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Review Article:

In plane Shear Strengthening of Clay Masonry Walls with Opening

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Abstract

In this paper, results of testing eight clay brick wall specimens are presented. Seven of the walls contained opening and one wall without opening. The walls were tested under constant uniformly distributed vertical load and gradually increased lateral load. Six walls were strengthened using the following techniques: reinforced concrete frame around the opening, reinforced concrete frame around the opening with the beam extended to the ends of the wall, steel channel frame inside the opening, reinforced concrete frame confining the boundary of the wall (outside confinement), near surface mounted strengthening with (4) mm steel bar and strengthening with ferro-cement.

The test results illustrated that walls strengthened with external frame showed increased lateral load carrying capacity, ductility and toughness compared to non-strengthened wall, and all other strengthened walls showed different ranges of increased shear strength capacity. However, the wall strengthened with ferro-cement did not improve the ultimate lateral load carrying capacity but ductility significantly improved. For the wall strengthened with near surface mounted steel bars, the ultimate lateral load carrying capacity dramatically increased but the lateral displacement did not increase notably.

1. Introduction

One of the most commonly used materials in buildings is masonry, construction with masonry form a significant part of the buildings around the world. Many of these buildings are exposed to danger due to earthquake or heavy wind as shown in Figure (1). Generally, walls contain openings like doors and windows and sometimes openings added during renovation. The wall resistance to lateral loads will be extremely reduced. In this case the seismic behavior of the wall is controlled by the creation of a system of wall spandrels and piers. At the beginning of loading horizontal cracks develop at bottom and top, because piers are slender members and rocking will happen during further loading Nayak, S., & Dutta, S. C.[1]

Many strengthening techniques have been developed for efficient strengthening of masonry walls Farooq et al.[2]. However, many of these techniques require skilled workmanship, and their application may affect the building operations in cost and time. On the other hand, the aesthetics of the building may be affected by some strengthening materials such as surface treatment with steel plates or fiber reinforced polymer (FRP) laminates. Therefore, an alternative method of strengthening with low influence to the aesthetics is developed.

Marcari et al.[3] Used Fiber Reinforced Polymers (FRP) composites to strengthen tuff masonry walls against shear-compression load, the test results proved that the lateral strength were increased largely without improvement of the inelastic deformation capacity of walls strengthened with FRP whose ductility was moderately lower than that of control panels.

Li et al.[4] tested six full scale concrete masonry walls, contained rectangular openings occupying 16% of the wall area. Strengthening was done by Near Surface Mounted (NSM) glass fiber reinforced polymers (GFRP) bars for both sides of the walls with different arrangements. The results illustrated that the mode of failure of an unreinforced masonry (URM) wall can be changed from brittle and sudden to ductile behavior by strengthening with carbon FRP (CFRP).

Voon and Ingham[5] explained that the absence of major damage in the solid grout-filled bond beam supported the notion of frame-type action being developed at a later stage of testing, this led to considerable inelastic displacement capacity of the partially grouted masonry walls.

Okail, et al.[6] has shown that confinement of unreinforced masonry wall (URM) with vertical and horizontal reinforced concrete ties improves the ductility and shear strength of the wall and the higher the reinforcement ratio the higher the strength will be.

Elsamny et al.[7] showed that the load carrying capacity of the strengthened walls using galvanized steel wire mesh increased by (135, 153, 167, and 184%) compared to the control wall by fixing steel wire mesh strip in cement mortar around the opening on both sides of the wall of (5, 10, 15 and 20) cm width respectively.

Balsamo et al.[8] showed that significant shear strength increase were achieved by using both GFRP and CFRP grids ranging between about 150%-320%. GFRP grids allowed higher shear strength increase to be achieved, especially if bonded with a cement based mortar. Strengthened specimens showed a uniform crack pattern characterized by cracks with a small width if compared to those observed on the unreinforced panels.

2. Objectives and research significant

The main objectives of this investigation were to study different types of strengthening techniques and their effect on the lateral load capacity of URM containing opening as well as the behavior of the strengthened walls in terms of lateral deflection, toughness and ductility. The techniques include some of the previously used methods with solid walls as well as some innovative techniques such as internal confinement with reinforced concrete and steel frame, near surface mounted steel bars in cement mortar placed diagonally.

3. Experimental program

3.1 .Test Specimens

The experimental program consisted of testing eight masonry wall specimens. The walls were constructed using locally existing hollow clay brick with dimensions (235 × 115 × 75) mm, and type (M) mortar according to ASTM C 270-3[9] was used with (10) mm joint thickness, The overall dimensions of the walls were (1100 × 1100 × 115) mm and were made using one leaf, header bond. All walls except one (CO) contained a door opening with dimension (775 × 450) mm (scale 1:3). Two reinforced concrete beams with dimensions (1100 × 115 × 105) mm were added to the top and bottom of all walls representing footing and slab. The wall specimens were designated as DCL, DCC, DCB, DOC, DSC, DNS and DFC as shown in Figure (2). CO is the wall constructed without opening, and wall DCL is with the door opening as the rest and used as a control wall without strengthening.

