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STRENGTHENING OF BRICK INFILLED R/C FRAMES WITH CFRP REINFORCEMENT

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ABSTRACT

Intensive experimental research carried out for decades showed that strengthening of reinforced concrete (R/C) frames by introducing R/C infills to the selected bays in both directions is an effective method for the rehabilitation of damaged structures. This procedure, however, requires evacuation of the building for several months. Therefore its applicability in the rehabilitation of the existing structures, which are currently in use, is neither feasible nor practical. Observations of poor building performance after the recent earthquakes in Turkey and elsewhere and the presence of the enormous size of the available building stocks necessitate an urgent need for the development of innovative strengthening techniques, which would not interrupt the use of the building during rehabilitation. An experimental study was initiated at the METU Structural Mechanics Laboratory, which aimed to develop such strengthening techniques. In this study, it was intended to convert the non-load bearing existing masonry walls and partitions into structural elements which would form a new lateral load resisting system by strengthening them with CFRP fabrics and integrating them with the existing structural system. In this context, six 1/3 scaled 2-story 1-bay reinforced concrete frames were tested. The frame of the test specimens was detailed to include the common deficiencies of the structures in Turkey. The height/width ratio of the infills, arrangement of the CFRP layers and the amount of CFRP used, the anchorage of CFRP fabric to the wall and the frame elements were the major parameters investigated. This paper summarizes the tests carried out to develop an efficient strengthening method for existing structures by the application of CFRP fabrics to the hollow clay tile infills.

Introduction

The colossal number of seismically deficient reinforced concrete structures throughout the world forced the researchers to work on developing rapid and effective rehabilitation techniques. The related research resulted in various rehabilitation methods, among which introduction of reinforced concrete (R/C) infills was proven to be very effective, (Sonuvar 2001; Canbay 2003). This particular method has found wide acceptance all over the world and has been applied successfully after the recent earthquakes in Turkey. The most serious limitation of this method is that it requires the evacuation of the building during the rehabilitation period. Due to this limitation, this technique does not seem to be feasible for the huge building stock that needs rehabilitation is concerned. Therefore, a faster and easier method, that would not interrupt the use of the building, should be developed, (Ersoy 2003; Erdem 2006; Özden 2003; Van

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Den Einde 2003). In this context, experimental studies were initiated at the Middle East Technical University Structural Mechanics Laboratory early in 2001.

The main objective of these experimental studies was to develop such a strengthening technique for the seismic upgrading of existing RC buildings. For this purpose, externally bonded Carbon Fiber Reinforced Polymers (CFRP) fabrics were used. To investigate the effectiveness of the proposed method on deficient R/C frames, seven 1/3 scaled, 2 story frames were tested. Six of these specimens were strengthened by applying CFRP fabric to the existing hollow clay tile infills. The seismic safety level aimed in this study was the prevention of collapse during a major earthquake. The main parameters investigated were the amount of CFRP fabric used, arrangement of the CFRP layers and the anchorage of the CFRP fabric to the wall and frame elements. The test results indicated that (Özcebe 2004):

- converting masonry infills into structural walls is possible by strengthening such non-structural members by CFRP sheets and strips connected to the frame members,
- applying CFRP sheets on both faces of the infills and anchoring them into frame members increased the strength of the specimen to levels more than twice that of the unstrengthened specimen,
- in strengthened specimens, no damage was observed on the infill even at high interstory drift ratios up to 1 percent,
- stiffness increase was about 30 percent of the unstrengthened specimen, and
- CFRP strengthening increased the energy dissipation capacity of the infilled frames significantly.

