Scientific Research and Essays Vol. 5(21), pp. 3336-3344, 4 November, 2010
Available online at http://www.academicjournals.org/SRE
ISSN 1992-2248 ©2010 Academic Journals

Full Length Research Paper

Assessing the experimental behaviour of load bearing masonry walls subjected to out-of-plane loading

Recep Kanit, Mürsel Erdal* and Ömer Can

Faculty of Technical Education, Construction Department, Gazi University, 06500, Teknikokullar, Ankara, Turkey.

Accepted 7 October, 2010

An important factor to consider is the resistance of load bearing masonry walls subjected to out-of-plane loading during an earthquake. In addition to in-plane forces, earthquake accelerations produce out-of-plane forces which may cause the wall to fail under flexure, very similar to a reinforced concrete slab, loaded perpendicular to its plane. Four wall specimens were tested under out-of-plane hysteretic loads to assess its behaviour. The paper also describes different measures taken to improve both the strength and the ductility of the original specimen. It appears from the tests that the proposed measures improve substantially the strength, but ductility was only slightly improved.

Key words: Load bearing, masonry walls, earthquake, out-of-plane failure, ductility.

INTRODUCTION

Majority of housing stocks all over the world including those in seismic zones consist of load-bearing masonry. Therefore, it becomes of utmost importance to understand and assess the seismic behaviour of load bearing masonry. However, load bearing masonry structures do not get the attention, which they so rightfully deserve, from the ongoing seismic research. A majority of the ongoing seismic research is concentrated on reinforced concrete or steel structures.

A very unique behaviour occurs in a masonry building subjected to seismic action. The load bearing masonry wall is subjected to out-of-plane forces, in addition to in-plane forces. Therefore, depending on the dominating forces, the load bearing masonry wall may fail due to out-of-plane loading exhibiting a flexural failure mode.

It is, therefore, important to assess the out-of-plane failure mode of a load bearing masonry wall. Many tests have been done and the in-plane failure behaviour of load bearing masonry walls is rather well determined (Hendry, 1990). Also, the strength and behaviour of masonry subjected to out-of-plane static loading is well researched and documented (Paulay and Priestley, 1992). Very little is known about the behaviour of masonry walls subjected to out-of-plane cyclic loading, hence concentrated research efforts are needed to define it.

Among the techniques available today for the strengthening of URM walls, the use of externally bonded fiber reinforced polymer (FRP) has been designated as an attractive structural solution (Triantafillou, 1998; Hamoush et al., 2001). Experimental investigations showed that the use of the externally bonded FRP laminates leads to an increase of up to 50 times in the strength of the masonry wall (Gilstrap and Dolan, 1998). Along with the improved stiffness, strength, and ductility, the experimental studies also revealed a broad range of physical phenomena that characterize the behaviour of the strengthened wall (Hamilton and Dolan, 2001; Albert et al., 2001; Kiss et al., 2002; Tumialan et al., 2003; Buyukozturk et al., 2004; Hamed and Rabinovitch, 2007, Galal and Sasanian, 2010).

Similar studies were conducted by other researchers like Hamoush et al. (2002), Turco et al. (2006), El-Dakhakhni et al. (2006); Mosallam (2007), Papanicolaou et al. (2008), Kaplan et al. (2008), Erdal (2010), Korkmaz et al. (2010). They studied the out-of-plane behaviour of FRP strengthened masonry walls with openings.

Recently, Korany and Drysdale (2006) developed an

*Corresponding author. E-mail: merdal@gazi.edu.tr. Tel: +90 312 2028870. Fax: +90 312 2120059.