3.2. Material Properties

The brick used was red burned clay brick from Aso factory with properties as summarised in Table (1). Mortar type (M) with mix proportion of (1:3) (cement: sand) is used for wall specimen construction and rendering and brick prisms tested according to ASTM C1314-03a[10]. Ordinary Portland cement was used according to ASTM C150-04[11] limits. Fine and coarse aggregate grading were according to specifications of ASTM C33-11[12] and ASTM C136-06[13]. Steel deformed bar (6) mm diameter was used for top and bottom beams and the reinforced concrete confinements with yield strength of (312) MPa and (4) mm diameter steel rebar for strengthening wall DNS with yield strength of (494) MPa. Steel wire mesh with opening size of (12.5* 12.5) mm and average diameter of (0.5) mm, with yield strength of (350) MPa coated with PVC was used for wall (DFC) and (4) mm steel plate with yield strength (250) MPa was used as channel (127 × 50) mm for wall (DSC).

3.3. Strengthening techniques

Strengthening techniques for the six walls were as follows:

1- Internal concrete confinement DCC: The strengthening techniques consisted of adding a reinforced concrete confinement frame with cross-section of (115 × 75) mm at the opening boundary to strengthen the wall, the depth of concrete confinement was in proportion to the brick size. The concrete frame was reinforced with minimum reinforcement, 4Ø6mm steel bar as main reinforcement and Ø6mm @ 150 mm C/C as tie reinforcement. The size of the confinement frame was so arranged so that the clear opening dimensions were kept the same as the control wall (DCL). The internal confinement consisted of two vertical ties forming a closed frame connecting the bottom beam with the door lintel which were cast monolithically with the bottom beam as shown in Figure (2-c).
2- Internal confinement with extended lintel DCB: the same confinement as (DCC) was used except that the top of interior confinement frame member door lintel was made continuous with the same cross-section extended to the ends of the wall as shown in Figure (2-d).
3- External concrete confinement DOC: the wall was strengthened with outside reinforced concrete confinement frame surrounding the wall (like infilled frame). The frame was prepared beforehand, the vertical parts were cast monolithically with the bottom and top beams with reinforcement lapped inside to improve the fixity of the corners joints as shown in Figure (2-e).
4- Internal confinement with steel DSC: A steel plate with (4) mm thickness was used for making a channel cross-section from which a frame was constructed to be used as an interior confinement for door opening. The cross-section of the U-shape was (127 × 50) mm for the whole frame as shown in Figure (2-f). The bottom of the frame was a steel plate 10 mm thick instead of the U-shape section and was fixed to the bottom beam by two screw 6mm diameter and 76mm in length to prevent the overturning of the steel confinement.
5- Near surface mounted strengthening with steel bars DNS: the wall DNS was strengthened with (4mm) steel rebar around the opening for both faces of the wall. Three vertical bars at 100 mm spacing at sides of the door on both faces and three bars at (45°) angle at both top corners of the door opening. The steel bars were fixed in grooves (10mm) wide and (15mm) deep which were cut out after construction of the walls and after curing period in order not vibrate the

wall. The steel bars were put into the grooves and fixed using cement mortar (1:2) of (cement: sand) as shown in Figure (2-g).

6- Wire mesh with plaster DFC: This wall was strengthened with PVC coated steel wire mesh (0.5) mm diameter and (12.5 × 12.5) mm spacing for both faces of the wall. First a layer of cement mortar (6 to 10) mm thick was applied to the wall surface, then the wire mesh was fixed on the wall with anchors (threaded bolts with (4)mm diameter and (38) mm length with washers, fixed into drilled holes at 250 mm spacing vertically and horizontally). After fixing the wire mesh, the second layer of cement rendering was added to cover and fix the wire mesh, the total thickness of the rendering was (20mm) as shown in Figure (2-h).

3.4. Test Setup and Instrumentation

A special steel frame was manufactured for testing specimens as shown in Figure (3). The testing frame consists of a rectangular self-balanced closed frame with internal dimensions 2000 mm length and 2000 mm height, the columns and top beam consist of two UPN 100 welded together to make a box of cross section dimension 150 x 200 m, the bottom beam consists of 4 UPN 100 making two boxes welded together to give a cross section (150 mm width and 400 mm height) to increase the stiffness of the frame. The connections are fixed with eight bolt (20mm) diameter. A long steel rectangular plate (15 × 100) mm is welded on the frame diagonally to prevent sway of the frame. The frame is strengthened at maximum moment locations to reduce displacement. The strengthening is done by welding extra steel plates on the tension or compression face. The vertical and lateral load applying jacks are fixed to the frame using 2 steel plates (300 × 400 × 10) mm and 4 steel bolts so arranged that they can be moved along the member they are attached to.

Two hydraulic hand operated jacks were used in the testing process, the vertical jack (30 T) capacity with pressure gauge fixed on top of the beam to apply vertical load through top beam and the Lateral hydraulic jack with (30 T) capacity is fixed horizontally to apply lateral load while testing the wall specimen. The vertical load applies predefined precompression load on to the wall through one or two steel beam (steel box (100 × 100 × 4) mm with strengthened sides to prevent web buckling. The distributing beam is supported on round bars on steel plate on the top reinforced concrete beam of the wall.

These bar rollers allow the wall to move freely horizontally. A vertical link is provided at the edge of the walls near the horizontal load assembly. The link consists of two rectangular steel bars (40 × 15) mm hinged at bottom with the beam of the frame and connected at top with a 40 mm pin supported on steel plate which in turn is supported on 4 round bar rollers. This link presents the wall from rocking (overturning) and allows the wall to move horizontally because of the presence of the roller and the top pin.