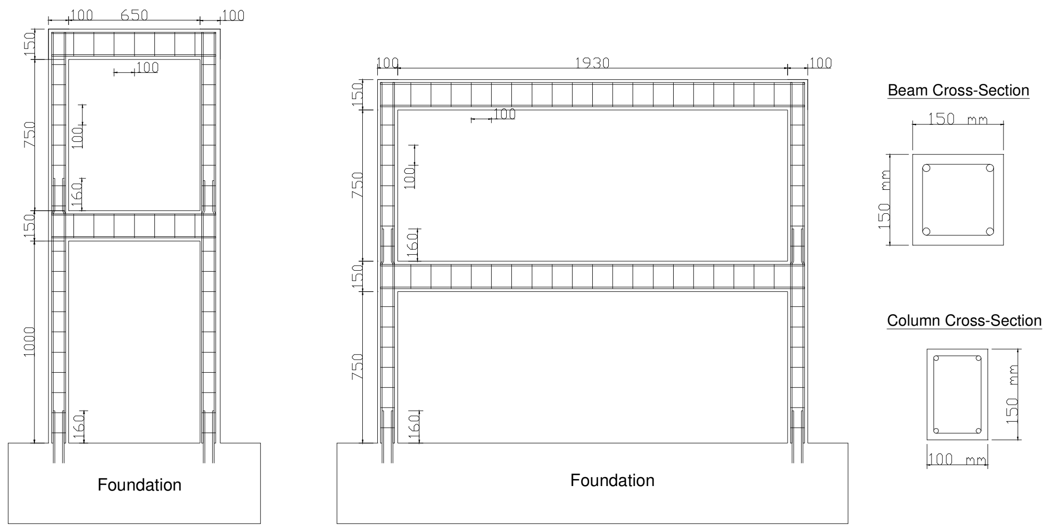
Above tests were performed on frame specimens having height/depth ratios being equal to 1. The main objective of the current study is to investigate the influence of height/width (h/w) ratio of the frame cells on the effectiveness of the previously developed rehabilitation methodology. For this purpose, six 1/3 scale 1-bay-2-story frames with varying h/w ratios were built and tested in the laboratory. This paper presents the preliminary findings of this investigation.

Test Specimens

Six one-bay, two-story RC frames with common structural deficiencies observed in the Turkish practice were constructed and tested in two series having two different h/w ratios. The test specimen was a 1/3 scale model of a non-ductile frame having weak columns and strong beams. The column and beam reinforcements were plain bars. No confinement was provided at the beam-column joints, i.e. ties were not closely spaced at the member ends, and the free ends of the ties 90º hooks anchored in the cover. No transverse reinforcement was provided at beam-column joints. Furthermore, the beam reinforcement was detailed considering gravity loads only. This led to inadequate anchorage of the beam bottom reinforcement. All frames had lapped splices in column longitudinal bars, made both at the first and second story floor levels. Although the lap splice length required for plain bars by the Turkish Seismic Design Code (8) was forty times the diameter of the longitudinal bars ($40d_b$), the longitudinal reinforcements of the columns had splices made at the floor levels with inadequate lapped length, i.e. 20 d_b. The quality of the concrete was poor, i.e. the compressive strength was about 16 MPa.

The geometry and reinforcement details of the test specimens are shown Figure 1. The sizes of the columns were 100 x 150 mm and beams were 150 x 150 mm. Four 8 mm diameter bars were used as the column longitudinal reinforcements. The beams had six 8-mm diameter bars. The transverse reinforcement was fabricated by using 4 mm diameter plain bars which were spaced at 100 mm. In the construction of infilled frames 1/3 scale hollow clay tiles were used. 10 mm thick plaster is applied on both faces of the infill. Upon the construction of the R/C frames, the hollow clay tile infills were constructed and plastered by a professional mason.

First series specimens had an h/w ratio of 1.4 (N-Series). This ratio was 0.4 in the case of second series specimens (L-series). Each series consisted of 3 specimens. First specimen was tested to see the performance of bare RC frame only. The second specimen, however, was prepared to assess the effect of masonry infill wall on the bare frame response. In a sense, these two specimens were used as reference specimens. The third specimen, on the other hand, was strengthened by using the rapid and user friendly rehabilitation methodology developed in the previous studies carried out in METU (Özcebe 2004; Özcebe 2006). The strengthening was made by applying one-directional CFRP sheets diagonally on both sides of the infill walls.

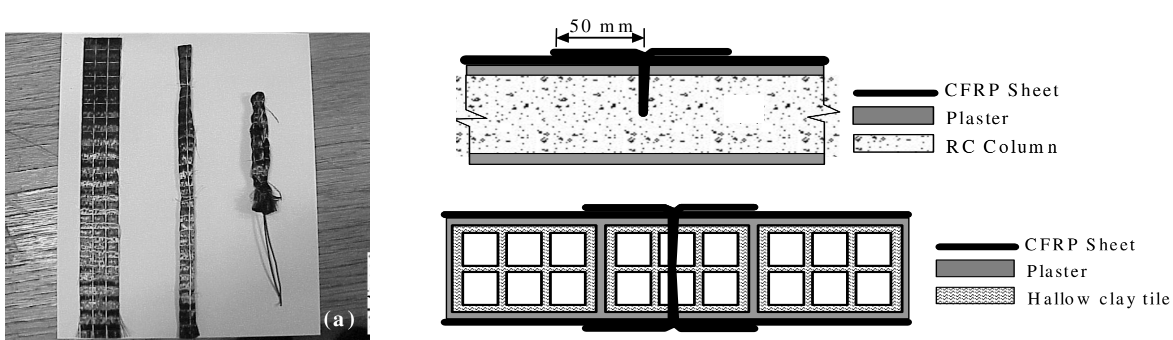


Series-N Specimens

Series-L Specimens

Figure 1. Dimensions and reinforcement details of the test specimens.