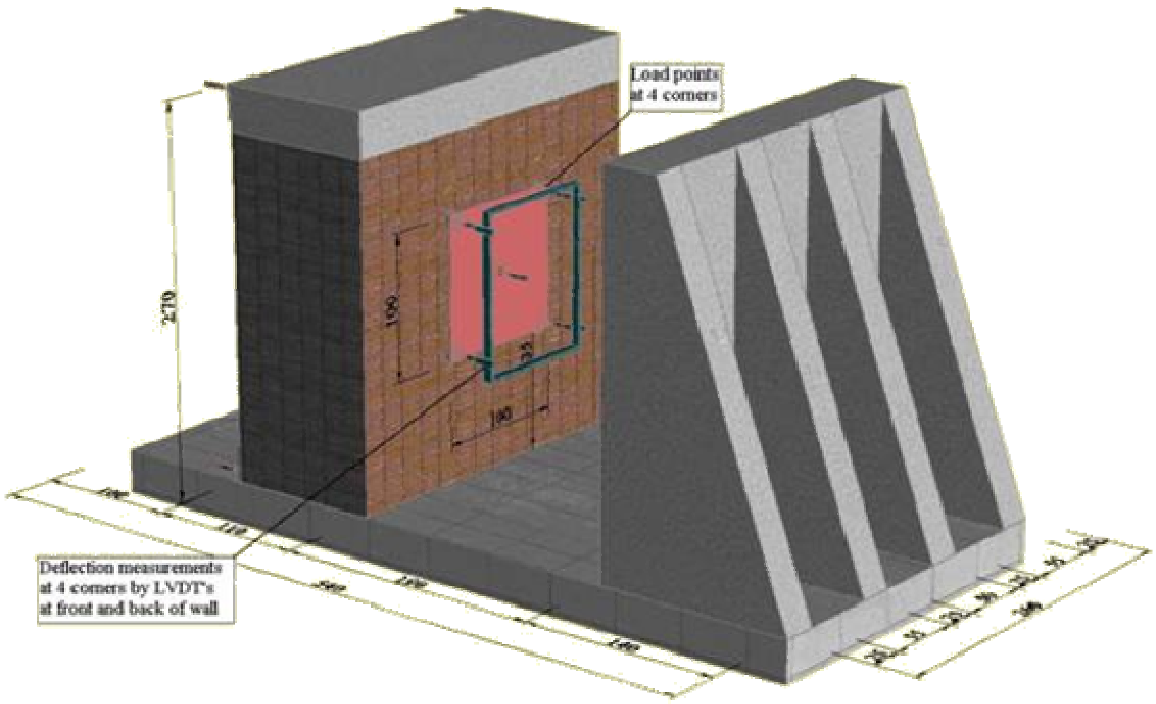

Figure 1. The test specimen and reaction wall.

unobtrusive composite rehabilitation technique using flexible carbon/epoxy cables, mounted near the surface of the façade walls in epoxy-filled grooves in the bed and head joints. Kanit and Donduren (2010) modelled masonry walls with similar geometrical properties using Ansys software and they compared numerical results with experimental results. They showed that numerical results and the experimental data results were close to each other.

EXPERIMENTAL PROGRAMME

In order to answer the questions mentioned above, an experimental programme was planned in the laboratory of the Earthquake Research Centre of Gazi University. The aim of this experimental programme was to understand the failure behaviour of load bearing masonry walls subjected to out-of-plane loading and search for methods of improving this behaviour.

In the construction of test specimens, bricks conforming to Turkish Standard TS 771-1 (2005) were used, with following properties:

Compressive strength : 23.17 MPa
Tensile strength : 2.61 MPa

The test wall specimens were one brick thick, having 20 mm rough plaster and 10 mm fine plaster, on both faces.

The testing setup and the dimensions of the test specimen are shown in Figure 1 (Kanit and Atimtay, 2006).

Evaluation of test result

It is well known that under seismic forces acting in-plane of the structural elements, the most vulnerable region of damage is at the foundation level. Contrary to this condition, for seismic forces acting out-of-plane, the most vulnerable region of damage is at the top of the structure. This is because of the linearly increasing acceleration, from ground level to the roof (Paulay and Priestley, 1992).

A 4-storey load bearing masonry building is considered. The number of storey is assumed to be a maximum under normal conditions in practice. Therefore, maximum out-of-plane effects are expected in such a building, Figure 2.

It is assumed that the triangular distribution of acceleration results in a triangular distribution of in-plane seismic loads and the centre of effective mass are $2/3(H_N)$ from the foundation level, H_N being the total height of the building.

The out-of-plane acceleration acting on wall between the roof level and 3rd floor level can be considered to be the average of corresponding accelerations. Therefore, the out-of-plane acceleration acting on wall at roof level becomes $a_r = 1.31g$ due to a magnification of 2.5 times of the maximum ground acceleration of $A_0 = 0.4g$. As a result, the maximum seismic force, acting out-of-plane on