3.5. Testing Procedure

The wall specimen was placed on the bottom beam of the testing frame, a steel plate 8mm thick was put on top beam of the wall, (7) rollers of (25mm) diameter were put between the steel plate and the spreading (distributing) steel beam to convert the vertical point load to uniformly distributed point load on the wall and allows the wall free lateral displacement (free to move in horizontal direction). A load was applied through the vertical jack fixed on top producing a constant stress of (0.35MPa) applied to the wall specimen. Then, the horizontal load was applied in increments of (10) bar (4.309) kN and both horizontal and bottom dial gauges were recorded. The horizontal displacement of the wall was measured at the center of the top concrete beam of the tested wall in the direction of the lateral load on the same line of action using digital dial gauge (CONTROL type) with accuracy (0.001)mm. At the base of the wall, on the horizontal load side, in order to measure the tilting or overturning displacement a dial gauge with accuracy (0.01) mm was installed on the bottom beam to measure the vertical movement as shown in Figure (3). The lateral load was applied to the wall gradually until failure, the dial gauge readings were recorded at every load increment.

4. Results and Discussion

4.1. Load carrying capacity of the walls

The loads at the occurrence of the first shear crack and at failure (ultimate) are summarized in Table (2). The table shows also the ratio of the failure to cracking load. It can be seen that the ultimate lateral load carrying capacity of the control wall CO was 179.47 kN, for the wall DCL without strengthening the maximum lateral load was 64.63 kN, the door opening affected the lateral load capacity of the wall and reduced to 36% compared to the wall CO.

For the wall DCC the ultimate lateral load was 94.8 kN which was 46% higher than DCL. The result of lateral load carrying capacity of the wall DCB was 86.18 kN, by comparing with DCL it can be seen that DCB was stronger than DCL by 33%. For the wall DOC the maximum lateral load was slightly more than DCL by 20% which was 77.56 kN, the reason behind this low lateral load capacity was the spacing of stirrups in the vertical ties was 150 mm, by reducing the spacing the shear resistance of the vertical ties will increase and the lateral load carrying capacity will increase. In the wall DSC the ultimate lateral load reached 107.72 kN which was the maximum among all the walls and was higher than DCL by 66%.

The internal confinement with steel channel was better than the confinement with reinforced concrete because the steel is a ductile material and concrete is a brittle material, the steel channel regained its original shape after unloading, but the concrete confinement showed many cracks and failed in shear finally as shown in the Figures (4-c) and (4-f). For the wall DNS the lateral load carrying capacity was 40% more than DCL. The wall DFC strengthened with ferro-cement, the ultimate lateral load capacity was almost similar to that of DCL which increased only 6%.

4.2. Lateral load versus lateral deflection behaviour

During an earthquake a building is subjected to the effect of lateral seismic load which make the building to deform laterally. Therefore, studying the behaviour of the wall under the effect of lateral loading is very important. How much the wall deforms before it starts cracking, how much is the deflection or displacement of the wall close or at failure.

These properties defines the capability of the wall to withstand lateral force. The general behaviour of all the wall whether strengthened or not followed a linear behaviour for a small lateral deflection, then followed by a non-linear portion of load deflection curve up to the appearance of the first shear crack, after which the relationship of load and displacement continued non-linearly with declining slope until the ultimate load reached which was identified mostly by occurrence of a crack indicated by the sudden drop in load displacement curve or the widening of the existing crack.

In this case the drop in load-displacement curve is less sudden and continuous. Because of the existence of the pre-compression load and the tie force preventing the overturning of the wall which adds to the amount of pre-compression on the wall, the shear resistance of the walls continues at lesser magnitude due to the friction produced by the pre-compression load. Although some of the walls could have sustained some lateral load capacity. However, the test was stopped because of the large displacement and the widening of the cracks as seen from the load-deflection curves and the crack patterns or failure modes as shown in Figure (4).

4.3. Failure modes

Masonry walls are heterogeneous system. Therefore, crack patterns cannot be predicted easily and the modes of failure depend on the properties of the brick and the mortar. In the case of weak mortar the cracks happen through the mortar joints, for walls with strong mortar the cracks pass through the mortar and the bricks. The results of the experimental tests showed that control wall CO without opening and strengthening was cracked along the compression diagonal which is the resultant of vertical and lateral load, in a stepwise pattern through the mortar joints along the diagonal as shown in Figure (5-a). After the first crack, second and third cracks happened in the same way along the diagonal almost parallel to the first crack. The first cracking load and failure load were the same (179.47) kN, because the wall was suddenly failed, the first crack defined the ultimate load and the load declined suddenly with large displacement.

The second control wall DCL with door opening (without strengthening), the first crack occurred at a load of (32.3) kN under the lintel, it developed horizontally at the connection of the right pier and the lintel and at the bottom of the pier where it is in contact with the bottom beam of the wall. After that, instantly followed by diagonal tensile cracks in a stepwise shape along the mortar joints starting from the left top corner of the door opening of the wall (Figure 5-b).

The door opening formed two weak piers leading to the failure at a load of (64.6) kN rocking or tilting behavior was clearly observed in the two piers. The wall DCC, with the internal concrete confinement changed the crack pattern and mode of failure, the first crack started from the top left corner of the confinement vertically through the top spandrel at a load of (17.25) kN.

The second crack was horizontal crack through the mortar bed of the spandrel and right side pier, due to further loading the previous cracks expanded and the vertical tie confinement cracked in the right side at the joint with the bottom beam, with increasing load the bottom beam failed in shear.

