleration
$f_i = m_i \, a_i$ = floor seismic forces
m_i = mass of the i-th floor
a_i = acceleration of the i-th floor

Considering Earthquake Zone I where $A_0 = 0.4$, the acceleration at the center of effective mass, which is the geometric center of Wall A is

$$A_0(g) = 0.4(g) \qquad (1)$$

$$S(T) = 2.5 \qquad (2)$$

$$a = 0.4(g)(2.5) = 1.0(g) \qquad (3)$$

Consequently, a_{max} becomes $\frac{4}{3}(g)$.

As can be seen from Figure 3, the ground acceleration $A_0(g)$ is magnified from base to roof level by a multiple of 3.33. However, at ground level, the sway is zero but the acceleration is not zero. It has a value of $A_0(g)$. This ground acceleration is superposed on

the acceleration profile by assuming that the height of the effective mass is at $\frac{3}{4}H$, where H is the total height of the building. It is thus assumed that the accelerations between the base and height y_e are controlled by $A_0(g)$ (Paulay and Priestley, 1992).

The top floor (2nd floor) of the masonry building shown in Figure 3 experiences a maximum acceleration of $\frac{3}{4}(g)$.

By similar reasoning, the acceleration of the 1st floor becomes as follows (Figure 3).

$$a(\text{1st floor}) = \frac{2}{3}(g) + 0.13(g) \qquad (4)$$

In the above analysis, it is assumed that the floor slabs are infinitely rigid in their own plane. If not, the acceleration of the floors would be further magnified in the ratio of (T_f/T_s), where T_f = fundamental period of the floor diaphragm, and T_s = fundamental period of the structure, as mainly dictated by walls on axes A, B, and C (Figure 1).

Due to a ground acceleration of $A_0(g)$, the floors of the 1st and 2nd story vibrate in the y-direction. Wall (A) extends vertically between the 1st and 2nd floors and is affected by the accelerations, a(2) and a(1).
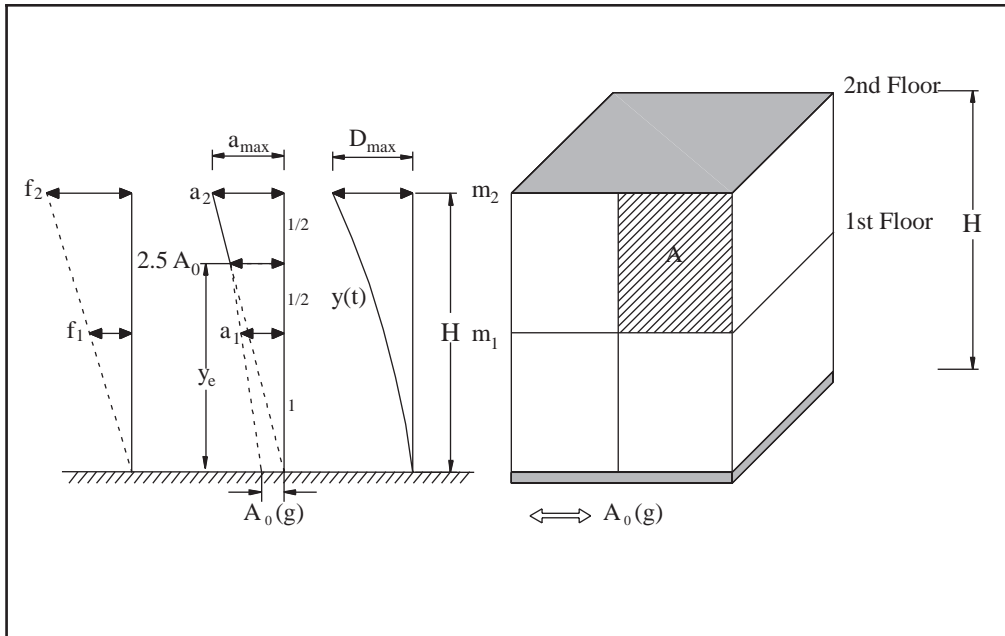

Figure 3. Dynamic response of the masonry building and the resulting acceleration and seismic forces as dictated by sway.

It can be approximated that the input acceleration of Wall (A) is the average of a(2) and a(1). Wall (A) responds to this energy input in the ratio of (T_{WA}/T_f) as shown in Figure 4, where T_{WA} = fundamental period of Wall (A) out-of-plane (like a floor slab), and T_f = fundamental period of floor diaphragm vibrating in its plane (Paulay and Priestley, 1992).