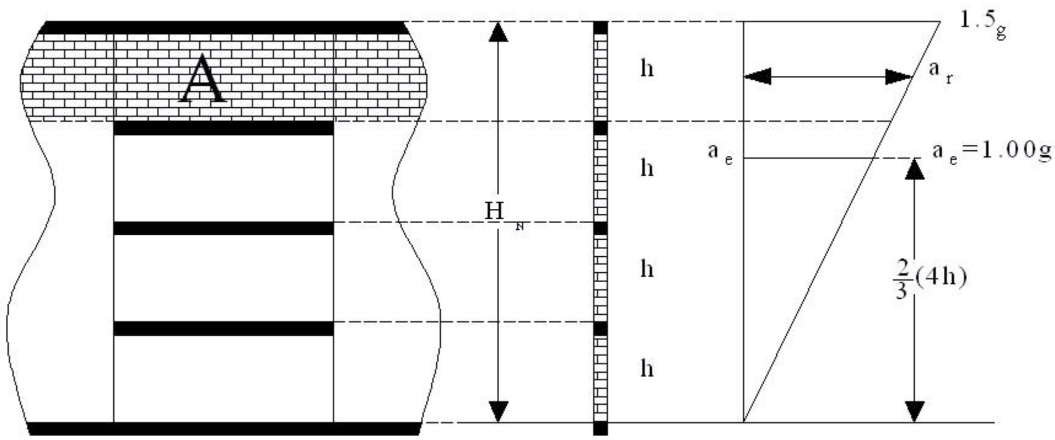

Figure 2. Maximum out-of-plane acceleration acting on the wall at roof level of a 4-storey building.

a wall at roof level of a 4-storey masonry building will be $F_E = 0.26 \times 3 \times 3 \times 20 \times 1.31 = 61.3kN$, assuming a wall thickness of 0.26 m, plan dimensions of $3 \times 3m$ and a unit weight of $20kN/m^3$. This is the elastic seismic force acting on a wall of one-brick thickness having a unit weight of $20kN/m^3$. Acceleration a_r interacts with the floor accelerations at the bottom and top of the wall, in the same way ground accelerations interact with the structure. As a result, the average acceleration on the wall is magnified. According to Paulay and Priestley (1992), this magnification can be taken as 2.0, after reviewing inelastic analyses data. If the wall can elastically resist an out-of-plane load of $61.3 \times 2 = 122.4kN$, use of the Structural Behaviour Factor of $R = 1$ will be possible. Otherwise, $R > 1$ is necessary, which in turn, necessitates the corresponding ductility to be applied.

Wall A

Wall A is a comparison specimen with no modification, other than the original properties. It was tested under out-of-plane loads in reversing character. A thick loading plate was used to apply loads on the wall at four points, to simulate a moment distribution similar to those produced by a uniformly distributed load acting out-of-plane on the wall. The same thick plate exists on the other side of the wall to apply the reversed loading. The hysteretic load-deflection behaviour of Specimen A is shown in Figure 3.

The following observations can be made from Figure 3 in relation to the original test Specimen A subjected to out-of-plane loading. The test wall behaves quite elastically up to the cracking load. The initial tensile cracking of the wall occurs at a load of $F_{cr} = 40kN$. The stiffness of Wall A, before initial cracking, is about $S = 40kN/mm$. The maximum load is reached at $F_m = 65kN$. The stiffness of Wall A, at maximum load, is about $S = 16.25kN/mm$. The stiffness of Wall A is reduced by 59% after initial cracking. Very little seismic energy dissipation occurs until maximum load is reached.

After maximum load is reached, there is a gradual and long unloading behaviour of the applied load. This indicates a very effective dissipation of seismic energy. If satisfactory and acceptable ductility is accepted as the point defined by 0.85 F_m, F_m being the maximum load, the deflection corresponding to the end of ductility becomes 6 mm. At maximum load F_m, the corresponding deflection is 3.5 mm. Then, according to Equal Deflection Principle, the R-factor obtained becomes $R = 122.4/65 = 1.88$. This is less than the R-factor used in the Turkish Earthquake Code (2007).

At this R-value, the required ductility becomes $\mu_A = 1.88$. The test Wall A presents a ductility ratio μ_A of $\mu_A = 4mm/3mm = 1.33$, where 3 mm corresponds to the yield deflection and 4 mm corresponds to $0.85F_m$. As such, the ductility requirement of the Turkish Earthquake Code (2007) is not satisfied. The cracks keep on increasing as the hysteretic cycles are repeated. At the final stage, the cracks resemble those that form in a two-way reinforced concrete slab. At ultimate stage, $F_u = 30kN$ is still carried by the masonry wall loaded out-of-plane. The appearance of cracking at ultimate stage is shown in Figure 4.

As can be observed from Figure 4, failure has occurred in the direction “away from the room”, producing tension forces along the vertical corners. In the L-wall, a crack parallel to the corner has formed, which gives freedom to rotate to the vertical joint. This crack makes the masonry wall, “simply supported” at the sides.