Finally after the wall reached its ultimate load carrying capacity at (94.8) kN, the right side pier suddenly failed in diagonal compression failure and the vertical concrete tie failed with the pier with sound as shown in Figure (5-c). The failure mode of wall DCB with extended lintel is different from both previous walls, the first crack happened horizontally simultaneously at the right side under the extended lintel and at the left side over the extended lintel at a load of (34.2) kN. The second crack started in the spandrel extended up to the top beam and in the left pier with the bottom beam.

With further loading a sudden diagonal failure of the left side pier occurred with the failure of the bottom beam in shear. The crack continued widening until the ultimate load was reached (86.2) kN, the spandrel and the left pier failed in compression due to high compressive strength as shown in Figure (5-d). The wall DOC with external confinement showed also a different mode of failure. First crack appeared at the bottom of right side pier and the top left corner of the opening extended diagonally to the top left corner of the wall. After increase the applied load, some horizontal cracks appeared in the upper half of the right side vertical tie and at the bottom right corner of the concrete confinement.

After that, a diagonal step wise crack happened in the spandrel, finally a sudden diagonal shear failure appeared in both piers together with low sound as shown in Figure (5-e). In the wall strengthened with internal steel confinement DSC the first crack was similar to the control wall DCL at (30.15) kN. After that, due to the rotation of both piers the cracks appeared at the bottom between the bottom beam and the piers, finally at the ultimate load of (107.75) kN, the left side pier failed in diagonal shear with sound as shown in Figure (5-f).

The wall DNS strengthened with (4) mm diameter steel bars near surface, the first crack started horizontally at the top of both piers under the concrete lintel at (34.25) kN, then the second crack appeared in the bottom of the right side pier and another diagonal crack from the left top corner of the opening to the top left corner of the wall.

With further increase in loading the cracks expanded to the spandrel along the first steel bar on the right side. After increasing the lateral load the right side of the spandrel separated from the top beam because the reinforcing bars connected the spandrel with the pier. Finally the left pier failed in diagonal shear due to high diagonal compressive strength. The DNS has a crack pattern different from all other walls because the steel bars distributed the cracks all over the area of the wall as shown in Figure (5-g).

Most of the steel bars were cut during the final stage of loading and some others were pulled out (debonded). Finally the wall DFC with ferro-cement strengthening the crack pattern was exactly similar to the control wall DCL in all stages of loading as shown in Figure (5-h). In summary, all tested walls behaved in similar manner and similar mode of failure after the appearance of first cracks that is by dividing the wall in to two piers on sides and a spandrel at top above the lintel and the final failure was due to the diagonal shear failure of one of the piers depending upon the type and arrangement of the strengthened materials.

4.4. Ductility and energy dissipation

Ductility factor is another parameter that to be considered in the behavior analysis of masonry wall. Ductility can be defined as the ability of a material to attain high deformation prior to failure. For masonry wall is the ability of a wall to deform without total failure. The ductility factor can be defined as the ratio of a lateral deflection at the end of elastic range to the lateral deflection in inelastic range at failure for a given masonry wall specimen Lee et al.[14]. Based on this definition the ductility factors are computed and summarized in Table (3). For the control wall CO the ductility factor was 3.72, because the wall behaved brittle manner. The lateral deflection at ultimate lateral load for DCL is more than that of CO. Therefore, the ductility factor for DCL is 7.57. By comparison the strengthened walls has more ductility than un-strengthened masonry walls.

The ductility factor of the DCC was 13.42 because the interior concrete confinement gave the wall a better ductility and the increased lateral deflection, the ductility factor for DCB is almost the same as DCC which is 12.5. For the walls DSC and DFC the ductility factors are 29.82 and 27.35 respectively, the steel U-shape and the steel wire mesh changed the behaviour of the walls from brittle to ductile where the lateral deflection at the failure load were 43.66mm and 51.96mm for DSC and DFC respectively. For the wall DNS the ductility factor is 17.69 because the steel bars changed the failure mode to ductile and increased the lateral deflection at failure load.

According to ASTM C 1018-97[15] energy dissipation can be defined as the area under load-deflection curve and it is an indication of the walls capacity to sustain large deformation before failure. For the purpose of comparison the area under the load-deflection curves are calculated between the beginnings of elastic range to the end of the inelastic range (failure).

The values summarized in Table (3), values indicate that the energy dissipation capacity for the wall DCC is higher than the control wall DCL by 276% because of the existence of the steel channel around the door opening. The energy dissipation value of the wall DSC higher than DCL by 314% because the lateral deflection increased and area under the curve (energy dissipation) increased.

For the walls DCB and DNS the energy dissipation are almost similar which is 160% and 169% respectively and higher than DCL. Finally from the calculated values of ductility factors and the energy dissipations of all wall specimens, it can be noted that the walls DSC, DCC and DFC are the best strengthening techniques for masonry walls with opening because both of them have the highest values of ductility factor and energy dissipation.