Consider Strip I and Strip II taken on Wall (A) in the vertical and horizontal directions, consequently. Wall (A) is subject to out-of-plane acceleration and this acceleration causes inertial forces to occur in the direction perpendicular to the plane of Wall (A). Of course, the out-of-plane acceleration experienced by Wall (A) is much greater than the in-plane acceleration. Paulay and Priestley (1992) suggest that this amplification be considered 2. Therefore, the out-of-plane acceleration of Wall (A) becomes as follows:

$$a(A) = \left\{ \frac{1}{2}[\frac{4}{3}(g) + \frac{2}{3}(g)] + 0.13 \right\} x2 = 2.26(g) \quad (5)$$

$$a_g = 0.4(g)$$
$$(6)$$

Ground acceleration

$$a(2) = \frac{4}{3}(g)$$
$$(7)$$

Maximum acceleration of 2nd floor

$$a(1) = \frac{2}{3}(g) + 0.13$$
$$(8)$$

Maximum acceleration of 1st floor

$$a(A) = [(a(2) + a(1))/2] \times$$

2 Maximum out-of-plane acceleration of Wall (A)
$$(9)$$

It is very interesting to note that the centroid of Wall (A) of a 2-story brick masonry building located in Earthquake Zone I subject to a maximum ground acceleration of 0.4(g) experiences an out-of-plane acceleration of 2.26(g).
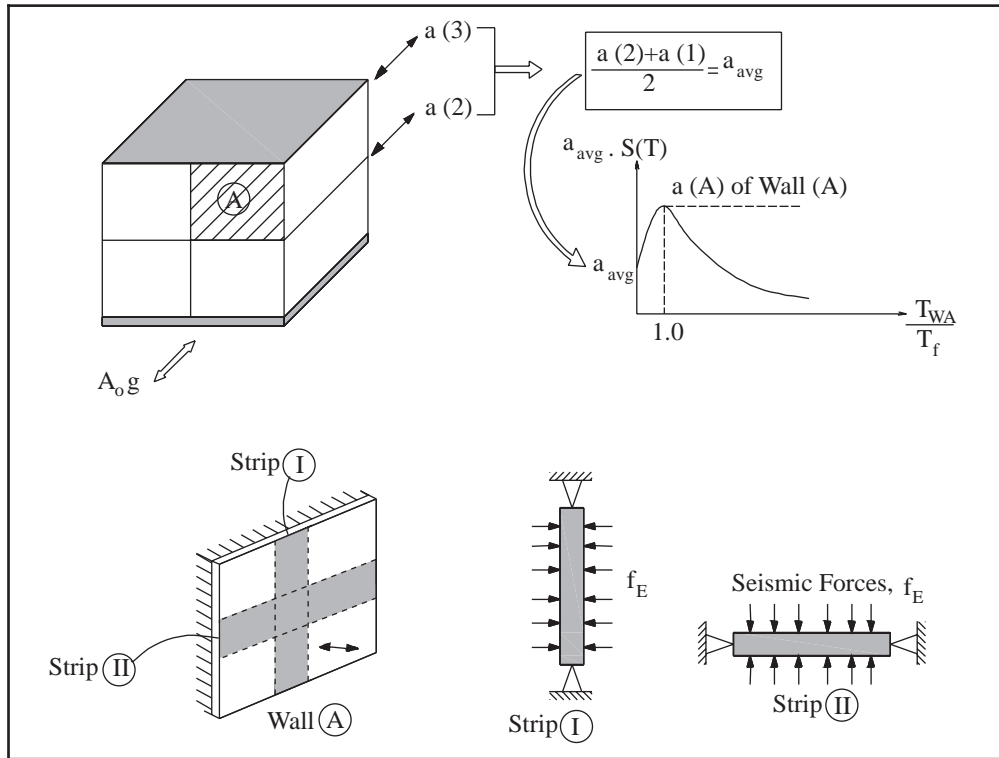

Figure 4. Dynamic response of wall (A) as affected by the ratio T_{WA}/T_f.

Figure 5. Out-of-plane failure of a 2nd story wall in the Bam Earthquake, 2003.

Therefore, it is logical to assume that masonry buildings may begin to fail initially by out-of-plane failure of walls in the upper stories. Figure 5 shows the out-of-plane failure of the second-story wall of a 2-story building in the Bam Earthquake, Iran (December 26, 2003).

Test of masonry unit loaded out-of-plane

The test specimen To understand the behavior and failure pattern of masonry walls loaded out-of-plane, the masonry unit shown in Figure 6 was tested in the Earthquake Research Laboratory of Gazi University.