For strengthening purposes, the CFRP sheets were bonded directly to the infill wall along the main diagonals of the infill walls, without removing the plaster, by using a special adhesive as recommended by the manufacturer. The width of the diagonal CFRP strips was taken as the bigger of the one-fifth of the wall length or one-fifth of the wall height. The CFRP strips were extended to the RC frame members. To prevent premature debonding of the CFRP, as well as the plaster, the CFRP sheets were anchored to the frame members using specially made CFRP anchors (Özcebe 2004), Figure 2a. Holes having a depth of 50 mm and a diameter of 10 mm were drilled into the frame members. After placing the CFRP sheets on the specimen, the drilled holes were filled with epoxy and the anchor dowels were implanted in these holes by using the guide wires. The fibers of the anchors outside the holes were pierced using a knife and then these fibers were bonded to the CFRP sheets, Figures 2b and 2c. In order to confine the lapped splice regions, the bottom ends of the columns were covered up with one-layer of CFRP. Before applying carbon fiber at lapped splice regions, corners of column concrete were smoothened to prevent any possible rupture of CFRP. The CFRP scheme of the strengthened specimens is given in Figure 3.



(c)

Figure 2. (a) Made-up Anchor dowels; and application of anchor dowels to (b) the frame members and (c) the masonry walls.

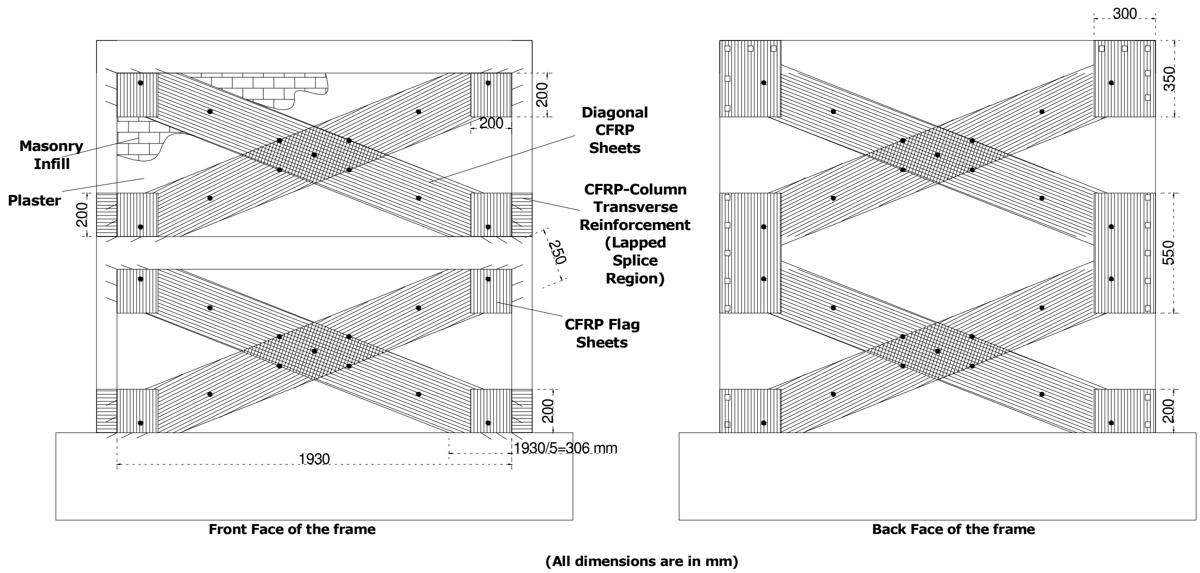


Figure 3. The CFRP strengthening applied on the frame.

A premature failure was observed in testing of the strengthened specimen of the first series (NSTR) due to the bond-slip occurring in the lapped splices in column longitudinal bars at the footing level during the early phases of the loading. As there was no visible damage, the test was stopped at this stage and the specimen was rehabilitated by removing the cover and welding the lapped longitudinal reinforcements at the exterior corners of each column to each others. This region was then filled up with repair mortar and wrapped by one-layer CFRP, over a height of 200 mm. This specimen was named as NREH. The properties of the test specimens are given in Table 1.