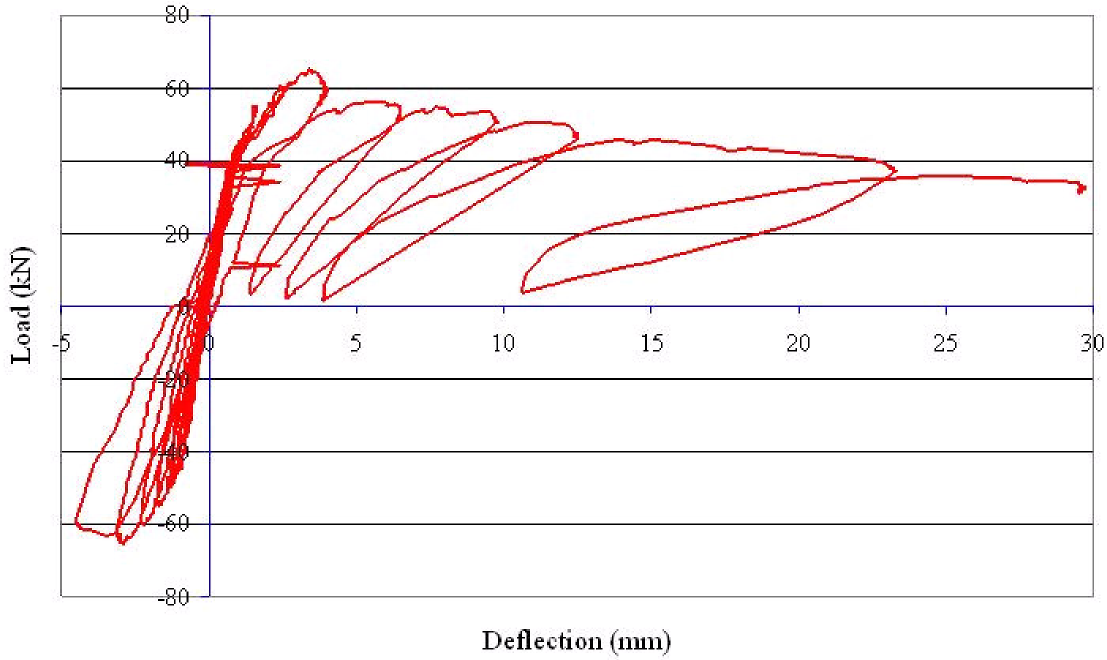

Figure 3. Hysteretic load-deflection behaviour of Wall A.

Figure 4. The cracking pattern of Wall A at ultimate stage.

It may be considered that, one big reason which causes the cracking map to resemble that of a two-way reinforced concrete slab is the existence of the 30 mm thick plaster, on both faces of the wall. This plaster has turned the otherwise discrete masonry wall into a continuous slab (Hendry, 1990). This point has been proved by Hendry, but it was interesting to observe this behaviour again and under cyclic loading.

Wall B

After observing the behaviour and reviewing the data of the masonry wall, hysterically loaded out-of-plane, ways to improve the seismic performance of Wall A were sought, especially the ductility.

It was considered that the occurrence of the tension crack parallel to the vertical corners was an important factor, in relaxing the edge restraints of the wall. It was decided to take measures to eliminate this crack. L-shaped plates with vertical dowels for anchorage were manufactured and placed in the corners at 500 mm spacing along the height of the wall. Everything else was kept exactly as Wall A. The application of L-shaped corner plates is shown in Figure 5.

The Wall B was also tested hysterically until ultimate stage. The hysteresis behaviour of Wall B is shown in Figure 6. Initial tension cracking occurs at $F_{cr} = 20kN$. This is lower than the cracking load of Wall A. This means that the cracking load which depends on the

Figure 5. The applications of L-shaped corners bars.