Figure 7 shows the loading mechanism. The load is applied by a 2-way action ram that can apply both compression and tension. The wall has a central hole, through which a rigid rod passes. A similar loading plate exists at the "back of the wall". By a push and pull action applied on the wall, the reversing effect of the seismic forces is simulated. It is assumed that the load applied at 4 points of the loading plate represents reasonably well the moment distribution produced by uniformly distributed seismic forces resulting from the out-of-plane accelerations on the wall.

Figure 8 shows the geometric properties of the test unit. It is apparent from the geometric dimensions that the masonry test unit is a prototype with dimensions of 2.7 x 2.1 m.

Masonry coursework The test unit is mainly composed of a wall with the brick coursework shown in Figure 9.

The mortar composition used in laying the bricks is as follows (for 1 m^3) :
Sieved fine sand: 1 m^3, Cement: 0.2 t, Water: 0.2 m^3

The plaster composition is as follows (for 1 m^3):
* Rough plaster (20 mm thick):
Hydrated lime: 0.330 m^3, Sand: 1 m^3
* Fine plaster (10 mm thick):
Hydrated lime: 0.330 m^3, Sand: 1 m^3
* The slab concrete is C16.

Observations and Discussion

The wall deflections are measured at 4 edge points of the loading plate, on both the front and back plates. The average of 4 deflection measurements is assumed to be the central deflection of the wall. The load and the corresponding deflections are considered positive when the load acts in a direction that produces tension in the edge walls perpendicular to the wall tested (Figure 10).

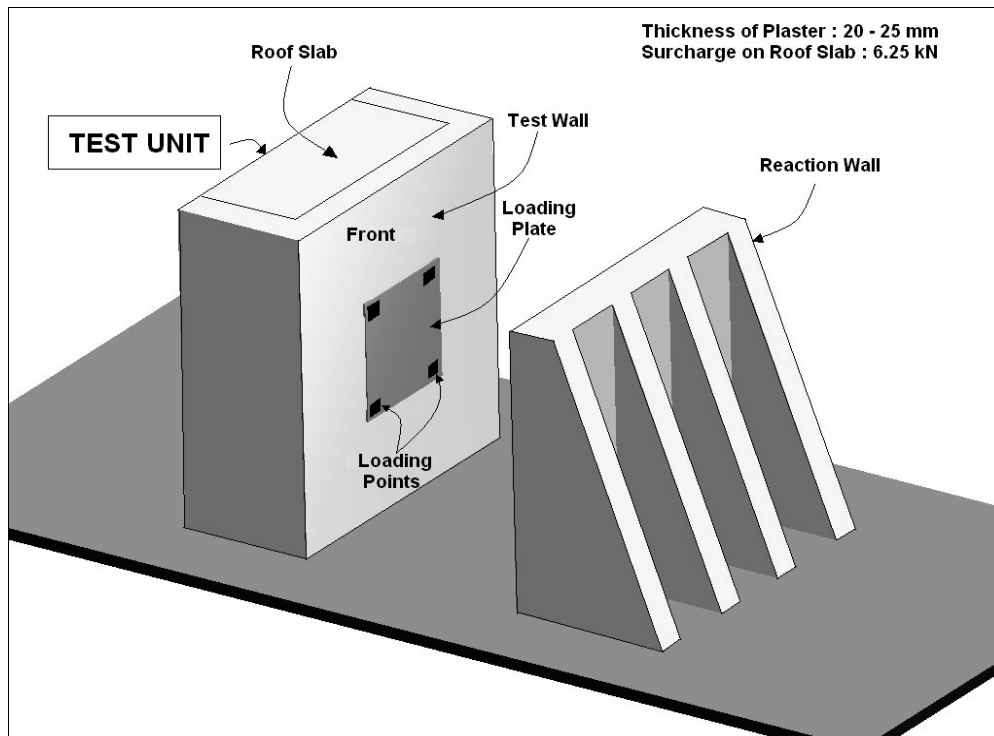The hysteretic response of the masonry test unit is shown in Figure 11.

