SEISMIC RETROFIT OF R/C FRAMES WITH CFRP OVERLAYS

Experimental Results

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Abstract: This study is part of a research program that is being carried out as a NATO Project 977231 "Seismic Assessment and Rehabilitation of Existing Buildings", led by Middle East Technical University. Within the framework of the research program a new seismic retrofitting methodology is experimentally under investigation. The behavior of hollow brick infilled reinforced concrete frames strengthened by Carbon Fiber Reinforced Polymers (CFRP) has been studied. The main deficiencies of the one-third scale one-bay, two-story test frames were insufficient column lap splice length, poor confinement, and inadequate anchorage length of beam bottom reinforcement. In all specimens beams were stronger than columns and no joint shear reinforcement was used.

Key words: Seismic Retrofit, CFRP, R/C Frame Structure, Lap Splice, Anchorage Length, Brick Infill

1. INTRODUCTION

Although the design philosophy of the Turkish Earthquake Code ensures strength, stiffness and ductility for reinforced concrete frame structures, a wide spread of heavy earthquake damage is observed in such structures during the recent earthquakes due to lack of inspection at the design and construction stages. Frequently encountered deficiencies of the existing

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buildings may be listed as; weak column-strong beam construction, inadequate anchorage length of beam bottom reinforcement, improper detailing and spacing of hoop reinforcement in columns, beams and joints, and insufficient lap splice length for column longitudinal reinforcement. As a result of these deficiencies the structural behavior deviates from a ductile mode and such buildings cannot resist major earthquakes.

Seismic retrofit of structures may either be "member strengthening" or "overall system upgrading". In the member strengthening technique the individual structural elements such as beams, columns or shear walls are strengthened to overcome their existing deficiencies. Such a retrofit approach may not be feasible in the case when numerous members are to be retrofitted. Moreover, member strengthening may usually be ineffective for structures having insufficient lateral stiffness. For such buildings, system upgrading turns out to be the most goal oriented and cost effective seismic retrofit technique.

By system upgrading, it is aimed to increase the lateral load bearing capacity and the stiffness of the overall structure. Addition of new reinforced concrete shear walls for system upgrading has been intensively studied by different universities and applied during recent earthquakes. In this technique, the brick infill walls are demolished before the reinforced concrete shear wall is cast in place. The time of construction for this application is relatively long and the disturbance to the occupants is mostly beyond the comfort limit especially for a pre-earthquake seismic strengthening process. Externally bonded carbon fiber reinforced polymers (CFRP) applied directly on the existing hollow brick walls may be a good solution for this problem due to the rapid application, light weight, and high strength capacity of the material.

2. PREVIOUS RESEARCH

The use of fiber reinforced polymer (FRP) composites in civil engineering applications has grown very rapidly in recent years. Nowadays, civil engineers are interested in exploiting composite materials in structural applications by taking advantage of their high strength-to-weight ratio, corrosion and fatigue resistance, and relatively low cost.

It is anticipated that the use of FRP on masonry will involve walls resisting in-plane and out-of-plane loads and, possibly infilled panels. Indeed, the majority of the work conducted to date has been on the out-of-plane capacity of walls with externally applied FRP.

The use of FRP as a repair and strengthening material may be an alternative to the conventional techniques. Schwegler (1994) and Weeks et

al. (1994) performed various tests on concrete and masonry elements reinforced with FRP. Results showed a marked improvement in the ductility and load carrying capacity of the elements tested [1,2].

Ehsani et al. (1997) studied the effects of shear strengthening with epoxy-bonded FRP overlays applied to the exterior surfaces of brick masonry. The studied parameters were the strength of FRP, fiber orientation and anchorage length. The results showed that both the strength and ductility of tested specimens were significantly increased with this technique [3].

Triantafillou (1998) presented the results of his research related to the use of FRP composites as strengthening materials against shear for concrete, masonry and wood members. He stated that the technique is highly effective and the design of members strengthened with FRP could be based on the classical truss analogy [4].

Triantafillou (1998) also studied the short-term strength of masonry walls strengthened with externally bonded FRP laminates on 12 identical small wall specimens. He developed design models for the dimensioning of FRP reinforcement in masonry walls under monotonic out-of-plane bending, in-plane bending and shear forces, all combined with axial load. It was concluded that highly reinforced locations near the highly stressed zones gave considerable strength increases and the shear capacity of strengthened walls could be very high especially for low axial loads [5].

Young-Joo Lee (1999) investigated the force transfer mechanism between FRP and brick specimens. Glass and carbon fiber reinforced polymer (GFRP, CFRP) sheets were used on brick wall specimens. It is reported that the specimens reinforced with wider FRP sheets showed premature failure modes such as FRP sheet breakage, de-bonding and brick failure. Brick prism specimens reinforced with CFRP showed de-bonding of sheet and shear failure in brick, while the specimens reinforced with GFRP sheet underwent sheet rupture and mortar joint failure [6].

Tumilan et al. (2001) published a paper on Strengthening Masonry Structures with FRP composites and concluded that FRP strengthening increased the shear capacity of the wall specimens and more ductile behavior was observed. It is stated that although the FRP application increased flexural and shear capacities of masonry walls, the effectiveness of this technique is highly dependent upon the FRP end anchorage to the surrounding beam, column or foundation [7].

Marjani (1997) studied the behavior of brick infilled reinforced concrete scaled frames. Effect of plastering on the brick was also investigated. It was found out that hollow clay infill increased both strength and stiffness of the reinforced concrete frames significantly. It was also reported that plastering both sides of the infill improved the behavior of the specimen and improved the ductility by delaying the infill diagonal cracking [8].

Altin (1990) investigated the behavior of reinforced concrete frames strengthened with reinforced concrete infill. Fourteen one-third scale, one-bay, two-story specimens were designed and detailed in accordance with the code and infill walls were introduced to undamaged frames. The main test variables were the pattern of the infill reinforcement, the connection of the infill to the frame, the effect of axial load and the strength of the columns. The loading was reversed cyclic. It was concluded that the reinforced concrete infill walls, which were properly connected to frame members, increased both strength and the stiffness significantly; besides, the column strength and axial load on the columns improved the overall behavior and lateral load capacity [9].

Sonuvar (2001) repaired the heavily damaged one-third scale, one-bay, two-story reinforced concrete frames with reinforced concrete infill walls. Test specimens were constructed with common deficiencies in Turkey such that lack of confinement, poor concrete quality, strong beam-weak column, inadequate lap splice length, poor detailing of beam bottom reinforcement and ineffective ties [10].

Canbay (2001) tested a two-story, three-bay reinforced concrete frame. After the structure was significantly damaged, reinforced concrete infill was introduced in the middle bay. It was reported that the strength, stiffness and energy dissipation capacity of the repaired specimen increased significantly and the newly introduced reinforced concrete infill carried 90 percent of the lateral load [11].

Mertol (2002) tested two one-third scale, one-bay two-story reinforced concrete frames with hollow tile brick infill under reversed cyclic lateral load with constant axial load, one being with CFRP overlay. Test results indicated the importance of CFRP anchorage to the surrounding beams and columns [12].