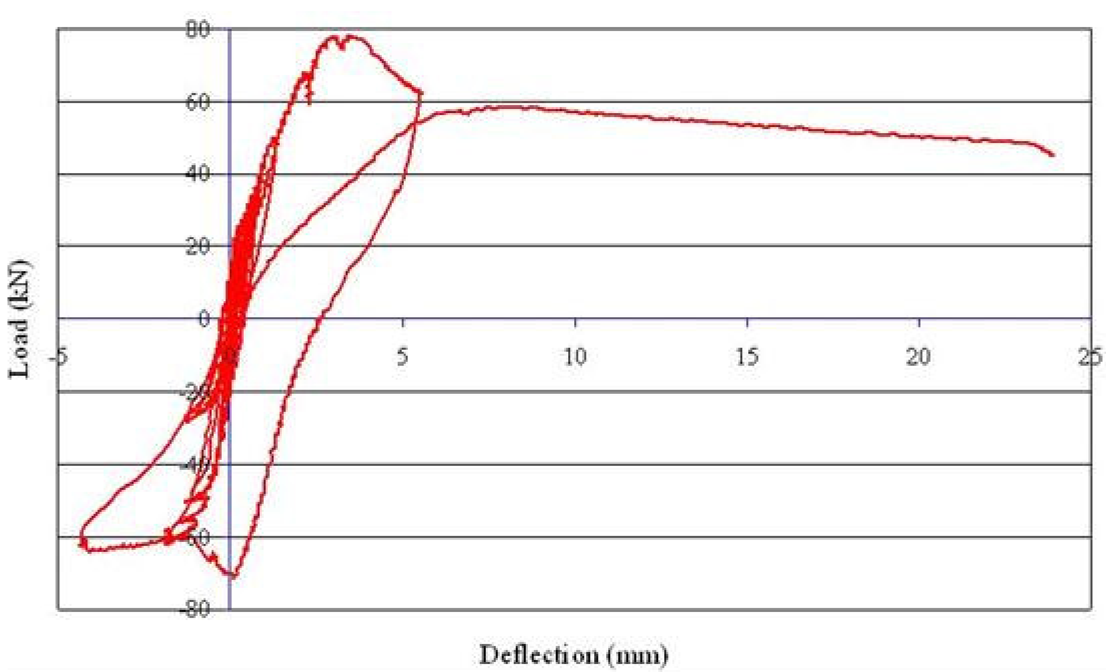

Figure 6. Hysteretic load-deflection behaviour of Wall B.

tensile strength of the brick and the plaster is very variable. Therefore, the initial occurrence of the tension crack is also variable.

As in the comparison specimen of Wall A, Wall B acts quite elastically until cracking. The stiffness that corresponds to cracking $S = 20/0.5 = 40kN/mm$. This is 2.46 times greater that of Wall A. However, because the tensile strength of the brick and the plaster is variable, the initiation of the first tension crack is not of much structural importance, because initial tension cracking of the wall has little affect on the behaviour of the out of plane behaviour. There is still a lot of reserved strength until failure occurs.

The maximum load is reached at a magnitude of

Figure 7. The cracking pattern of Wall B corresponding to ultimate stage.

Figure 8. The application of L-shaped reinforcing plates from outside of the wall.

$F_m = 75kN$ corresponding to a central deflection of 3.5 mm. Consequently, the stiffness of Wall B corresponding to maximum load stage becomes $S_m = 78/3.5 = 22.3kN/mm$.

The R-factor obtained from the test, by applying the Equal Energy Principle, becomes $R = 122.4/75 = 1.43$ which satisfies the Turkish Earthquake Code (2007). After the maximum load is reached, a very sharp drop in the load has occurred. Only, a small plateau has formed which can hardly be called ductility. Again, applying the $0.85F_m$ rule, as a measure of satisfactory and acceptable ductility, $F = 63.75kN$ and the corresponding central deflection of 5 mm is found. Consequently, applying the Equal Displacement Principle, a displacement ductility factor of $\mu_A = 5/3 = 1.67$ is found. This is about the same what the Turkish Earthquake Code would require.

As in Wall A, Wall B also shows ductile behaviour, even though it may be considered “unsatisfactory”. The cracking map corresponding to ultimate stage is shown in Figure 7.

Observations on Figure 7 reveal that at ultimate stage, the width of cracks is not as large as those of Wall A. The vertical cracks parallel to the sides of the wall have not occurred. This is considered as proof that the L-shaped plates that have been used during construction have strengthened the corners, providing rotational rigidity.

Wall C

It was decided to apply the L-shaped reinforcing plates from outside of the wall (Figure 8). L-shaped plates were applied from both sides and each of these plates was bounded using six steel bolts. The method could be used as a strengthening measure to existing buildings. Two purposes were considered in doing this. One purpose was to reinforce the corners more effectively by moving the position of the L-shaped reinforcing plate to the extremity of rotation. The second purpose was to create the possibility of using this technique as a repair and strengthening method of existing buildings.