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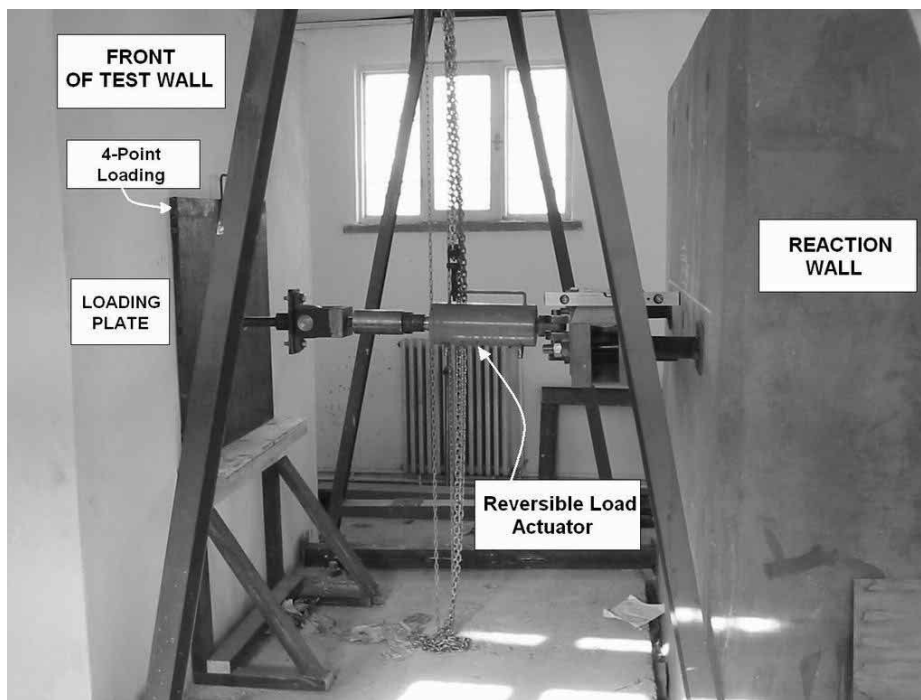**Figure 6.** Masonry test unit and the reaction wall.

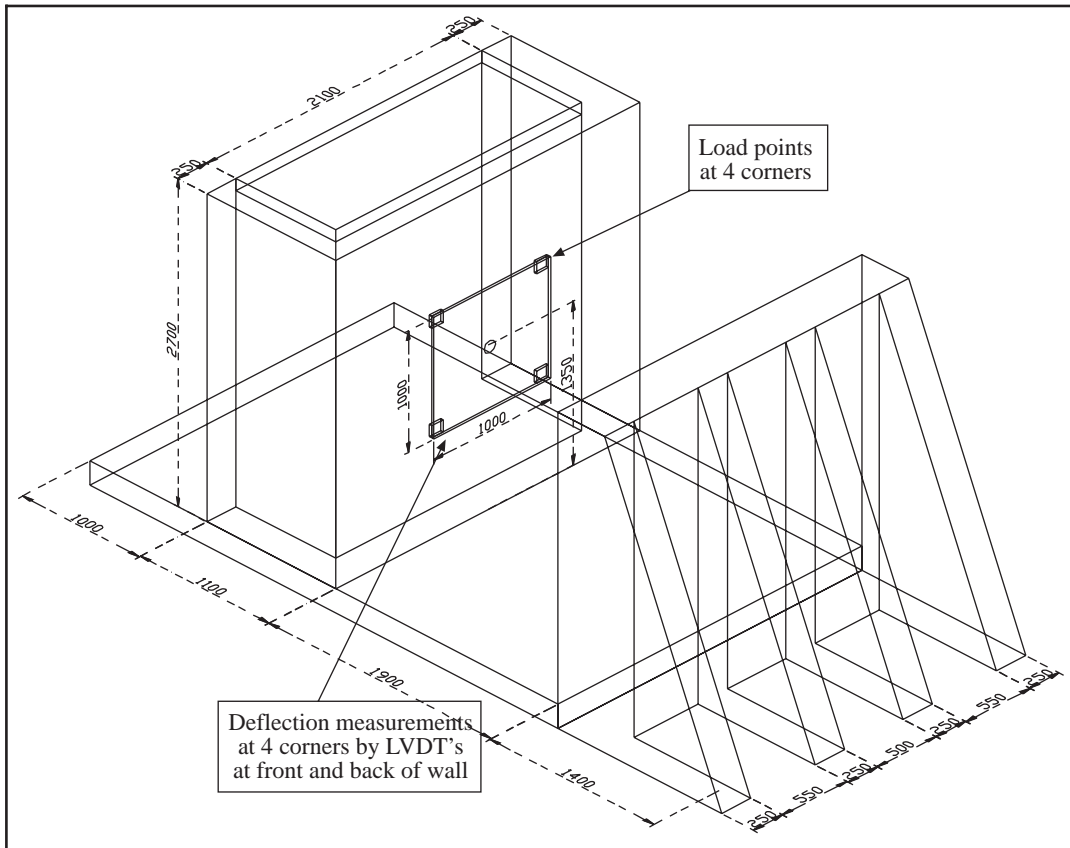**Figure 7.** The loading mechanism to simulate reversible seismic action.