Table 1. Properties and design details of the test specimens.

Specimen	Type	Lap Length (mm)	f_c' (MPa)	f_m' (MPa)
LREF1	Bare	160	16.1	N/A
LREF2	Infilled	160	16.3	3.9
LSTR	Strengthened	160	16.7	3.9
NREF1	Bare	160	13.0	N/A
NREF2	Infilled	160	17.1	3.9
NREH	Rehabilitated	160	17.1	3.9
Material	Type	f_y (MPa)	f_u (MPa)	E (MPa)
Steel	Stirrup	268	398	210,000
	Longitudinal	405	605	205,000
CFRP		N/A	3,430	230,000
Epoxy		N/A	50	3,500

The Test Setup and Instrumentation

The test setup consisted of a strong floor, reaction wall, lateral and axial loading systems, out of plane displacement restraining frame, and an electronic instrumentation and data acquisition system, Fig. 4. The specimens were tested under reversed cyclic lateral loading. In all tests, a load-controlled loading scheme was adopted until the peak resistance was achieved. After this point the displacement-controlled loading scheme was adopted. Lateral loading was applied with a double acting hydraulic actuator. The capacity of the actuator was 600 kN in compression and 300 kN in tension. In all tests lateral load was divided into two by a steel spreader beam and applied both at the first and second story beam levels, so that two thirds of the applied load goes to the upper story. Axial load was applied by means of a vertical load distributing beam directly to the columns. The intensity of the axial load was kept constant throughout the test at

$0.10P_o$ (i.e. 10 percent of the column axial capacity, $P_{applied}$ = 2x30 kN). An electronic data acquisition system with control feedback was used to measure the level of applied load and the in-plane lateral displacements. Displacements were measured by means of strain gage based linear variable differential transducers (LVDTs). Two LVDTs were mounted on each floor level to measure the average story displacement. For infill specimens shear deformations on the brick infill, horizontal base slip, and frame base rocking was also measured by means of LVDTs. All measurements were relative to the frame foundation. Finally, strain gages were attached to the CFRP cross overlays to determine the strain level on the CFRP sheets. The instrumentation arrangement for each specimen may also be seen in Fig. 4.

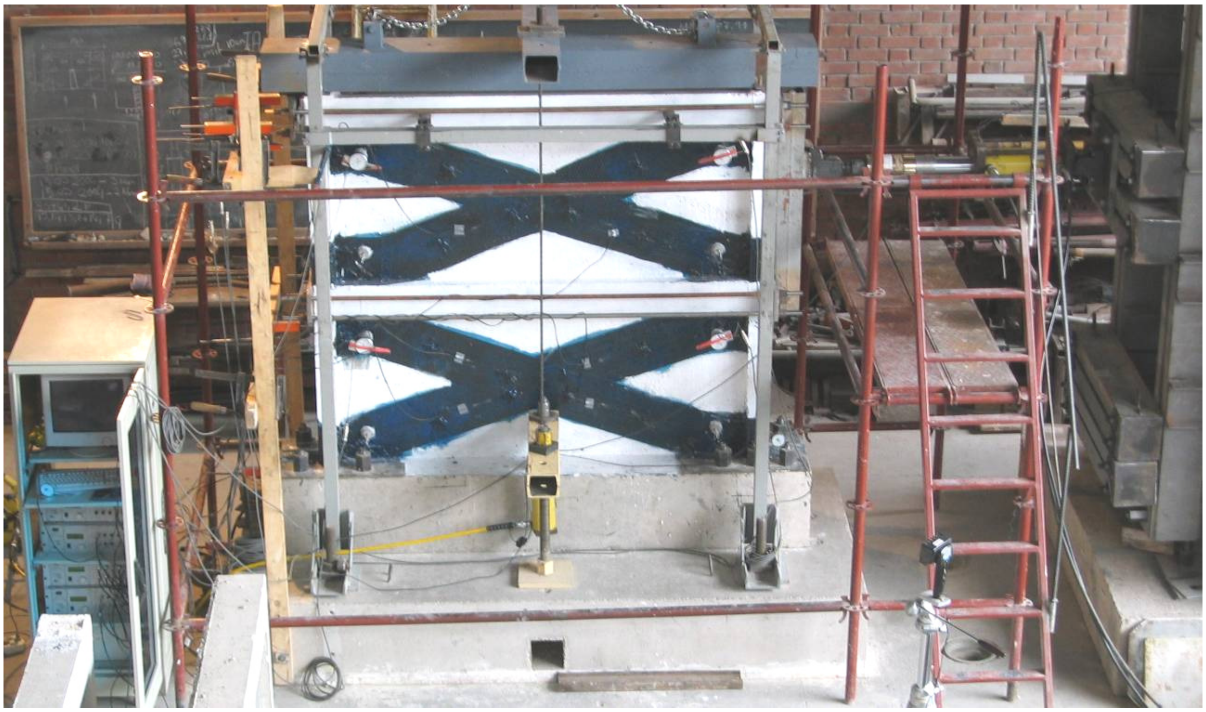


Figure 4. The test setup.