Wall C was also the same and tested the same as Wall A. The hysteretic behaviour of Wall C is shown in Figure 9. Wall C has acted almost elastically until maximum load of $F_m = 65kN$. Initiation of tension cracking and the corresponding reduction of stiffness due to cracking have not occurred. The maximum load has been reached at a value of $F_m = 65kN$ and a deflection of 1.25 mm. Consequently, the stiffness of Wall C at maximum load stage is $S = 65/1.25 = 52kN/mm$.

To determine the amount of ductility, the deflection corresponding to $0.85F_m = 55.25kN$ is 2.5 mm. Applying the Equal Displacement Principle, R-factor becomes $R = 122.4/65 = 1.88$. This R-factor is less than what the Turkish Earthquake Code (2007) would state and therefore, acceptable. The cracking map corresponding to ultimate stage is shown in Figure 10.

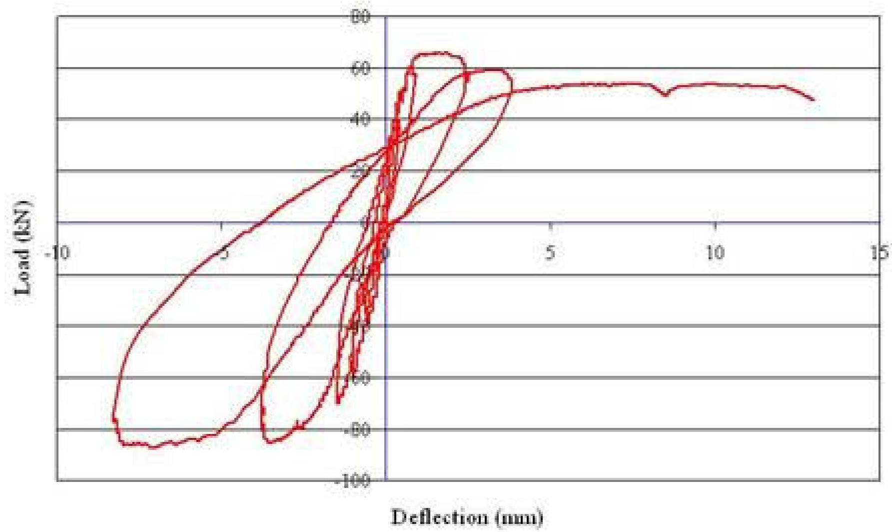

Figure 9. Hysteretic load-deflection behaviour of Wall C.

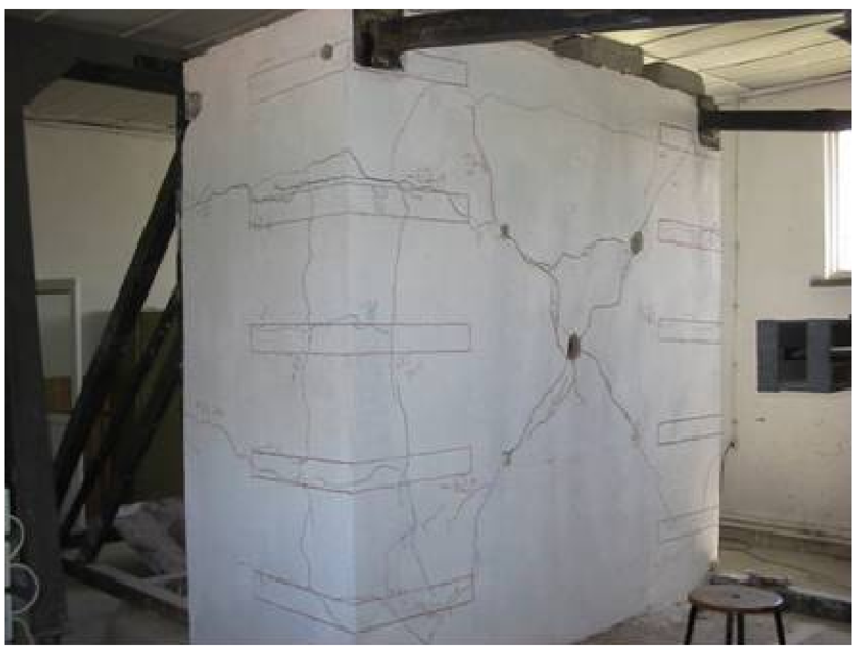

Figure 10. The cracking pattern of Wall C corresponding to ultimate stage.

The deflection ductility ratio required for an R-factor of 1.88 becomes $\mu_A = 2.5/1.25 = 2.0$ which is greater than 1.88. So, Wall C meets the ductility requirement of the Turkish Earthquake Code (2007).