Keskin (2002) studied the behavior of brick infilled reinforced concrete frames strengthened with CFRP reinforcement on two similar specimens tested by Mertol. In the first test, only one side of the specimen was strengthened with CFRP overlay and it was extended to the frame members. The second specimen was with CFRP overlay applied on either side of the infilled frame. It was concluded that the CFRP anchorage via CFRP dowels play an important role in the behavior and capacity of the strengthened frames [13].

Erduran (2002) also studied the behavior of brick infilled reinforced concrete frames strengthened with CFRP reinforcement on similar specimens tested by Keskin. The CFRP was applied as X-braces on to the brick wall and connected to the surrounding reinforced concrete members. It was pointed out that the tested CFRP detail resulted in a significant increase in the energy dissipation and lateral load carrying capacity. On the other

hand, the failure of CFRP strengthened specimens was relatively brittle once the ultimate load capacity was reached [14].

3. TEST PROGRAM

Four specimens were tested to highlight the effect of brick infill and CFRP overlays on the strength and behavior of poorly constructed reinforced concrete frames (Table 1). All four specimens, namely Pilot, U1, U2, and U3, reported herein, are one-third scale, one-bay and two-story reinforced concrete frames tested under reversed cyclic lateral load. The vertical load on the frames kept constant at a level of approximately 10 percent of the column axial capacities. The main deficiencies of the test frames may be listed as follows:

a) Insufficient lap-splice length of column longitudinal reinforcement.
b) Poor confining reinforcement for both columns and beams.
c) Inadequate anchorage length of beam bottom reinforcement.
d) No joint shear reinforcement.
e) Strong beam – weak column design.

The first set of the specimens, namely Pilot and U1, were tested to observe the bare frame behavior. The difference between bare frame specimens were the concrete compressive strength, anchorage detail of the beam reinforcement to the beam-column connection, and the lap splice length of the column longitudinal reinforcement. Specimens U2 and U3 were tested with hollow tile brick infill in both stories. The brick infill walls were plastered on both sides. The difference between specimen U2 and U3 was the existence of CFRP cross-overlay on the subsequent specimen U3. Frame reinforcement detail for specimens U1, U2 and U3 was kept the same, while the concrete compressive strength was alike for these specimens. Brick infill details for specimens U2 and U3 are given in Figure 1.

3.1 Loading Setup

Bare and infilled frame specimens were fixed at the base and loaded horizontally with a deformation controlled 250 kN capacity hydraulic actuator (Figure 2). For bare frame specimens, namely Pilot and U1, the horizontal cyclic loading was applied to the second story beam level only, while the load was divided into two via a steel spreader beam and applied both at the first and second story levels for brick infilled specimens U2 and U3, such that two thirds of the applied load goes to the upper story level.

Out of plane deformation of the bare frames was not restrained during testing. On the other hand, a steel frame was constructed around the brick

infilled specimens to restrain the out of plane deformations. In either cases, deformations were measured and recorded throughout the test (Figure 3).

3.2 Instrumentation

An electronic data acquisition system with control feedback was used to measure the level of applied load, displacements and rotations. In all the specimens, the reversed cyclic load level and the frame top displacement were monitored to apply the predetermined loading pattern. Curvature measurements on bare frame columns were made to highlight the effect of inadequate lap splice length on the behavior. Two different measurement lengths were used on one of the first story columns to emphasize the effect of the gauge length on curvature readings (Figure 4).

Out of plane displacements were continuously monitored and recorded both for the bare frame (Pilot, U1) and infilled frame (U2, U3) specimens. However, the infilled frames were restrained against such deformations by means of a steel frame constructed in the test rig. For specimens U2 and U3 shear deformations on the brick infill, horizontal base slip, and frame base rocking were also measured (Figure 5). The measurements were relative to the frame foundation in all specimens.

3.3 Reinforcement Detail

Reinforcement detail is the same for all specimens except specimen Pilot; differences being the lap splice length of column reinforcement (l_p=200mm) and the anchorage detail of the beam reinforcement to the beam-column connection as given in Table 1. The longitudinal reinforcement both for beams and columns were 8 mm diameter plain bars with a yield strength of f_y=312 MPa. The yield strength of 4 mm diameter transverse plain bars was f_y=277 MPa. The reinforcement detail of the specimens is given in Figure 6.

4. TEST RESULTS

The reversed cyclic lateral load behavior of the specimens is discussed below. Loading histories, load versus roof drift ratio curves and moment curvature graphs are given for each specimen. In addition, photographs taken after the test are given to show the damage level attained.

4.1 Specimen: Pilot

Specimen Pilot was a reinforced concrete bare frame with a column re-bar lap splice length of $l_b=25D_b$. Concrete compressive strength measured on 150x300 mm cylinders was f_c'=22.4 MPa. The specimen was tested under the lateral load history given in Figure 7. The lateral load was applied as a point load to the top story beam level. The lateral load versus roof drift ratio curve of the specimen is given in Figure 8. The moment curvature behavior of the column sections with lap splice length deficiencies was monitored by two curvature-meters mounted on the same column, with different gauge lengths (Figure 9 and 10). The axial load on each column was 30 kN throughout the test.

During the test, no visible cracks were observed until the end of the first full load cycle where the load level was 5.8 kN. A constant lateral load of approximately 8 kN, was applied for the next three cycles, and during these load cycles flexural cracks were observed at both ends of the first story beam. The flexural cracks on the columns began to widen at the 7th load cycle where the lateral load level was 12.5 kN. Until this load cycle, no visible cracks were observed in the beam column joint since the beam longitudinal bars were not anchored to the joint effectively; hence the transferred joint shear was small.

Between load cycles 8 and 11, the load level was in the range of 13-14 kN and the flexural damage was concentrated on the first story beam ends and both ends of the first story columns. During the load cycle 10 spalling of the concrete cover was observed at the upper end of the first story column to the west. The crack widths at this load level were approximately 3 mm on either first story column bases and approximately 1.5 mm on both ends of the first story beam. The first visible x-cracks on the connection were observed at load cycle 12. The flexural cracks at the upper end of the first story columns were located right at the beam bottom level and widened concurrently with the widening of the flexural cracks at the base of the same members. Minimal cracking was observed on the second story beams and columns.

4.2 Specimen: U1

Specimen U1 was the second of the bare frame specimen set. The reinforcement detail of U1 was exactly replicated for the specimens with brick infill, namely U2 and U3. Concrete compressive strength was f_c'=15.4 MPa. The reversed cyclic lateral load history, lateral load versus roof level drift ratio, and first story column moment-curvature graphs of specimen U1 are given in Figures 11, 12 and 13 respectively. The lateral load was applied

at the second story beam level as a point load. There was a constant axial load of 30 kN on each column throughout the lateral load history.

The first visible cracks were observed concurrently on the base of the lower east column and at the west end of the first story beam in load cycle 3, where the lateral load level was in the range of 7 kN. Hairline cracks on the beam column connection were first observed in 6th load cycle where the load level was 9.5 kN.