Figure 8. Geometric dimensions of the masonry test unit (All dimensions in mm).
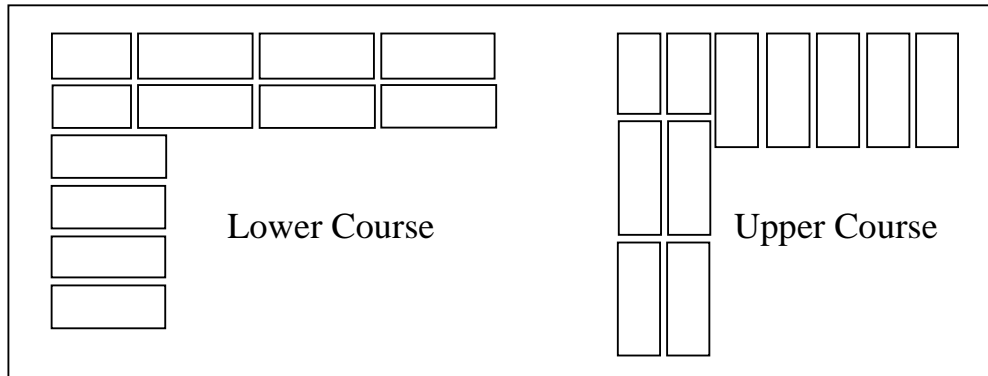

Figure 9. The brick coursework of the masonry test unit.

a. The face loaded wall presents different behavior dependent on the directions of the reversing loads.

b. Until initial cracking occurs, the face loaded wall acts elastically under load reversals.

c. Initial cracking occurs under positive direction of the load, which puts the corner supports under tension.

d. Initial cracking occurs in the middle surface of the wall where moments are expected to be the greatest at a load level of $F_{cr} = 40$ kN. After cracking, the stiffness of the wall is reduced by 50%.
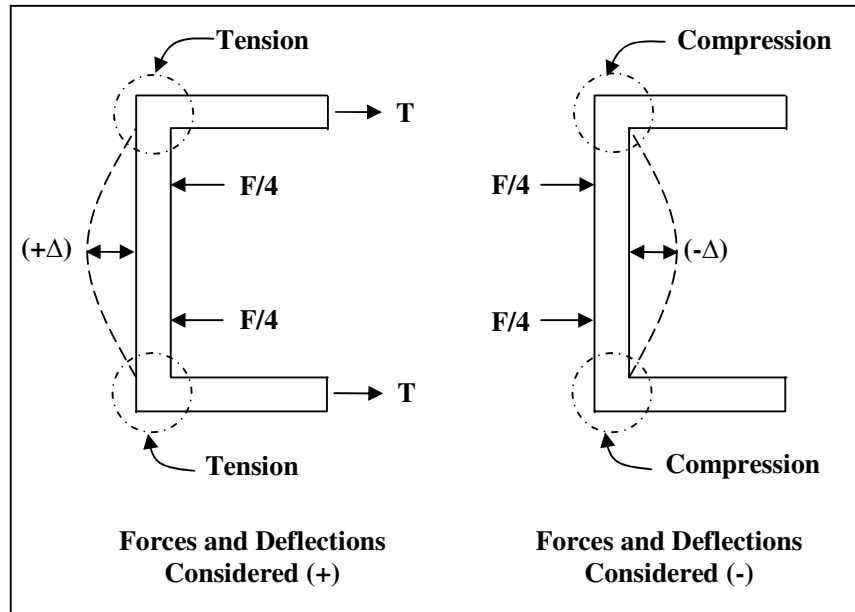
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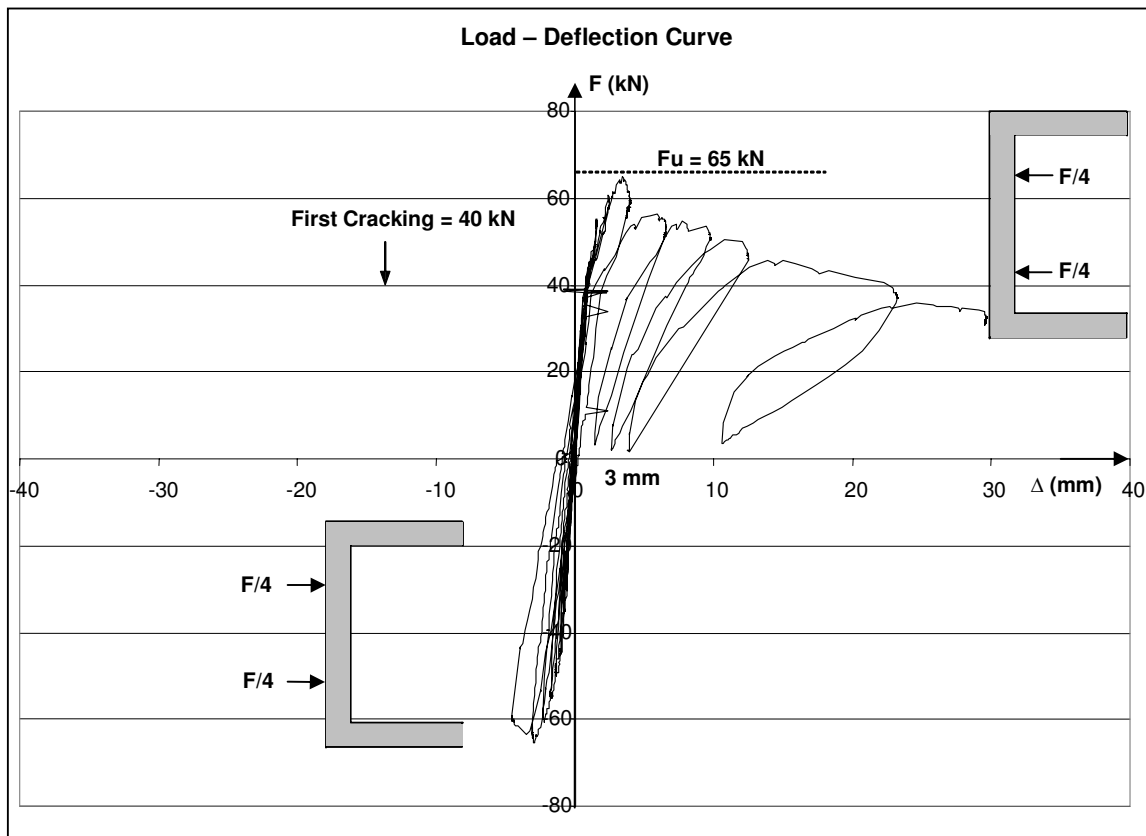**Figure 10.** Sign convention of the hysteretic response shown in Figure 11.