As can be seen from Figure 9, the cracking map similar to that of a two-way reinforced concrete slab has formed, as also shown by Hendry (1990). The cracks parallel to the vertical corners have again appeared. However, these cracks are cut by the applied L-shaped and probably restrained, as well.

In Wall C, a long ductile behaviour occurs, beginning from the maximum load to the ultimate stage. The slope of unloading is rather gradual.

Wall D

To strengthen and improve the seismic behaviour of

Figure 11. The masonry wall strengthened by FRP-sheet at centre: Wall D.

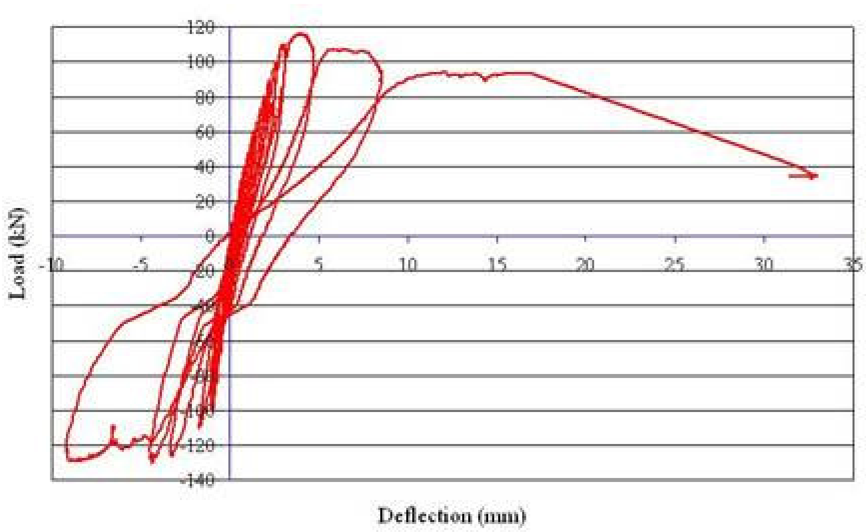

Figure 12. Hysteretic load-deflection behaviour of Wall D.

masonry walls of existing buildings, for out-of-plane action, a fibre reinforced plastic sheet (FRP) which has a width of 300 mm was applied exactly at mid-centre of the wall, from the top of wall to the bottom. The FRP sheet was bolted to the masonry wall at third point at both faces of the wall, at the front face and the back face, Figure 11.

Wall D was also tested, exactly the same way as Wall A. The resulting hysteretic load-central deflection relationship is shown in Figure 12. It can be observed from Figure 10 that Wall D acts almost elastically until maximum load $F_m = 115kN$ is reached, at a corresponding deflection of 4 mm. No initial tension cracking and the corresponding reduction in stiffness occur. Maximum load corresponding to a deflection of 4 mm begins to unload almost immediately but very slowly. The ductility definition load of $0.85F_m = 97.8kN$ corresponding to a deflection of 8 mm. As such, the R-factor becomes

$R = 122.4/115 = 1.06$ which satisfies the Turkish Earthquake Code. The ductile factor provided is $\mu_A = 8.5/4 = 2.125$, which is well above what is required. Unloading is quite gradual until 15 mm of central deflection is reached. Then, very rapid unloading begins and the FRP-sheet ruptures.

Conclusions

According to the observations from experiments following results were obtained: 1. The out-of-plane behaviour of load bearing masonry walls is quite brittle. 2. Under out-of-plane hysteretic loading, masonry walls fail “into the room” not “out to the street”. Such failure can be seen in tests as (-) loading direction, whereas the final failure came under the (+) loading direction, in all test walls. 3. By strengthening and behaviour improvement measures, it was possible to increase the strength of walls against out-of-plane action, but not ductility to any extent. 4. All walls show a rather ductile unloading behaviour. 5. The best result was obtained by a FRP application.

ACKNOWLEDGEMENT

This study was conducted with financial support from the State Planning Organization of Turkey with grant number BAP-08-11-DPT.2004K120760. Authors would like to express their sincere thanks to State Planning Organization staff.

REFERENCES

Albert ML, Elwi AE, Cheng JJR (2001). Strengthening of unreinforced masonry walls using FRPs. J. Compos. Constr., 5(2): 76-84.

Buyukozturk O, Gunes O, Karaca E (2004). Progress on understanding debonding problems in reinforced concrete and steel members strengthened with FRP composites. Constr. Build. Mater., 18(1): 1-19.