During load cycles 7 and 8, new cracks started to form on the lower column ends and on the beam-column connections. Beyond the load cycle 8, the lateral load capacity of the specimen stabilized under increasing lateral displacements. At the end of load cycle 13, the measured crack widths on the first story beam ends and column bases were in the range of 1 and 2 mm respectively (Figure 14).

Damage due to lateral loading was accumulated in the first story columns and the beam. Heavy cracking was observed on the first story beam column joints compared to the specimen Pilot (Figure 15). Two individual flexural cracks on the lower column bases were observed, one being right at the base and the second being at a height of 50 mm (Figure 16).

4.3 Specimen: U2

Specimen U2 was the first specimen tested with brick infill. Frame concrete strength was f_c'=14.7 MPa, while the mortar strength used between the bricks and used as plaster was f_m'=5.5 MPa. Specimen U2 was tested under the lateral load history given in Figure 17, and the lateral load roof drift ratio curve for this specimen is given in Figure 18.

The brick infill was constructed such that it was flush with the outer surface of the reinforced concrete frame in specimen U2. Due to such an eccentric location of the brick infill, specimen U2 was prone to out-of-plane deformations under in-plane lateral loading. Therefore it was restrained against out-of-plane deformations with a steel frame mounted in the test rig (Figure 19). Roller bearings mounted at the second story beam level had no restraining effect in the direction of lateral load. The axial load on each column was 30 kN throughout testing. The lateral load was divided into two via a spreader beam as shown in Figure 2 such that 66 percent of the actuator load goes to the second story level.

The first visible crack on specimen U2 was observed at the second story brick wall to top beam interface at the 5th load cycle, where the lateral load level was 40 kN. In the next cycle, these hairline cracks were extended down through the column-wall interface. Subsequently, new hairline cracks were observed at the first story wall-to-beam interface.

In load cycle 7 horizontal cracks were observed on the first story columns located approximately 200 mm from the base. Concurrently, first story brick walls were cracked at the frame corners and the crack orientation was almost perpendicular to the wall diagonal axis. The load level in this cycle was 50 kN.

In load cycle 8, where the load level was 55 kN, no new cracks were observed. It was seen that the plaster on the frame started to spall off in load cycle 9 and the lateral load level started to decrease. In load cycle 12, the cracks observed in the previous cycles started to widen suddenly and the load capacity degradation was more pronounced (Figure 20). The accumulated damage on specimen U2 at the end of the test is shown in Figure 21.

4.4 Specimen: U3

Specimen U3 was a brick infilled frame specimen strengthened with CFRP (*SikaWrap Hex-230C*) X-overlays. Manufacturer specified "fiber tensile strength" was 4100 MPa, and the tensile E-Modulus was 231,000 MPa for the CFRP. 200 mm width CFRP diagonals were applied on both sides of the specimen U3 (Figure 22). X-overlays were anchored to the reinforced concrete frame and brick infill through CFRP anchors at locations shown in Figure 22. The anchorage depth of CFRP anchors on the reinforced concrete frame was 50 mm (Type A and B) (Figure 23). Type C anchors pass through the brick wall and were bonded to the overlays. In either case CFRP anchors were bonded to overlays over a circular area with a radius of 50 mm.

Concrete compressive strength was f_c'=16.0 MPa and the mortar strength was f_m'=5.1 MPa. The applied lateral load history is given in Figure 24. Load was applied in increments of 5 kN at each load cycle until failure, and displacement controlled cycles were applied beyond that point. Lateral load was applied through a steel spreader beam, through which the story loads were shared as 2 to 1 for the second and the first stories. The specimen was restrained against out-of-plane deformations with the same steel frame used for specimen U2 as shown in Figure 19. Axial load level was kept constant at 30 kN on each column throughout the test.

No visible cracks were observed until load cycle 5 was reached, where the load level was 40 kN. The first crack was observed at the interface between the second story brick panel and the upper beam. The length of the crack was approximately 300 mm. Concurrently, horizontal flexural cracks were observed on the first story columns, at a height of 145 mm from the foundation level (crack number 5 in Figure 26).

In load cycle 8, where the load level was 55 kN, the above mentioned flexural cracks started to be continuous around the column indicating a shift from flexural behavior to an axial tensile behavior of columns. The column base also failed under tension at a further load level (cycle 10, P=65 kN) (crack number 20 in Figure 26). The specimen lateral load capacity was reached when the horizontal cracks on the columns started to widen suddenly and the first story brick panel started to separate from the foundation.

Damage until the ultimate load level was concentrated to the first story columns in the form of flexural and flexural/tensile cracks. The number of cracks on the first story columns was bigger than that of the specimen U2. No cracks were observed on the second story beam and columns throughout the test. Furthermore, no cracking was observed on the first story beams. Damage was concentrated in the first story columns and the brick infill panels throughout the testing (Figure 27).

The following observations were made relative to the CFRP overlay behavior:

1. No tensile failure of X-overlays was observed.
2. x-overlays peeled off under compression,.
3. x-overlays close to the frame corners buckled under compression and cracked.
4. CFRP anchors at the foundation level on the inner side of the frame were pulled out while the ones on the outer side were ruptured.
5. The pull-out-with-concrete-cone type of CFRP anchor failure indicated the deficiency of anchorage depth.
6. Capacity loss may be associated with the failure of CFRP anchors. The first drop in load level is observed concurrently with the pull-out failure of the anchors at the foundation level on the inner side of the frame. Consequently, the lateral load resistance increased again and the second drop in load is associated with the rupture of the CFRP anchors at the foundation level on the outer side of the frame.

Beam-column connections remained uncracked throughout the testing. The dimension of the mostly damaged brick infill area located close to the column-foundation connection was 35x35cm as shown in Figure 27. Failure of the brick in this region made the CFRP overlay buckle under compressive deformations. Besides, first story columns were overstressed at this level.

5. DISCUSSION OF TEST RESULTS

Specimens tested within the framework of this study did not conform to the requirements of the Turkish Earthquake Code [15]. Detailing

deficiencies of the specimens may be listed as: inadequate hoop detailing in beams and columns, the lack of ties in the beam-column connection, insufficient lap splice length of column longitudinal and beam bottom reinforcement. In addition, the concrete strength was around f_c'=20 MPa. Plain bars were used as longitudinal reinforcement and the workmanship was poor.

The lateral strength and stiffness of the frames tested were closely related to the reinforcement detail, concrete compressive strength and the existence of hollow clay brick infill and CFRP x-overlays.

Beam longitudinal reinforcement anchorage detail is different between specimens Pilot and U1. As a result of this difference the overall crack pattern is significantly affected. Flexural cracking was concentrated on the first story beam ends in specimen Pilot while heavy cracking in the beam column joint was observed in specimen U1. This may be due to the smaller shear force transferred to the joint in specimen Pilot in comparison with U1.

The first flexural cracking in bare frames was observed on the both ends of east column at the first story during a westward load cycle. In further load cycles beyond the ultimate capacity, the flexural cracks on the first story columns just beneath the first story beam were widened and led the specimen to sudden failure. The reason may be the insufficient lap splice length of the column re-bars at the first story level.