Figure 11. Hysteretic response of the masonry test unit loaded out-of-plane.

e. After cracking, the fundamental period of the face loaded wall is lengthened by about 1.4 times. This, of course, reduces the energy interaction between the vibrating floor diaphragms and the wall loaded out-of-plane. However, the elongation of the fundamental period, as observed, seems to be rather inadequate to terminate magnifications of out-of-plane accelerations in the face loaded wall. This judgment is based on the design response spectra of structures vibrating in conformance with the structure's fundamental mode, as given in the Uniform Building Code (1997).

f. Subject to the load producing compression at the vertical edges, the wall behaves almost elastically, dissipating very little energy. There is a slight sign of cracking at -F = 45 kN as indicated by a slightly reduced stiffness, very similar to cracking behavior under the load in the other (+F) direction (Figure 11).

g. Strength degeneration and final failure occur under the loading producing tension at the vertical edges of the wall. It is interesting to note the location and width of the crack parallel to the vertical edge line of the wall. This is definitely a tension crack separating the wall from the edge supports (Figure 12).

h. The cracking pattern of the wall loaded out-of-plane is shown in Figure 13. It is obvious that failure has occurred at the corners under tension and over

the surface of the wall. By observing this behavior, it is normal to expect that failure of the face loaded wall will occur "out from the building", not "into the building" by seismic forces acting in the direction as shown in Figure 12. Figure 14 shows a close-up view of the corner at failure.

i. With load increases above 40 kN, the cracks progressed both at the edges of the wall and in the surface of the wall. The final failure came at $F_u = 65$ kN in the wall after development of surface cracks that resemble yield lines in a rectangular reinforced concrete slab.

j. The crack pattern proves that the masonry wall loaded out-of-plane behaves very much like a solid 2-way reinforced concrete slab. Therefore, the load applied on the surface of the wall must have been transferred in both directions towards the edge supports, very much resembling the load transfer in a 2-way solid slab. It can be further concluded that the brick layout and the plaster on both faces of the wall have resulted in a continuous load transfer medium similar to that of a reinforced concrete slab (West and Hasetline, 1977; Hamoush et al., 2002).