5. Conclusions

In this experimental investigation brick masonry walls with opening were strengthened using various techniques and were subjected to in plane lateral loading, from the results of the experimental work, it can be concluded that:

1- Opening in walls had a significant effect on the wall behavior and decreased the lateral load carrying capacity of the wall to 36% when compared DCL with CO.
2- All strengthening materials used in this study such as internal and external confinement with reinforced concrete, U-shape steel for internal confinement, near surface mounted steel bars and ferro-cement are effective in improving the performance of URM walls with door opening, lateral load carrying capacity ranged from (120% to 167%), energy dissipation ranged from (160% to 276%) and ductility factor ranged from (118% to 394%).
3- All walls with opening had almost the same first crack loading, while the ultimate lateral load carrying capacity improved according to the type and arrangement of the strengthening materials and techniques, the percentage of improved ranged from (4%) to (67%) based on the material of strengthening.
4- Among the types of strengthening techniques used in this investigation, the strengthening of wall using steel section around the opening DSC showed the highest lateral load capacity (107.25 kN), ductility factor (29.82) and ratio of energy dissipation capacity to DCL (3.14).
5- Strengthening with ferro-cement on both sides of the wall did not show obvious contribution to increase the lateral load carrying capacity, on the other hand ferro-cement significantly increased the ductility factor and energy dissipation of the wall.
6- Strengthening with near surface mounted 4mm steel bars increased the lateral load carrying capacity and energy dissipation of the wall to a large extend (140% and 169%) respectively.

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تقوية مقاومة القص الجانبية للجدران الطابوقية الطينية المحتوية على الفتحات

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المستخلص

تتضمن هذه الدراسة عرض نتائج الفحوصات المختبرية لثماني جدران مبنية من الطابوق الطيني وتحت تأثير أحمال جانبية في مستوى الجدار. أحتوت سبعة من الجدران على فتحة. وتم تقوية ستة منها بتقنيات مختلفة وواحدة بدون تقوية وواحدة بدون فتحة. كانت أنواع التقوية كالآتي : إطار من الخرسانة المسلحة داخل الفتحة ، إطار من الخرسانة المسلحة داخل الفتحة مع تمديد العتبة فوق الباب الى نهايات الجدار ، إطار من مقطع حديدي داخل الفتحة ، وإطار خارجي من الخرسانة المسلحة ، تقوية الجدار بقضبان فولاذية بقطر (4) ملم قرب السطح مع تغطيتها بمونة السمنت والرمل وأخرها تقوية الجدار بمشبك حديدي مع مونة السمنت والرمل. أظهرت نتائج الإختبارات بأن تقوية الجدران بالاطارات الداخلية أو الخارجية لها مقاومة جانبية أفضل مقارنة بالجدران الأعتيادية. كما إزدادت إستطالتها وقدرتها على تبديد الطاقة. كما وأن تقوية الجدار بالمشبك الحديدي لم يتحسن مقاومته للحمل الجانبي ولكنه أظهر تحسن ملحوظ في الإستطالة. وكذلك تحسنت مقاومة الجدار المقواة بالقضبان الحديدية بشكل كبير ولكن الإزاحة الجانبية لم تزداد بشكل ملحوظ.

الكلمات المفتاحية : إستطالة ، جدران كتل البناء ، فتحة باب ، طابوق الطيني ، مواد التقوية .

Fig.1: Earthquake in Darbandixan (2017). (Source: Researcher)