On the other hand, first cracking was observed between the brick infill panel and the frame members in the case of the infilled specimens. Significant capacity loss and relative slip between panels and the frame elements was observed concurrently in the case of specimen U2. Consequently, infill wall compression struts started to fail resulting in rapid degradation in strength and stiffness.

The specimen with CFRP x-overlay U3, experienced a behavior different than U2. Initial cracking was observed in the first story columns at a height of 145 mm, where the lap splice length was 160 mm. Significant loss in lateral load capacity was observed associated with the sudden widening of these cracks on either of the first story columns and the pull-out failure of the anchors at the foundation level on the inner side of the frame. Consequently, the lateral load resistance increased again and the second drop in load is associated with the rupture of the CFRP anchors at the foundation level on the outer side of the frame. At further load cycles, overall frame base experienced slip and base rocking in specimen U3. The negative slope after the peak load was comparable in specimens U2 and U3.

The difference in behavior due to the existence of brick infill walls and the CFRP x-overlay can be seen in Figure 28 clearly. The initial stiffness of brick infilled specimen and the specimen with CFRP x-overlay were similar. Peak displacements and lateral loads are given in Table 3.

The peak loads were attained at 1.0 0.2 and 0.3 per cent roof level lateral drift ratios in specimens U1, U2 and U3 respectively. The same ratio was 1.7 for specimen Pilot. It should be noted that U2 and U3 were infilled specimens, latter being strengthened with CFRP overlays. The lap splice length of specimen U1 was smaller than that of the Pilot specimen.

6. CONCLUSIONS

The conclusions given here are mainly based on the test results reported by the authors.

a) The improper lap splice length in the columns mainly governed the ultimate load level and the failure mode of the specimens.
b) Existence of brick infill reduced the drift level attained at the peak lateral load, and the application of CFRP x-overlays seemed ineffective in increasing the drift levels at failure load.
c) Initial stiffness values were comparable in specimens with brick infill and with CFRP x-overlay.
d) Under tensile forces CFRP x-overlays remained intact, but under compressive forces they peeled off and buckled in the post-peak region.
e) The specimen with CFRP x-overlay failed due to the insufficient CFRP anchorage detail provided. Pull-out and rupture type of CFRP anchor failures were observed on the same specimen.

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NOTATION

D_b : reinforcing bar diameter (mm)
f_c' : concrete compressive strength (150x300mm cylinders), (MPa)
f_m' : mortar compressive strength (50x50x50mm cubes), (MPa)
P : corrected total lateral load (kN)
l_p : lap slice length (mm)
Δ : roof level displacement under lateral load (mm)

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Table 1. Specimen Properties

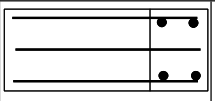
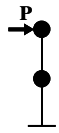
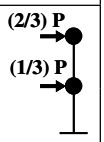
Specimen	Column Re-bar Splice Length	Beam Re-Bar Anchorage Detail	Concrete Strength f_c' (MPa)	Mortar Strength f_m' (MPa)	Total Vertical Load, N (kN)	Lateral Load Applic. Point
Pilot	$25 \times D_b$ (200mm)		22.4	N/A	60	
U1	$20 \times D_b$ (160mm)		15.4	N/A	60	
U2			14.7	5.5	60	
U3			16.0	5.1	60	

Table 2. Roof Displacement and Lateral Load Peaks in Positive Quadrant

Pilot		U1		U2		U3	
H (kN)	Δ (mm)	H (kN)	Δ (mm)	H (kN)	Δ (mm)	H (kN)	Δ (mm)
6.03	1.69	4.05	0.72	20.50	0.01	19.19	0.06
7.77	3.20	6.04	1.99	26.10	0.03	25.21	0.14
8.08	4.10	7.55	3.95	30.00	0.07	32.08	0.32
7.70	4.07	7.48	4.43	34.10	0.10	35.05	0.39
9.52	5.98	7.24	4.54	39.90	0.26	40.36	0.52
11.24	10.55	9.48	6.66	43.90	0.78	44.00	0.61
13.18	14.02	9.89	7.87	49.10	1.64	48.70	0.74
13.75	17.84	11.00	10.98	53.40	2.77	54.99	1.03
14.02	20.81	11.16	17.20	59.30	3.93	59.52	1.25
14.05	29.21	11.07	20.38	44.20	6.14	64.87	1.70
13.80	36.68	10.28	22.24	39.37	6.64	70.01	2.17
13.24	43.75	10.42	25.71	40.55	7.88	73.90	3.19
11.42	52.13	9.64	35.35	38.55	10.49	75.15	4.93
9.96	59.48	7.93	46.02	31.88	15.78	64.14	6.37
		6.16	56.53			58.73	7.73
						51.64	9.53
						55.77	12.41
						62.72	15.72
						70.42	25.21
						44.64	31.81