Observed Behavior of the Test Specimens

Series-L Tests

The first specimen tested in this group was the bare frame, LREF1, which was the first reference specimen. First flexural crack was observed on the tension side of the first story column in the 2nd positive cycle, when the applied lateral load was 8 kN. At a load level of 13 kN, specimen reached its ultimate load capacity. Failure was observed with the crushing of the cover concrete just above the foundation level (in the lapped splice region) and wide shear cracks at the beam-column joints.

The second frame was the specimen with infill walls, LREF2. This test was performed to assess the effect of the infill walls to the overall behavior. During the third cycle in the positive direction, at a load level of +40 kN, first flexural cracks were observed at the column-footing interface. In the 6th cycle, the specimen reached its lateral load resisting capacity at a load level of 70 kN. The specimen displayed similar resistances in the positive and negative loading directions. Under increasing lateral deflections, the infill wall acted as a diagonal strut, accompanied by the separation of the infill was observed on the opposite side. The corner crushing of the infill wall was observed at 2.67% interstory drift. After that stage, the response of the specimen rapidly deteriorated and approached the response displayed by the bare frame.

The last specimen of this series was Specimen LSTR, which was strengthened by CRFP diagonal strips.

The idea of the FRP retrofit scheme was to reduce the inter-story deformation demands by using FRPs to act as tension ties similar to a steel cross-brace configuration. During the test, the first hairline cracks were observed in the 5th positive displacement cycle, at a load level of 70kN, at the intersection region of the first story infill wall and foundation. The first visible flexural crack developed at the column-footing interface at lateral load level of 80kN. The performance of the CFRP anchorages and cross overlay sheets was quite satisfactory under the applied cyclic lateral loads up until the ultimate load level of 120kN, which was reached at a lateral drift ratio of 0.5 percent. After this stage, especially during the large lateral deformation cycles, the diagonal CFRP sheets experiencing large compressive deformations started to debond at the corners of first story infill walls. During the consecutive half cycle, the CFRP strips in the debonded regions started to split, which ceased the beneficial contribution of the CFRP reinforcement on the frame response. The CFRP sheets, which were used as lateral reinforcement at the lapped splicing region of the first story columns, became ineffective upon crushing of the cover concrete, and this event marked the end of the test. Figure 5 shows specimen LSTR at the end of the test.

Figure 5. The specimen LSTR at the end of the test.

Series-N Tests

In this group, the bare reference specimen, NREF1, was the first tested frame. Hairline flexural cracks were observed on the columns just above the foundation level at a load level of 6 kN. The lateral load capacity that the frame reached was very low, nearly 10 kN. Towards the end of the test, wide flexural cracks were formed in the lapped splice regions. The failure of the specimen was due to crushing of the core concrete at the bottom of the first story columns.

The second specimen was the infilled frame, NREF2, which serve as the second reference frame. At a load level of 19 kN, first cracks were observed on the columns above the lapped splice region. The maximum lateral load reached was 26 kN, after which decrease in stiffness started and a displacement based loading was applied. The contribution of the masonry walls to the system behavior continued until significant separation of the infill walls from the neighboring frame members took place. After this point, dispersion and widening of the flexural cracks on the first story columns and shear cracks at the beam-column joints were observed and the system response approached the bare frame response.

As explained previously, some unprecedented problems occurred during the third test of strengthened element in this series. It was not possible to transfer the load on frame members when the specimen started to rock about its base due to bond slip developing in the lapped splices. As there was no visible damage on the specimen, the test was terminated. The specimen was then rehabilitated by welding lapped reinforcement located at the exterior corners of the first story columns. These regions were then covered with repair mortar and the CFRP application was wrapped around columns ends for confinement. Once the repair operation was completed the specimen was re-tested. First cracks were observed simultaneously as a shear crack on the masonry wall and as flexural cracks on the columns, at a load level of 40kN. During the test the maximum load of 50kN was reached. Damage concentrated in the wall was at the wall- frame interface. Similar to LSTR, debonding of CFRP was observed at an interstory drift ratio of 2.87 percent. Following this event, significant damage started to accumulate at the bottom ends of the first story columns. At the end of the test, the specimen displayed a flexural failure by the crushing of the core concrete and buckling of the longitudinal reinforcement at the bottom ends of the first columns. Figure 6 shows the specimen at the end of testing.