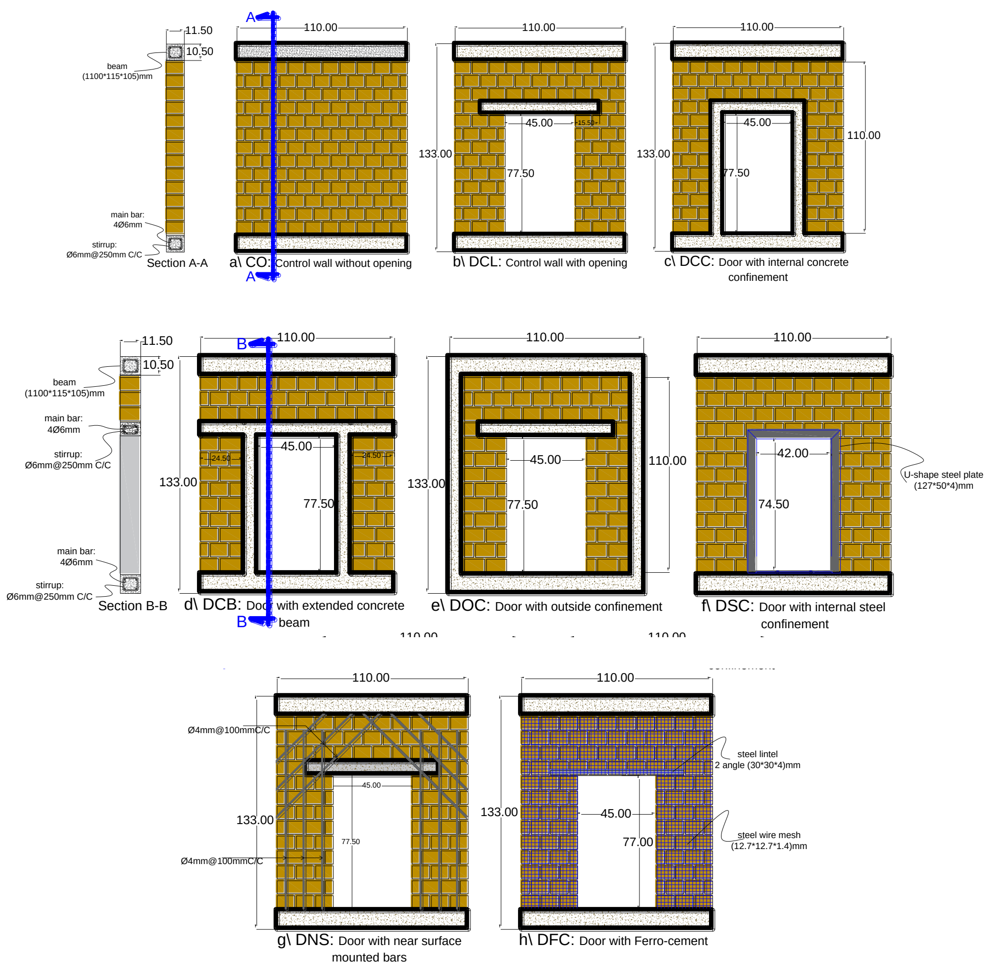
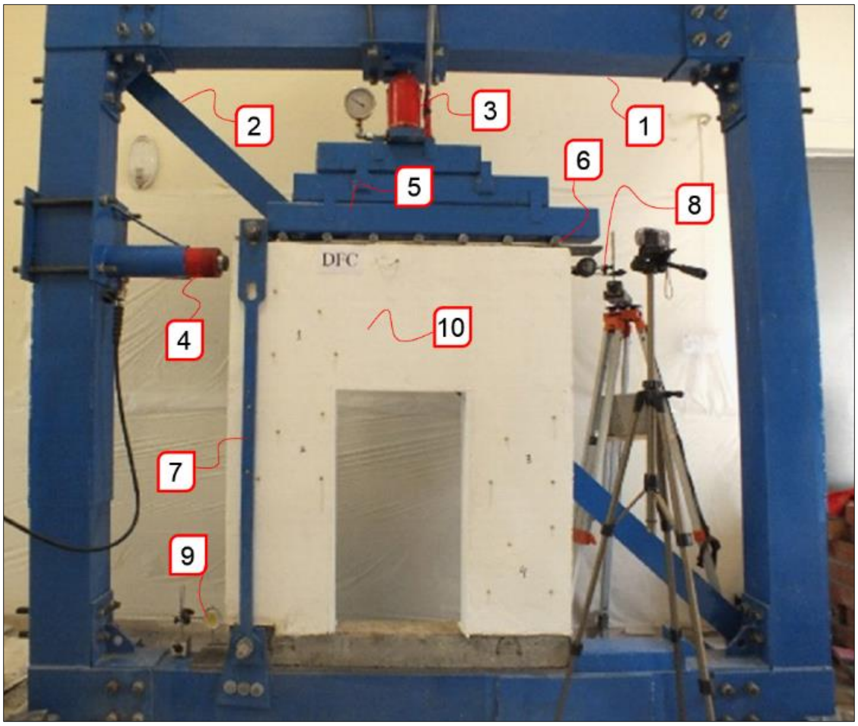
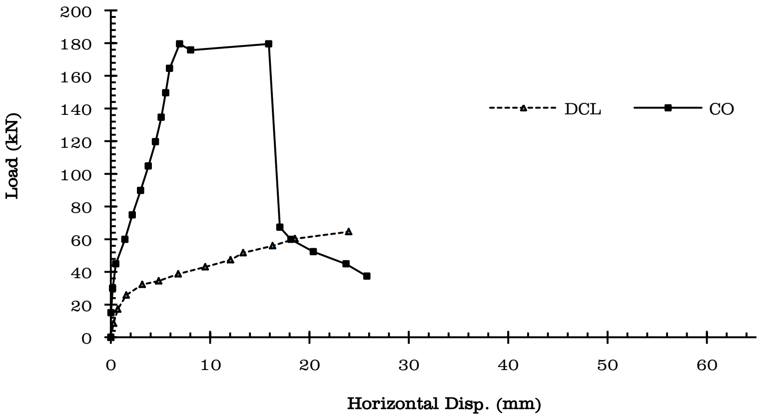


Fig. 2: Detail of tested walls and strengthening techniques. (Source: Researcher) *(All dimensions in centimeter).*

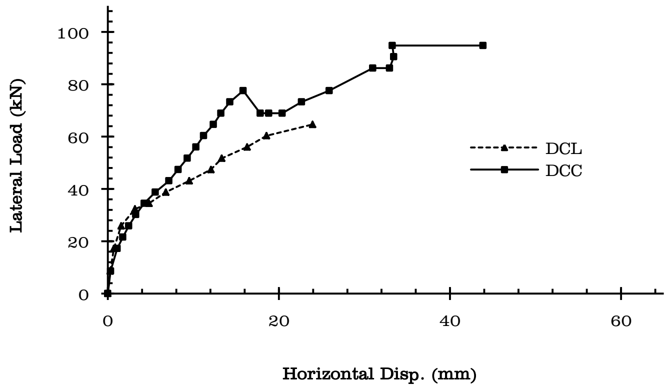


1- Self-balanced closed frame.
2- Rectangular steel plate (15×100) mm.
3- Vertical jack (30 Ton) capacity.
4- Horizontal jack (30 Ton) capacity.
5- Steel square beams (100×100×4) mm.
6- Roller steel bars (25) mm diameter.
7- Vertical link rectangular steel bars (20×40) mm.
8- Dial gauge to measure horizontal displacement.
9- Dial gauge to measure vertical displacement.
10- Wall specimen DFC.