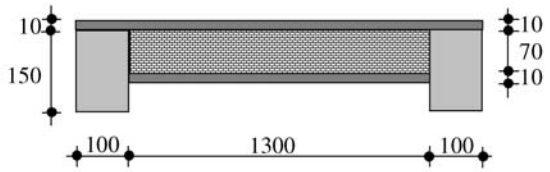


Figure 1. Brick Infill and Plastering Detail for Specimens U2 and U3

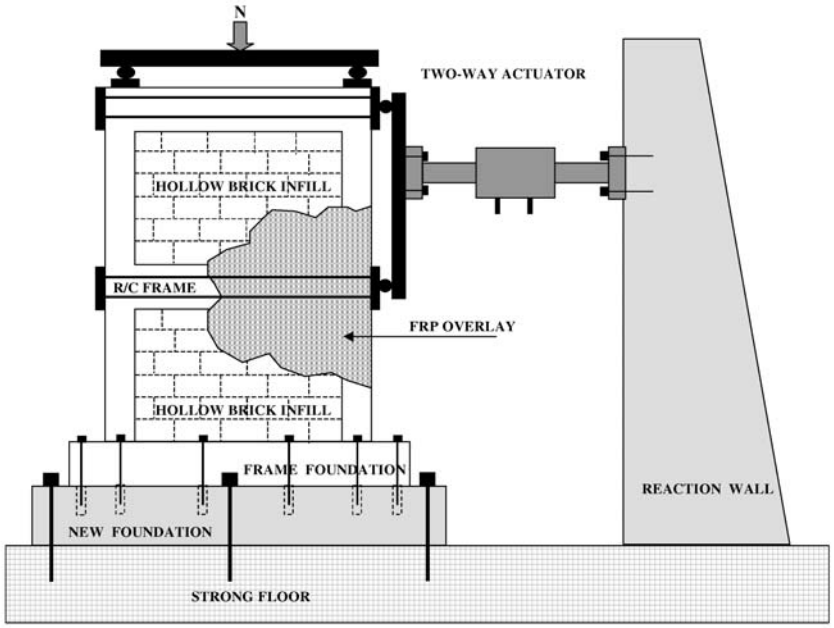


Figure 2. Test Setup

Figure 3. Loading and Data Acquisition Systems for Specimen Pilot

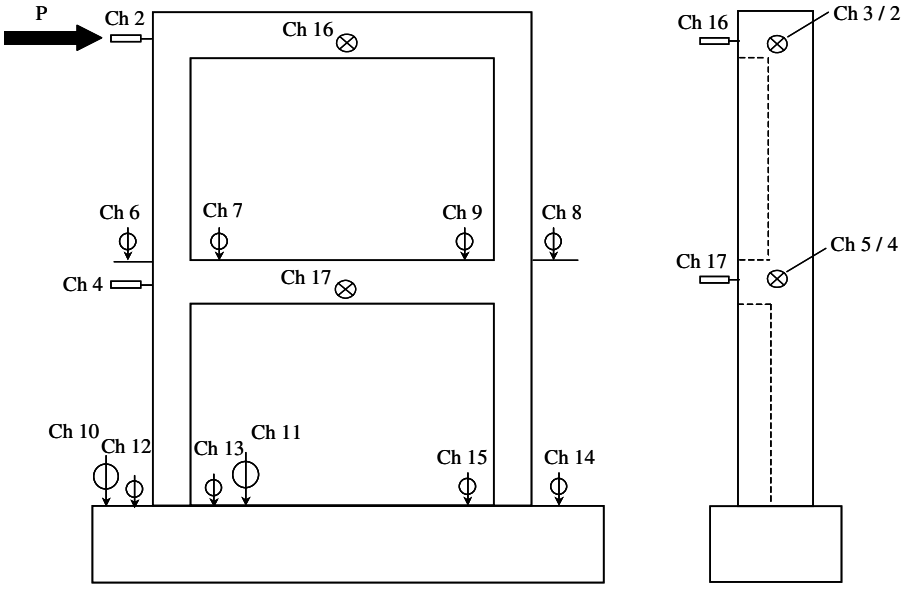


Figure 4. Measurement Points for Bare Frames (Pilot and U1)

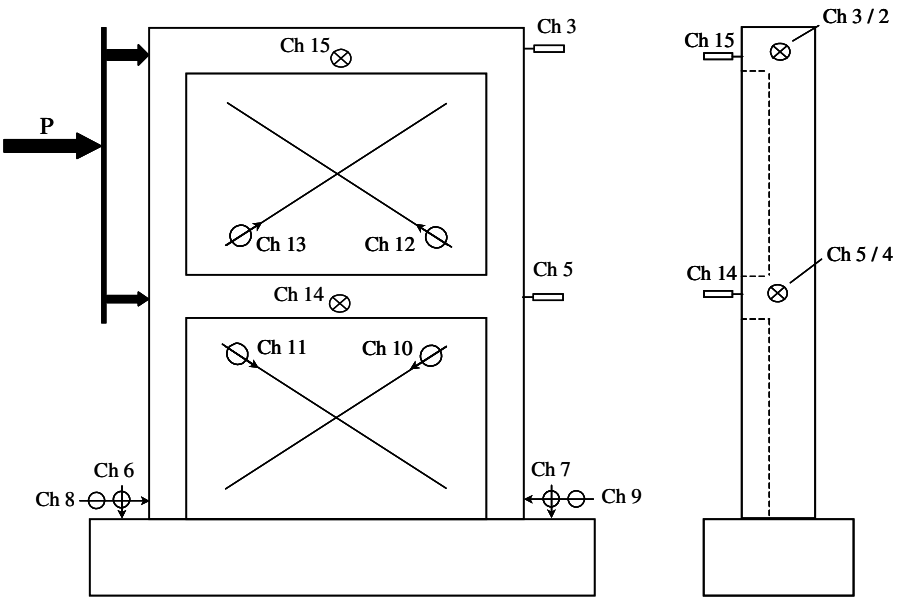


Figure 5. Measurement Points for Infilled Frames (U2 and U3)