k. After the ultimate load is reached at $F_u = 65$ kN, a very rapid degeneration in strength occurs. The wall quickly unloads, similar to the behavior observed in a shear failure of a reinforced concrete wall (Sinha et al., 1979).
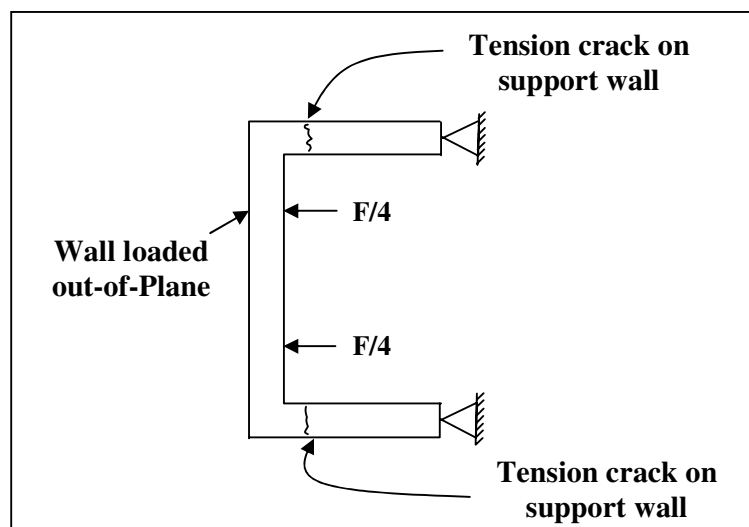

Figure 12. Tension crack at edges of slab.

Figure 13. Cracking pattern of the test wall at failure.

Figure 14. Close-up view of corner cracks at failure.

l. The masonry wall loaded out-of-plane shows a rather brittle failure, dissipating very little seismic energy. After the maximum load of $F_u = 65$ kN is reached, the consequent loading cycle goes up to only $F = 0.85F_u = 55$ kN. In the following loading cycles, the maximum load that can be reached is further decreased until failure occurs (Figure 11).

m. The test wall fails by developing a complex pattern of cracking that, in general, resembles yield lines of a reinforced concrete slab. Of course, for a brittle material like brick masonry, ductility, as implied by yield lines of a reinforced concrete slab, is not possible. The resemblance is only geometrical and in appearance. It may be more appropriate to identify these cracks as "fracture lines" (Sinha, 1978; Hendry, 1990).

n. It is not possible to pronounce a unique failure stress. It may be more appropriate to state a level of out-of-plane acceleration that will induce out-of-plane failure of the wall.

Consider the wall tested to be subject to out-of-plane accelerations. The maximum out-of-plane acceleration will produce maximum distributed inertia forces acting transversely to the plane of the wall and forcing the wall to bend like a floor slab (Figure 15).

$$f_E = m(a)$$

f_E= distributed seismic forces that act out-of-plane of the wall

m = mass per unit area of the wall

a = out-of-plane acceleration

In the mathematical model of the masonry building on which the behavior of a wall loaded out-of-plane was discussed (Figure 2) Wall A is subject to an acceleration of 2.26(g).

$$f_E = (w/g)(2.26g) \tag{10}$$

$$w = 0.25(1.0)(1.0)20kN/m^2 = 5kN/m^2 \tag{11}$$

$$f_E = (5/g)(2.26g) = 5(2.26) = 11.3kN/m^2 \tag{12}$$

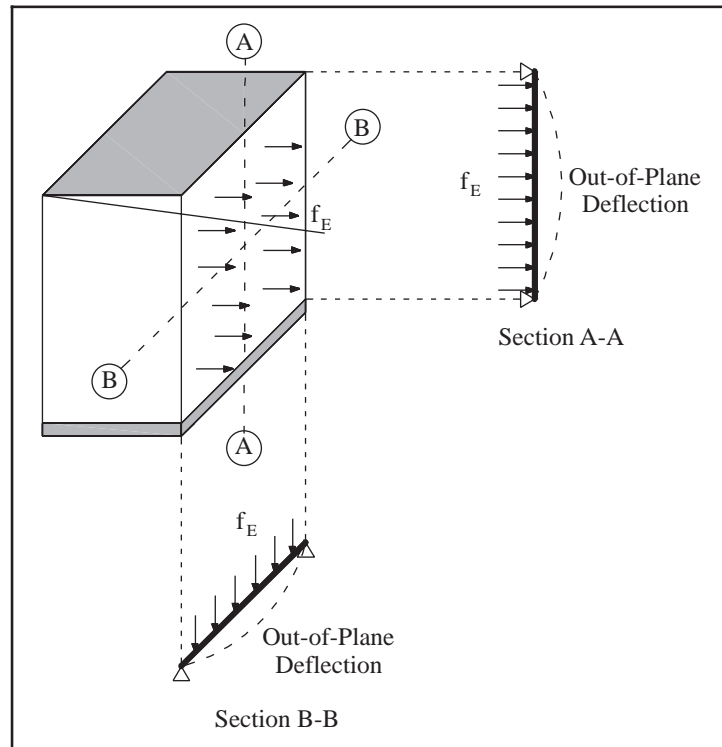The total load that acts on the wall of dimensions 2.7 x 2.1 m is:

Figure 15. Out-of-plane deflection of wall subject to inertia forces.

$$F_E = (11.3)(2.7)(2.1) = 64.1 kN \qquad (13)$$

o. Will the wall fail under this total load of F_E = 64.1 kN acting out-of-plane?

This answer can be given by comparing the above calculated transverse load demand with the test load.

$$F_E = 64.1 kN > F_{cr} = 40 kN \qquad (14)$$

F_E = total inertia force acting on the wall as calculated

F_{cr} = total load applied on the wall at which cracking occurs

The wall under consideration will show vertical cracking at the edges and also some cracking in the surface of the wall.

$$F_E = 64.1 kN < F_m = 65 kN \qquad (15)$$

The wall under consideration will not fail and will still continue to resist the pulsating inertial forces. After cracking, the wall is less stiff and the maximum response acceleration cannot sustain its maximum level of 2.26(g). Thus, the acceleration will decrease

and the risk of failure will be less. Of course, depending on the quality of workmanship and the lack of plaster, local failure of the wall may occur (Figure 16). It should be noted that the masonry wall shown in Figure 16 is composed of hollow bricks.

p. At final failure of the wall tested, the equivalent distributed out-of-plane load can be calculated as follows:

$$F_u = 65 kN \qquad (16)$$

$$f_E = 65/(2.1 \times 2.7) = 11.46 kN/m^2 \qquad (17)$$

This equivalent distributed load corresponds to ground acceleration as given below.

$$a_g = 0.4g(11.46/11.3) = 0.406g \qquad (18)$$

The test wall will fail at a ground acceleration of 0.406(g), which is slightly greater than 0.4(g). Therefore, this test wall may be considered just barely safe subject to the ground accelerations of 0.4(g) in Earthquake Zone I.

Figure 16. Out-of-plane failure of an infill wall.

q. The Turkish Earthquake Code (1997) permits the elastic seismic forces to be reduced by a factor of R_a $(T_1) = 2.5$. This reduction, of course, assumes the realization of a certain degree of inelastic action and ductility. However, the hysteretic response of the test unit does not indicate acceptable levels of inelastic behavior and ductility. The maximum load unloads rather quickly and continuously. As observed by this test and the geometrical conditions of the test unit, the use of a load reduction in the order of 2.5, as given in the Turkish Earthquake Code (1997), is questionable. It is possible that this reduction factor indicates the inelastic action and ductility that can occur in a wall loaded in-plane, but not a wall loaded out-of-plane.

Conclusions

1. The failure of masonry walls subject to out-of-plane accelerations makes load-bearing masonry buildings very vulnerable to earthquake damage and possibly collapse.

2. Due to the dynamic interaction between the vibrating structure, slab diaphragms and the wall loaded out-of-plane, greatly increased accelerations occur on the face loaded wall, resulting in greatly increased inertia forces. These distributed forces make the wall fail, forming fracture lines on the surface of the wall similar to the failure of a 2-way reinforced concrete slab.

3. The existence of well composed and applied plaster helps to produce a continuous medium of load transfer. Therefore, to enhance the earthquake safety of masonry walls, plaster on both faces of the wall should not be omitted.

4. The failure of the wall under reversing inertia forces occurs in the direction that puts the edge supports of the wall under tension. When failure of the wall in this direction occurs, the same wall loaded in the reverse direction still behaves rather elastically.

5. The out-of-plane failure of the test wall is brittle, resembling a shear failure. Therefore, it dissipates very little seismic energy. As such, the face loaded wall is very vulnerable to seismic failure.

6. The Turkish Earthquake Code (1997), 10.2.1, permits the reduction of elastic seismic forces by a factor of R_a $(T_1) = 2.5$. The hysteretic behavior of the test wall, as shown in Figure 16, does not have enough ductility to justify a seismic force reduction of R_a $(T_1) = 2.5$.

7. The strength of the wall tested slightly exceeds the seismic demand imposed by an earthquake with a ground acceleration of $0.4(g)$. However, this is true for the specific dimensional and material properties of the test wall. Other geometric dimensions including door and window openings and other material combinations must also be investigated.

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