Fig. 3: Testing frame. (Source: Researcher)

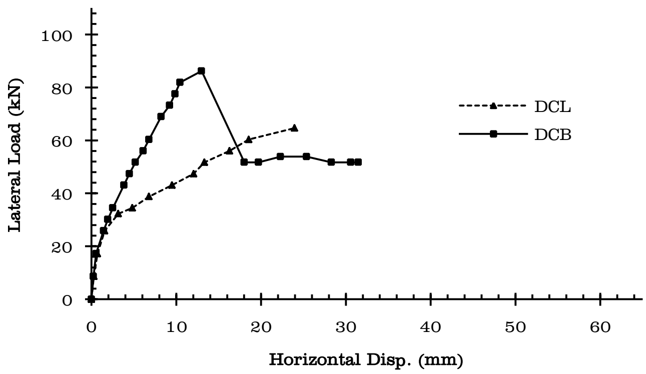


(a)



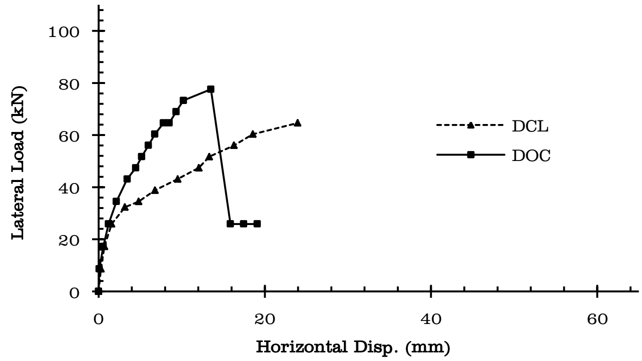
Lateral Load (kN)
Horizontal Disp. (mm)
DCL
DCC

(b)



Lateral Load (kN)
Horizontal Disp. (mm)
DCL
DCB

(c)



Lateral Load (kN)
Horizontal Disp. (mm)
DCL
DOC

(d)

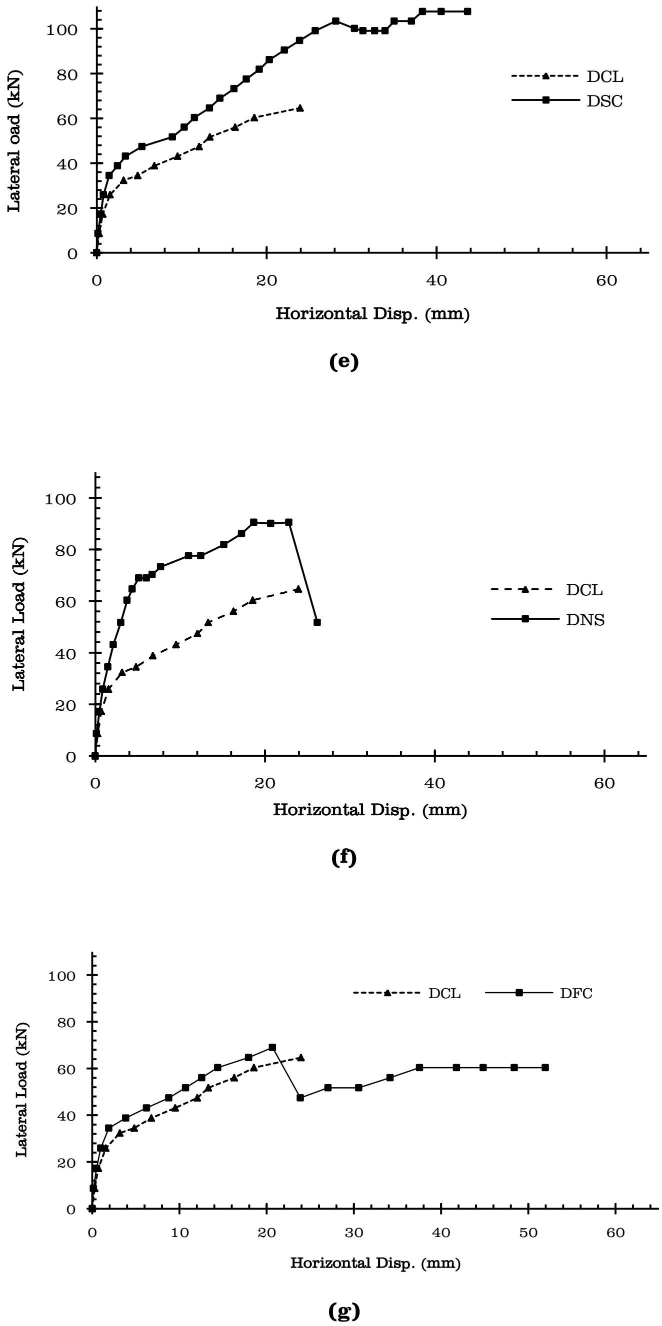
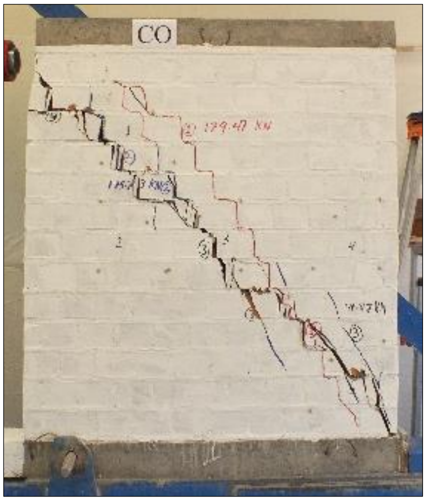


Fig. 4: Load-displacement graphs foe all tested walls. (Source: Researcher)