Figure 6. The specimen NREH at the end of the test.

Discussion of the Test Results

In this section, strength, stiffness, and global drift characteristics of the test specimens were compared for the two series, separately. The test results, in terms of cracking load, maximum load, stiffness and top story displacements at the cracking, at the peak and at 85 percent of the peak load are shown in Table 2. The lateral load versus roof deflection hysteretic relationships of Series-L and Series-N specimens are shown in Fig 7a and 7b, respectively.

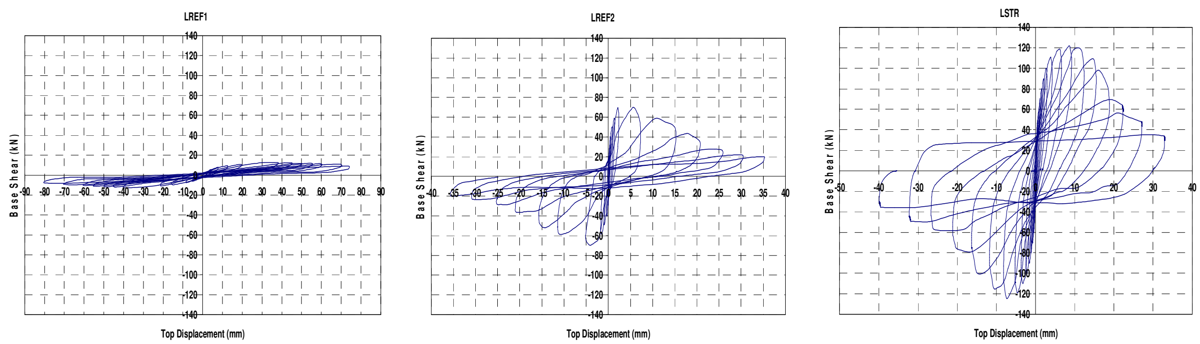
Table 2. Comparison of Test Results.

Spec.	P_{cr} (kN)	P_y (kN)	P_{max} (kN)	Δ_y (mm)	Δ_m (mm)	Δ_{85} (mm)	K^+ (kN/mm)	Δ_{85}/Δ_y (%)	Failure Mechanism
LREF1	4.8	11.0	12.8	21.5	38.2	75.4	0.88	3.5	Flexural failure
LREF2	29.17	-	70.0	-	2.2	10.0	51.4	4.6	Crushing of infill corners
LSTR	70.0	110	122.0	3.4	8.6	16.8	49.7	4.9	Debonding of diagonal CFRP sheets, crushing of infill corners
NREF1	3.0	9.2	9.8	16.2	27.1	60.3	1.48	3.7	Flexural failure
NREF2	9.4	26.0	25.4	5.6	7.9	30.2	10.19	5.4	Separation of infill and frame, crushing of infill corners
NREH	21.0	49.4	50	6.1	10	60.3*	19.68	9.9*	Debonding of CFRP sheets, crushing of the cover concrete of column

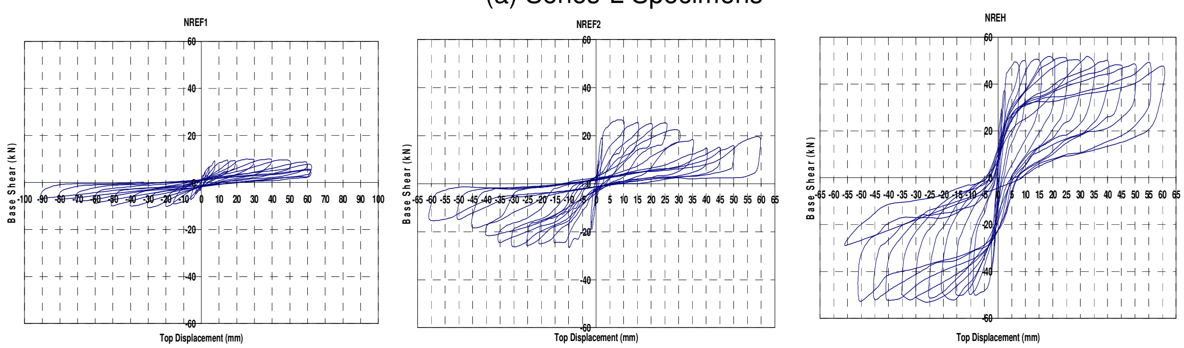
* This figure corresponds to 94 percent of the load at which the test was terminated

The envelope curves of the hysteretic relationships given in Fig. 7 were constructed, by connecting the maximum points of the hysteretic curves, for a better comparison of the test results. The envelope curves thus obtained are shown in Figure 8, for each series separately.