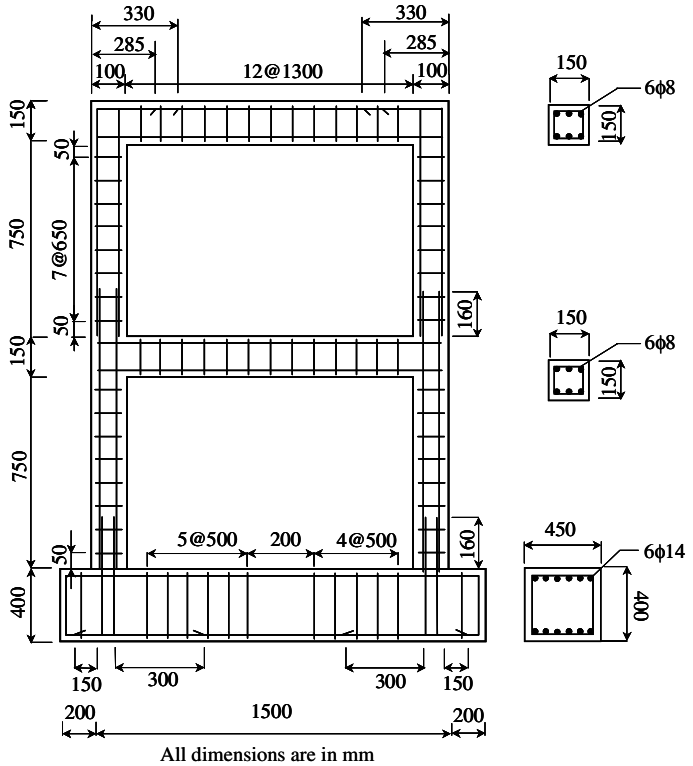


Figure 6. Reinforcement Detail of Specimens

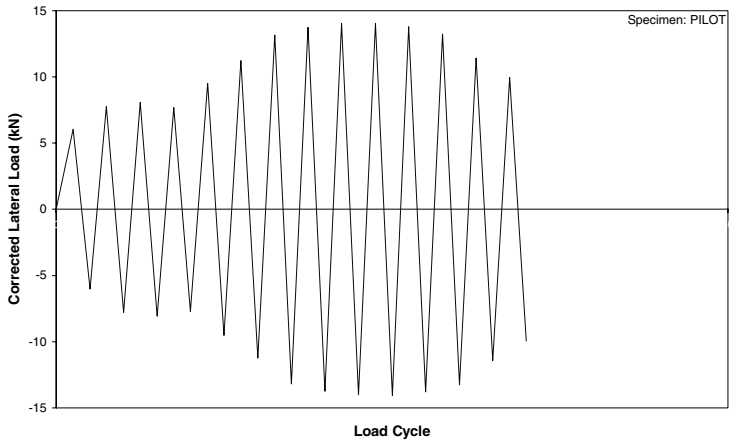


Figure 7. Loading History of Specimen Pilot

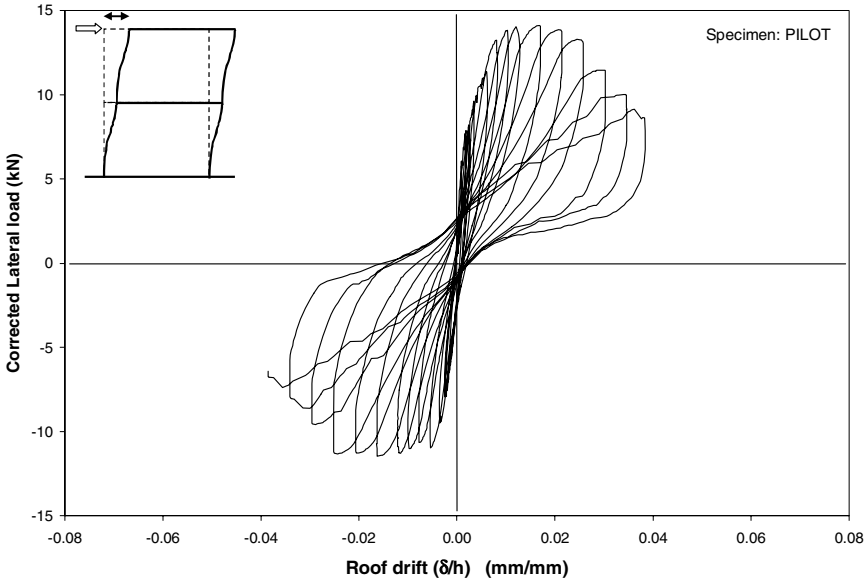


Figure 8. Load–Roof Drift Curve of Specimen Pilot

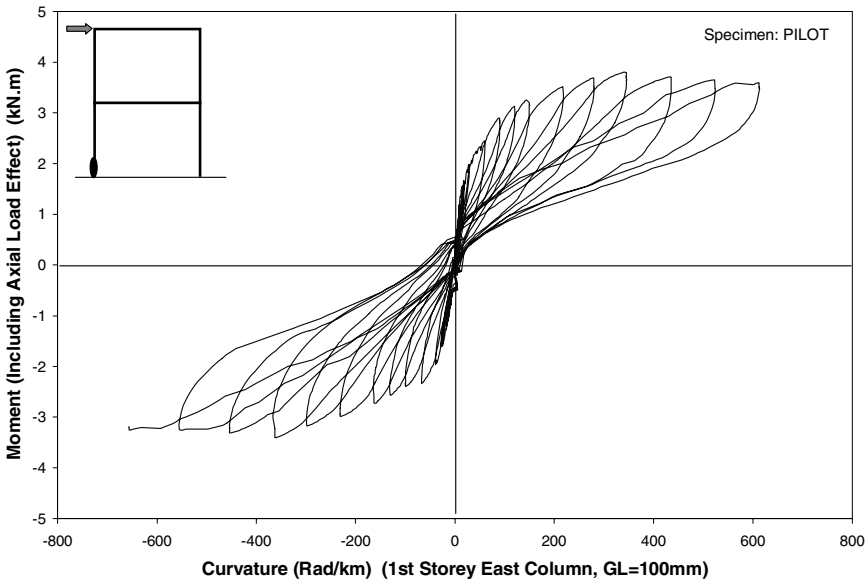


Figure 9. Column Moment-Curvature of Specimen Pilot (GL=100mm)

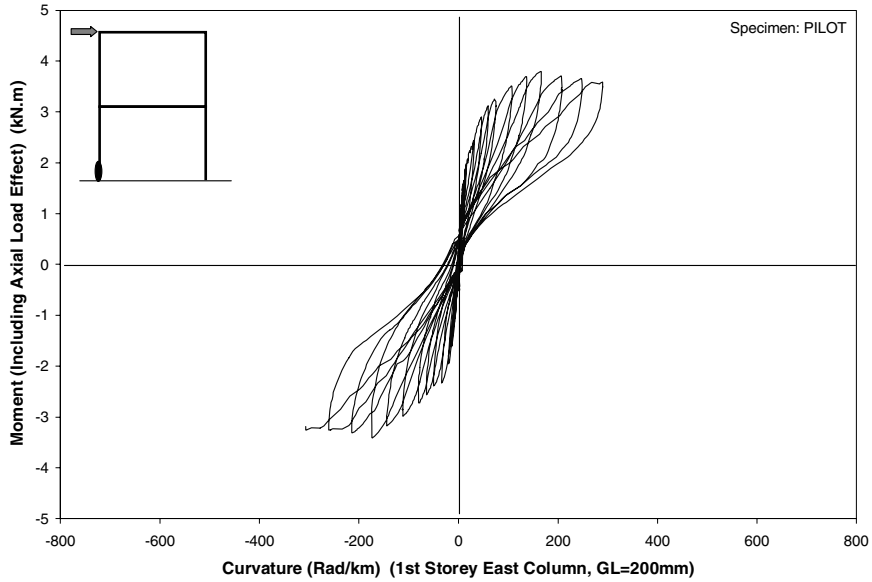


Figure 10. Column Moment-Curvature of Specimen Pilot (GL=200mm)