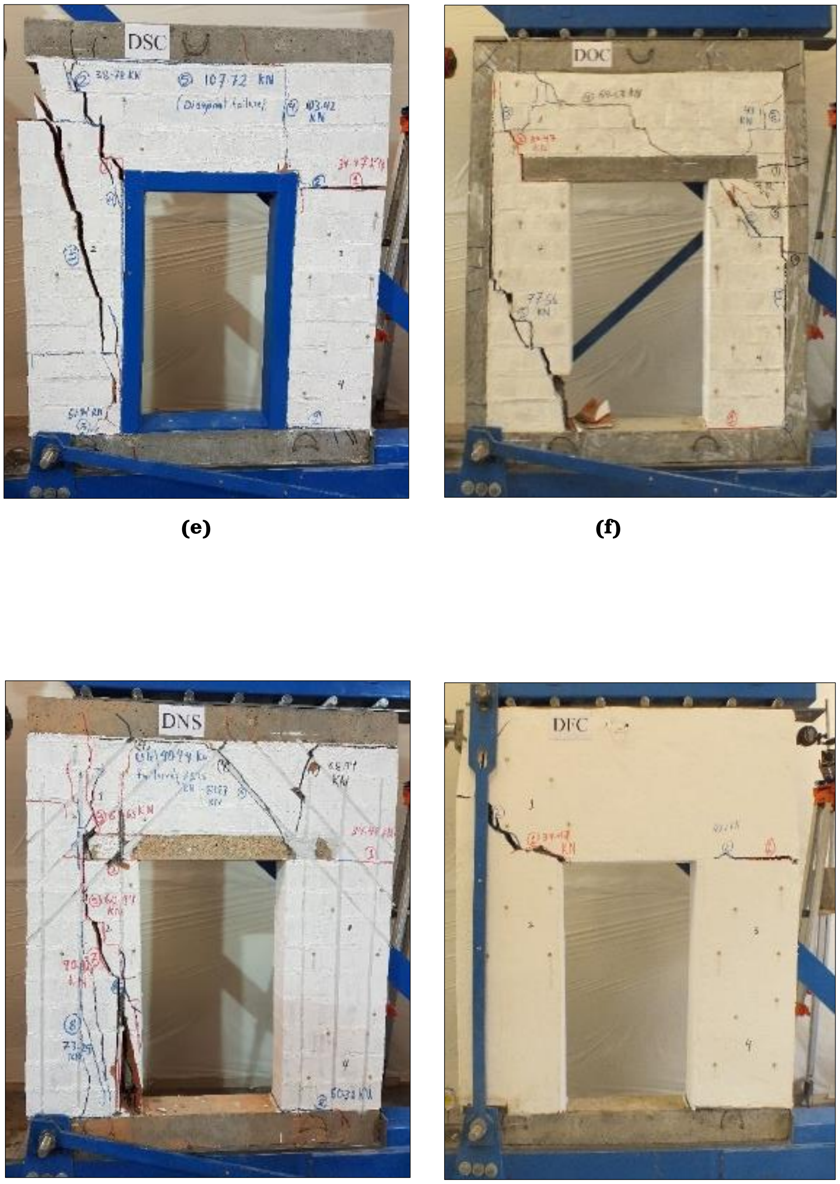


(a)

(b)

(d)

(c)



(e) (f)

(g) (h)

Fig. 5: Failure modes and crack patterns. (Source: Researcher)

Table 1: Properties of used materials. (Source: Researcher)

Material	Compressive strength (MPa)	Flexural strength (MPa)	Tensile strength (MPa)	Specification
Brick	28.39	8.0	---	IQS No.24[16] ASTM C67-03[17]
Cement mortar	44.58	7.74	2.37	ASTM C109-02[18] ASTM C348-02[19] ASTM C307-03[20]
Concrete cubes	37.52	---	---	BS EN 12390-3:2009[21]
Brick prism	43.02	---	---	ASTM C1314-03a[10]

Table 2: Lateral load carrying capacity for the tested walls. (Source: Researcher)

No.	Wall description	Failure due to first crack load (kN)	Failure due to ultimate Failure load (kN)	Ratio of ultimate load to cracking load	Ratio of lateral load to lateral load of DCL
1	CO	179.47	179.47	1.00	2.78
2	DCL	32.30	64.6	2.00	1.00
3	DCC	31.25	94.8	3.03	1.47
4	DCB	34.25	86.2	2.52	1.33
5	DNS	34.25	90.50	2.64	1.40
6	DOC	34.25	77.56	2.26	1.20
7	DSC	30.15	107.75	3.57	1.67
8	DFC	34.25	67.00	1.96	1.04

Table 3: Ductility factor and energy dissipation for tested walls. (Source: Researcher)

Wall	Deflection at first crack (mm)	Deflection at ultimate load (mm)	Ductility factor	Energy dissipation (N.m)	Ratio of E.D. to E.D of DCL
CO	6.92	25.78	2.73	2887.33	2.58
DCL	3.16	23.94	7.57	1117.11	1.00
DCC	3.26	43.85	13.42	3084.85	2.76
DCB	2.52	31.47	12.5	1789.24	1.60
DOC	2.14	19.09	8.93	951.42	0.85
DSC	1.46	43.66	29.82	3505.10	3.14
DNS	1.48	26.17	17.69	1884.40	1.69
DFC	1.90	51.96	27.35	2819.73	2.52