As can be seen in Figs 7 and 8, addition of the mortar applied masonry infill walls increased the base shear capacity of the bare frames considerably. The increase was 5.5 folds in the case of Series-L specimens and 2.6 folds in the case of Series-N specimens. CFRP application on the masonry infill walls (without removing the plaster) resulted in further increases in the lateral load capacity of the test specimens in each series. The base shear capacity of specimen LSTR was 1.74 times that of LREF2 and 9.53 times that of LREF1. Similar increases were observed in the case of Series-N specimens. The ratio of the base shear capacity of NREH to that of NREF2 and NREF1 were 2.0 and 5.1, respectively. These observations indicated that, regardless the h/w ratio of the masonry infill walls, the proposed rehabilitation technique considerably increases the base shear capacity of the frames. This capacity increase may be in the order of 97 percent in the case of slender specimens. For squat frames, however, the increase was about 74 percent. The readers should keep in mind that, in the case of specimen NREH, besides the CFRP application on the masonry walls, the lapped reinforcement located at the exterior corners of the first story columns were also welded together. The difference between capacity increases in Series-N and the Series-L specimens may be primarily due to this operation. To be able to assess the real effect of welding in increasing the base shear capacity, the authors are planning to perform an additional test in Series-L where welding of the lapped reinforcement will also be included in rehabilitation.



(a) Series-L Specimens



(b) Series-N Specimens

Figure 7. Load-Roof Deflection hysteresis curves of the test specimens.

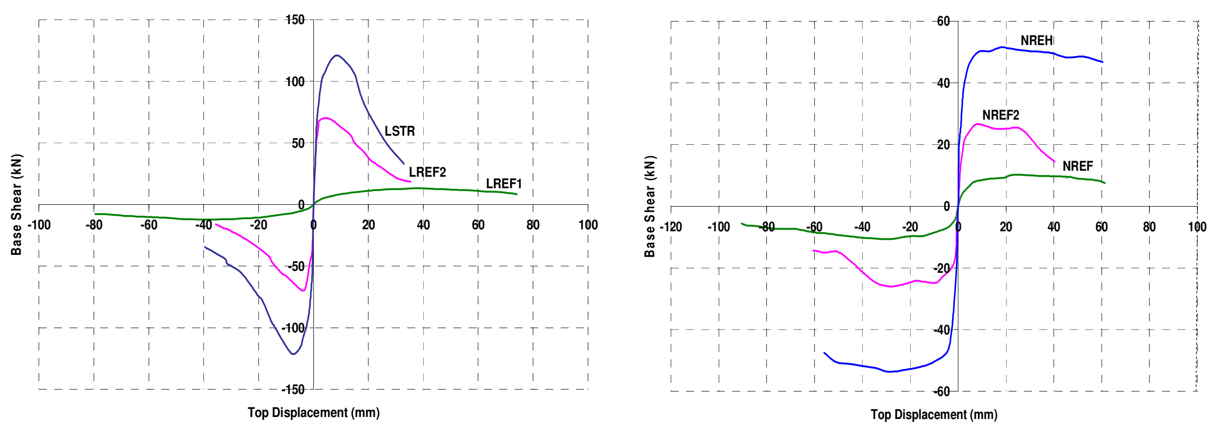


Figure 8. Envelopes base shear-roof displacement curves of the test specimens.

Stiffness comparisons are based on the envelope curves. The K^+ entries in Table 2 are the secant stiffness measured on the positive cycle envelopes. It indicates the slope of the line connecting the point corresponding to 60 percent of the ultimate load (on the ascending part of the curve) to the origin. In both series, plastered masonry walls increased stiffness of the bare frame considerably. This increase was more notable in Series-L than Series-N. In Series-L application of CFRP reinforcement on the masonry wall did not change the stiffness of the system. This indicates that reinforcing masonry infill walls with CFRP does not affect the stiffness of the original structure, but do increases the base shear capacity. This further indicates that the proposed rehabilitation methodology increases the base shear capacity of the system without increasing the seismic demand on the structure during a particular earthquake, as the stiffness characteristics of the building remain unchanged. In Series-N specimens, although an increase in system stiffness is attained due to the introduction of plastered infill walls, this increase was not as high as in the case of Series-L specimens. In this series, welding of the lapped bars located at the exterior corners of the first story columns, however, increased the system stiffness by nearly 100 percent. This increase was mainly due to the elimination of the loss of lateral stiffness resulting from column bar slip deformations.