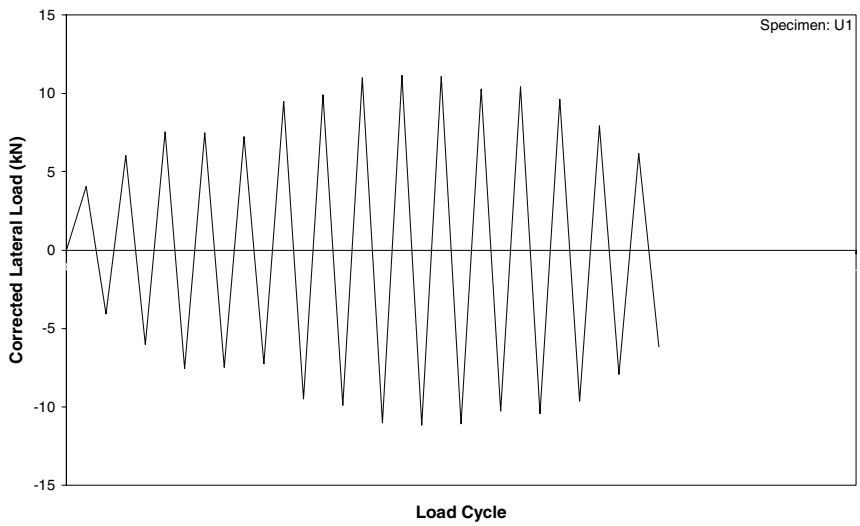


Figure 11. Loading History of Specimen U1

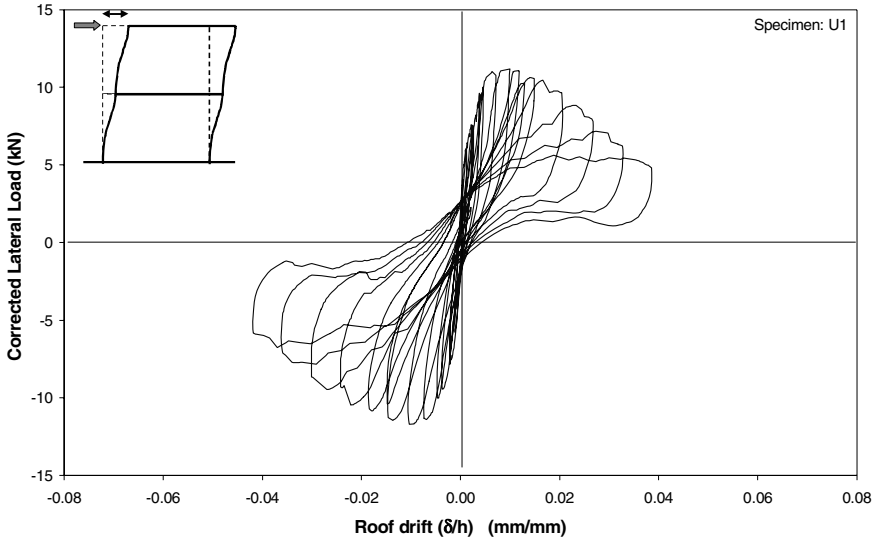


Figure 12. Load–Roof Drift Curve of Specimen U1

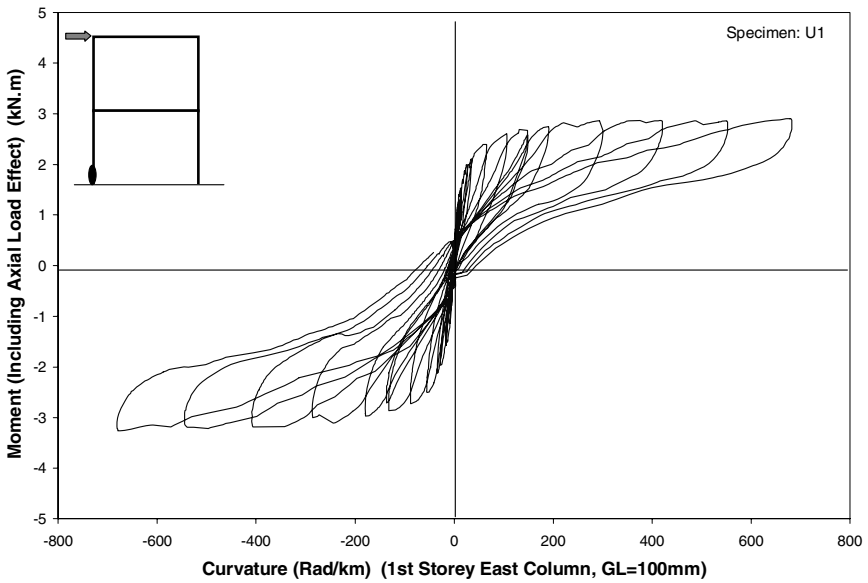


Figure 13. Column Moment-Curvature of Specimen U1 (GL=100mm)