El-Dakhakhni WW, Hamid AA, Hakam ZHR, Elgaaly M (2006). Hazard mitigation and strengthening of unreinforced masonry walls using composites. Compos. Struct., 73(4): 458-477.

Erdal M (2010). Improving out-of-plane strength and ductility of unreinforced masonry walls in low-rise buildings by centrally applied FRP strip. Int. J. Phys. Sci., 5(2): 116-131.

Galal K, Sasanian N (2010). Out-of-Plane Flexural Performance of GFRP-Reinforced Masonry Walls. J. Compos. Constr., 14(2): 162-174.

Gilstrap JM, Dolan CW (1998). Out-of-plane bending of FRP-reinforced masonry walls. Compos. Sci. Technol., 58(8): 1277-1284.

Hamed E, Rabinovitch O (2007). Out-of-plane behavior of unreinforced masonry walls strengthened with FRP strips. Compos. Sci. Technol., 67(3-4): 489-500.

Hamoush SA, McGinley MW, Mlakar P, Scott D, Murray K (2001). Out-of-plane strengthening of masonry walls with reinforced composites. J. Compos. Constr., 5(3): 139-145.

Hamoush SA, McGinley MW, Mlakar P, Terro MJ (2002). Out-of-plane behavior of surface-reinforced masonry walls. Constr. Build. Mater., 16(6): 341-351.

Hamilton HR, Dolan CW (2001). Flexural capacity of glass FRP strengthened concrete masonry walls. J. Compos. Constr., 5(3): 170-178.

Hendry AW (1990). Structural Masonry. MacMillan Education Ltd. Hong Kong.

Kanit R, Atimtay E (2006). Experimental assessment of the seismic behavior of load-bearing masonry walls loaded out-of-plane. Turk. J. Eng. Env. Sci., 30: 101-113.

Kanit R, Donduren MS (2010). Investigation of using ansys software in the determination of stress behaviors of masonry walls under out-of-plane cycling load. Int. J. Phys. Sci., 5(2): 97-108.

Kaplan H, Yilmaz S, Akyol E, Sen G, Tama YS, Cetinkaya N, Nohutcu H, Binici H, Atimtay E, Sarisin A (2008). 29 October 2007, Cameli earthquake and structural damages at unreinforced masonry buildings. Nat. Hazard Earth. Sys. Sci., 8(4): 919-926.

Kiss RM, Kollar LP, Jai J, Krawinkler H (2002). Masonry strengthened with FRP subjected to combined bending and compression, Part II: Test results and model predictions. J. Compos. Mater., 36(9): 1049-1063.

Korany Y, Drysdale R (2006). Rehabilitation of masonry walls using unobtrusive FRP techniques for enhanced out-of-plane seismic resistance. J. Compos. Constr., 10(3): 213-222.

Korkmaz, HH, Korkmaz, SZ, Donduren MS (2010). Earthquake hazard and damage on traditional rural structures in Turkey. Nat. Hazard Earth. Sys. Sci., 10(3): 605-622.

Mosallam AS (2007). Out-of-plane flexural behavior of unreinforced red brick walls strengthened with FRP composites Compos. Part: B, 38(5-6): 559-574.

Papanicolaou CG, Triantafillou TC, Papathanasiou M, Karlos K (2008). Textile reinforced mortar (TRM) versus FRP as strengthening material of URM walls: out-of-plane cyclic loading. Mater. Struct., 41(1): 143-157.

Paulay T, Priestley MJN (1992). Seismic design of reinforced concrete and masonry buildings, John Wiley and Sons Inc. New York. USA.

Triantafillou TC (1998). Strengthening of masonry structures using epoxy bonded FRP laminates. J. Compos. Constr., 2(2): 96-104.

TS EN 771-1 (2005). Specification for masonry units-Part 1: Clay masonry units, Turkish Standards Institution, Ankara, Turkey.

Tumialan JG, Galati N, Nanni A (2003). Fiber-reinforced polymer strengthening of unreinforced masonry walls subjected to out-of-plane loads. ACI Struct. J., 100(3): 321-329.

Turco V, Secondin S, Morbin A, Valluzzi MR, Modena C (2006). Flexural and shear strengthening of un-reinforced masonry with FRP bars. Compos. Sci. Technol., 66(2): 289-296.

Turkish Earthquake Code (2007). General Directorate of Disaster Affairs Earthquake Research Department, Ankara, Turkey.