The comparison of the envelope curves provided important information about the drift properties of the test specimens. Except NREH, where the lapped reinforcement was welded, all masonry infill frames displayed a behavior tending to bare frame response at large displacement amplitudes. NREH, on the other hand, displayed the most ductile response among all specimens. Welding of the lapped reinforcement prevented formation of large concentrated bond-slip rotations at the bottom ends of the first story columns and led to a flexure dominated response. The load-displacement hysteresis curves of NREH, however, displayed some pinching. This pinching may be due to partial welding of the lapped reinforcement. As pointed out earlier, only two of four column longitudinal bars were welded. Those located close to the masonry infill were not treated to eliminate bond-slip.

Table 2 gives top story displacements at yield, at the ultimate and at a load level corresponding to 85 percent of the yield load on the descending branch of the load – displacement hysteresis. In the same table the displacement ductility ratio, Δ_{85}/Δ_y is provided. A close inspection of these figures indicates that, for Series-L specimens, the CFRP intervention did not increase the system ductility significantly. Although the roof story drift at 85 percent of the peak load of the specimen LSTR increased by 68 percent, compared to that of LREF2, the displacement at yield also increases by the same amount. This proportionate increase resulted in no change in the ductility ratio. However, due to increased drift and base shear capacities, it is evident that the strengthened specimen has a superior energy dissipating property.

The rehabilitation applied in Series-N, however, significantly improved both strength and ductility properties of the test specimen. The specimen NREH had a capacity corresponding to 94 percent of its ultimate capacity when the test is terminated due to debonding of the CFRP after buckling of bars at first story column ends. This specimen achieved system displacement ductility as high as 10.

It should be noted that the strength and ductility improvements reported in this study are by no means superior than those can be achieved by using reinforced concrete infills. The test reported on RC infill frames pointed out much higher strength and ductility improvements, (Sonuvar 2001).

It is, however, believed that together with the strength improvement attained, the applied rehabilitation procedure led to improved seismic behavior. As this rehabilitation technique involved both welding of lapped reinforcement and CFRP application on the masonry walls in Series-N, at this stage it is difficult to comment on the effect of aspect ratio on the overall response. However, after applying a similar rehabilitation in Series-L (i.e. FRP rehabilitation followed by welding of column bar splices), the effect of this parameter on the performance of the FRP retrofit scheme will be more clear.

Conclusions

Conclusions summarized below are based on six 1/3 scale specimens tested. These conclusions should not be generalized without due judgment. Further experimental studies on larger scale, multi-bay specimens are needed. Such a testing program is being carried out at METU Structural Mechanics Laboratory.

- Tests have revealed that converting masonry infills into structural walls is possible by strengthening such non-structural members by CFRP sheets and strips connected to the frame members.
- As the application of this technique does not require the evacuation of the building under consideration, it seems to be a feasible and economical solution.
- After rehabilitation, the lateral strength of all specimens showed a significant increase under reversed cyclic loading. The strength increase was about 74 percent for Series-L specimens and 97 percent for Series-N specimens.
- The CFRP reinforcement did not change the system stiffness. Stiffness increase in Series-N specimens is attributed to welding of lapped bars.
- The drift characteristics of the test specimens improved considerably. The CFRP reinforced squat walls were able to undergo twice as large displacements when compared with frames having unreinforced masonry.
- A comparison of the test results reported here with the test results on frames with reinforced concrete infills revealed that the behavior of masonry infilled frames strengthened with CFRP is not as ductile as frames with reinforced concrete infills, (Sonuvar 2001).
- The rehabilitation applied in Series-N, however, led to higher improvements in the drift characteristics. In this series the system ductility of the unreinforced masonry infill frame increased by nearly 85 percent.
- In Series-L specimens, the CFRP sheets used as lateral reinforcement at the lapped splice regions of the columns prevented local failures at these locations. This conclusion cannot be extended to Series-N specimens. To overcome this deficiency welding of lapped reinforcing bars is essential.
- The comparison of the hysteresis curves indicates that the CFRP strengthening increases the energy dissipation capacity of the infilled frames significantly.

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