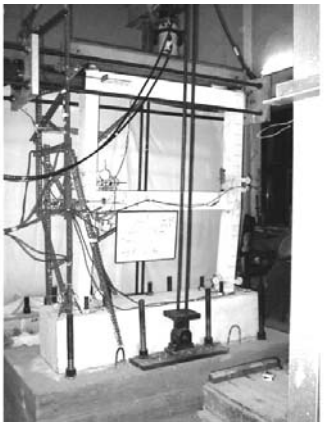


Figure 14. Specimen U1 after the Test

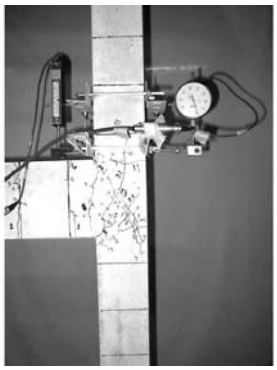


Figure 15. Connection Damage on Specimen U1

Figure 16. Column Base Damage on Specimen U1

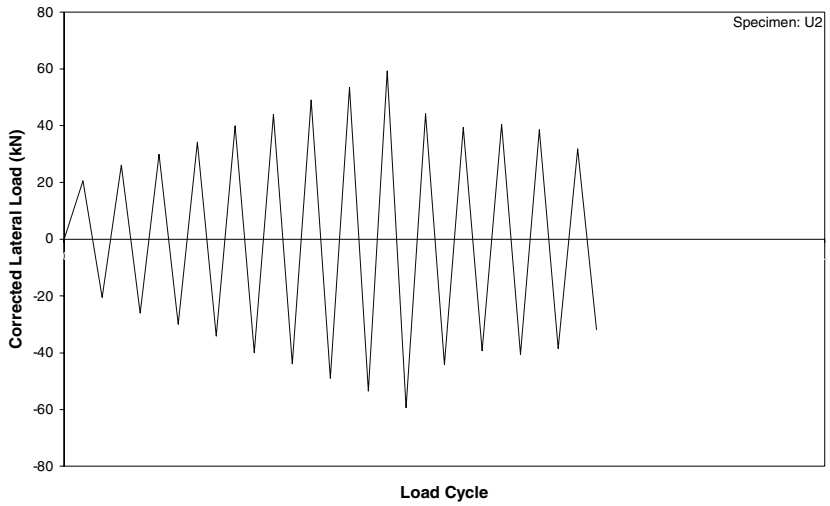


Figure 17. Loading History of Specimen U2

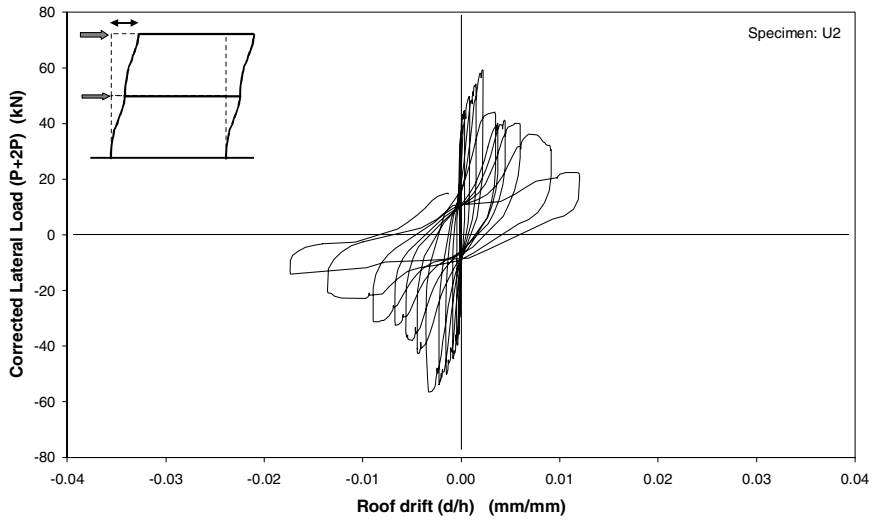


Figure 18. Load-Roof Drift Ratio of Specimen U2

Figure 19. Restraining Steel Frame and Specimen U2

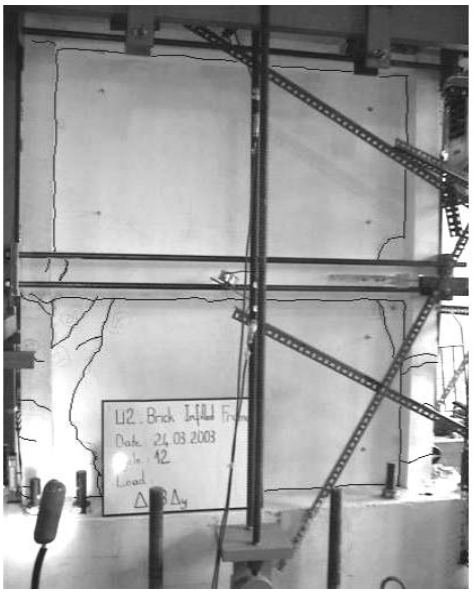


Figure 20. Crack Propagation on Specimen U2 at Load Cycle 12

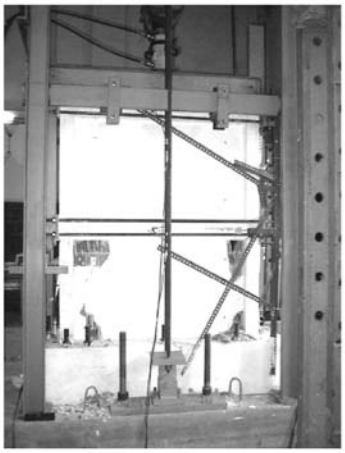


Figure 21. Specimen U2 after Test

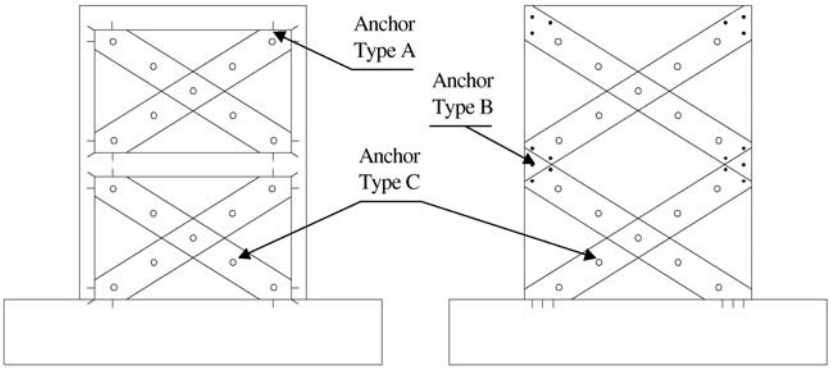


Figure 22. CFRP X-Brace Detail in Specimen U3

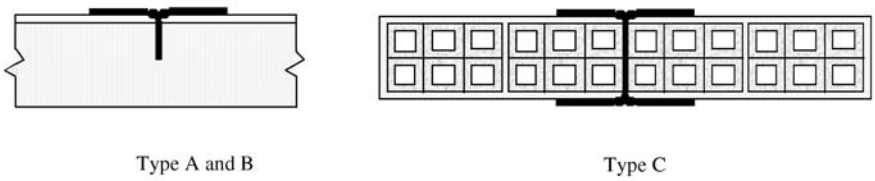


Figure 23. CFRP Anchor Detail for Specimen U3

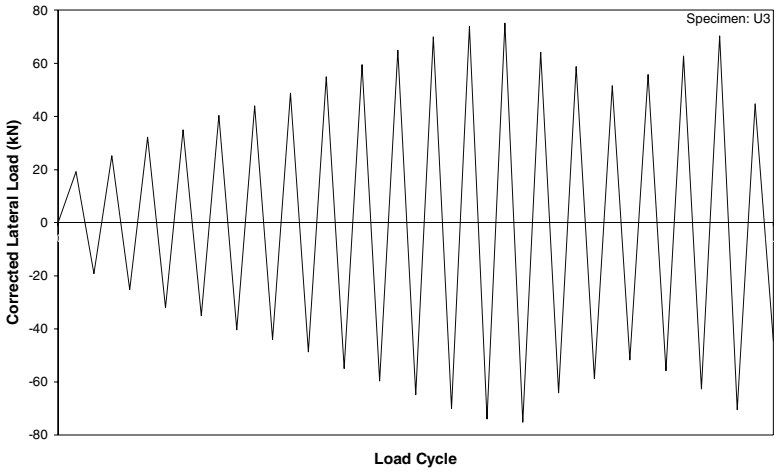


Figure 24. Loading History of Specimen U3

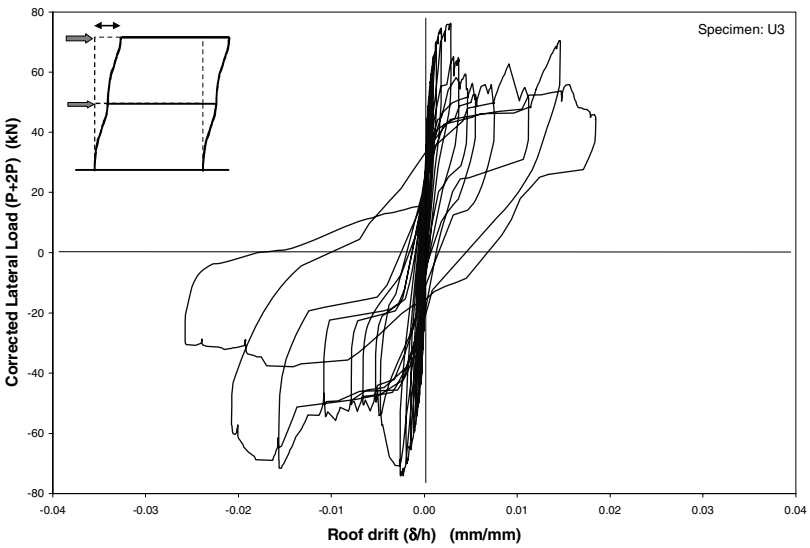


Figure 25. Load-Roof Drift of Specimen U3

Figure 26. Column Cracking Due to Inadequate Lap-Splice Length – Specimen U3

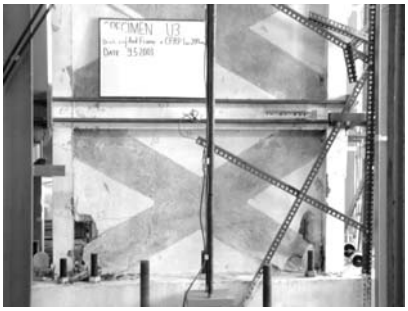


Figure 27. Specimen U3 after Testing

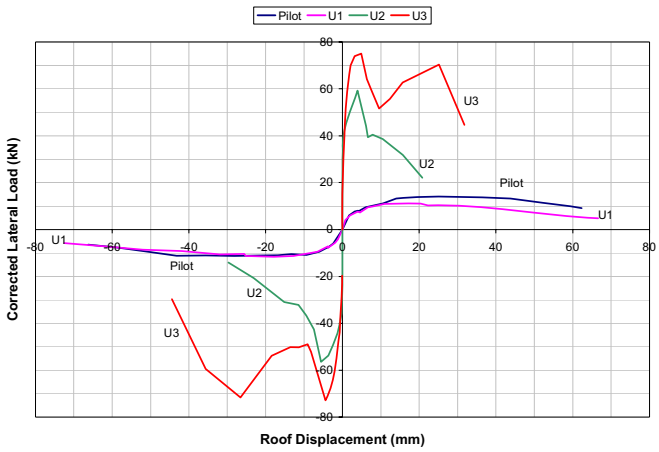


Figure 28. Load-Roof Displacement Envelope Curves

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