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REHABILITATION OF EXISTING REINFORCED CONCRETE STRUCTURES USING CFRP FABRICS

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SUMMARY

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. However, this procedure 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 enormous existing building stocks necessitate urgent 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, 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. All together seven specimens were tested. The arrangement of the CFRP layers, 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, [1-9]. 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

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rehabilitation is concerned. Therefore, a faster and easier method, that would not interrupt the use of the building, should be developed. In this context, an experimental study was initiated at the Middle East Technical University Structural Mechanics Laboratory.

The main objective of the experimental study reported here 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.

TEST SPECIMENS

Seven geometrically identical, one-bay, two-story RC frames with common structural deficiencies observed in the Turkish practice were constructed. The test specimen was a 1/3 scale model of a non-ductile frame having weak columns and strong beams. Insufficient confinement was provided at column and beam ends and no confinement was provided at beam-column joints. The ties used in beams and columns of the test specimens had 90 degree hooks at their free ends. 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 second story column longitudinal bars, made at the column base. Although the lap splice length was conforming to the Turkish Seismic Design Code [10] (forty times the diameter of the longitudinal bars), the transverse reinforcement provided in this region did not comply with the code requirements. Figure 1 presents the dimensions and the reinforcement details of the R/C frames. Upon the construction of the R/C frames, the hollow clay tile infills were constructed and plastered by a professional mason.

Prior to the test, six of the seven specimens were strengthened using one-directional CFRP sheets. One of the infilled specimens (SP-1) was not strengthened. This served as a reference specimen.

The infill of Specimen SP-2 was strengthened by covering both faces of the infills with two orthogonal layers of CFRP. This type of CFRP application which fully covers the infill was designated as, “Blanket Type”. The CFRP sheets did not extend to the frame members and were not connected to these members. The CFRP sheets were bonded to the infills by using a special adhesive as recommended by the manufacturer.

In Specimen SP-3 only the exterior face of the infill was fully covered by two orthogonal CFRP sheets (blanket type) which were also extended to the frame members. The CFRP sheets were bonded to the infill as in Specimen SP-2. The CFRP sheets were anchored to the frame members using special anchors developed. These anchors were made by rolling the CFRP sheets. The rolled CFRP anchors were folded and tied to a guide wire having a diameter of 1 mm, 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 anchors 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. Number, location and configuration of the anchors for specimen SP-3 are shown in Figures 2b and 2c.

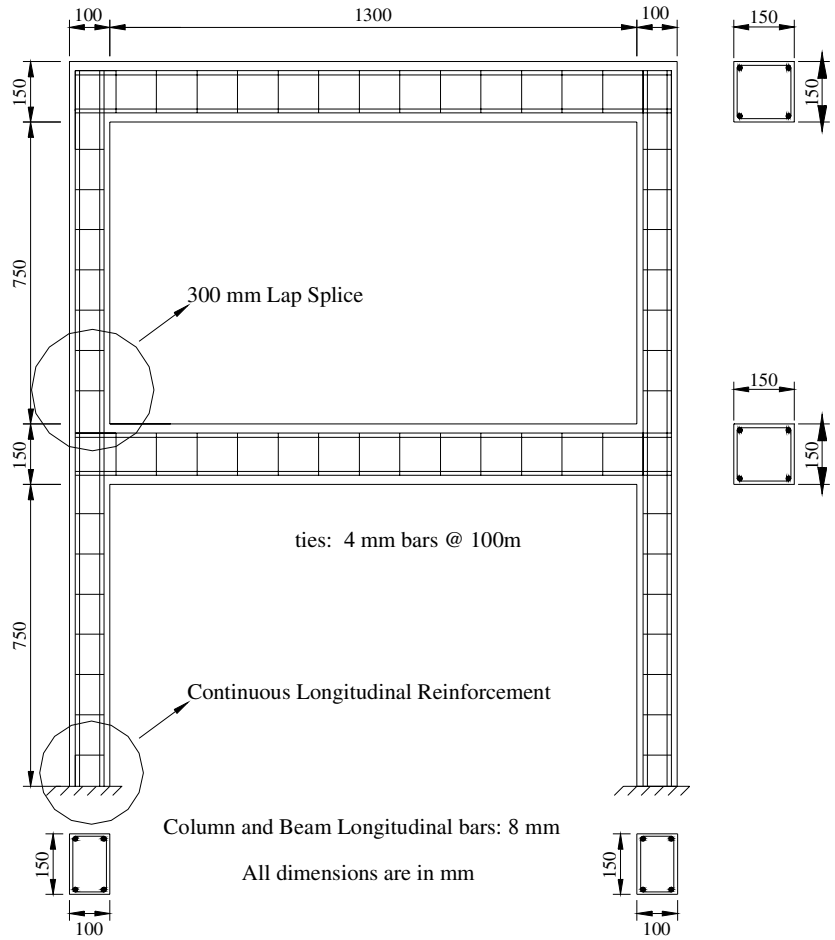


Figure 1 – Geometry and the Reinforcement Details of the Test Frames

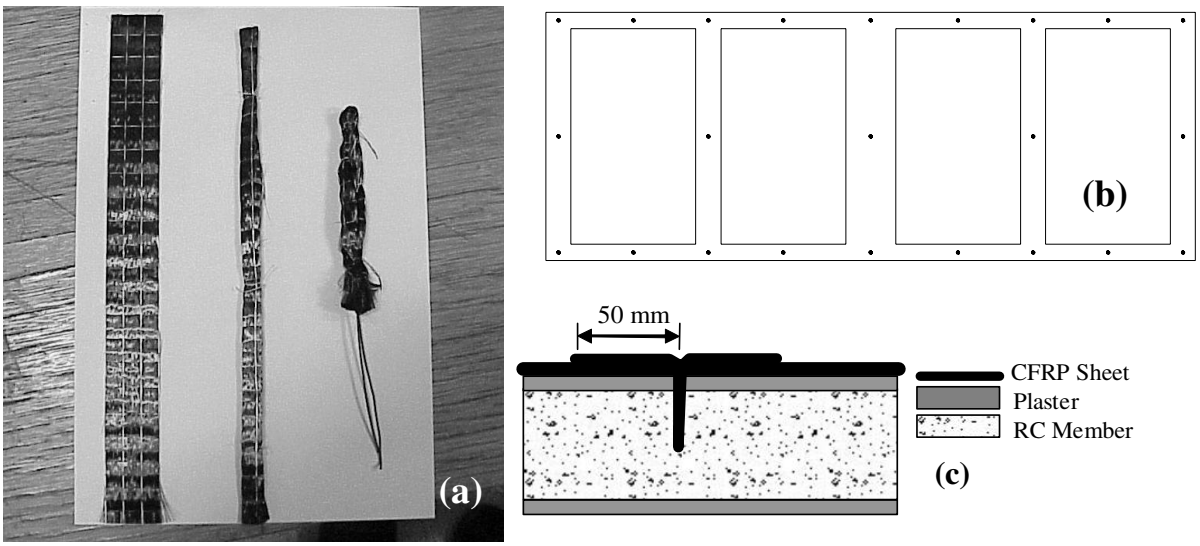


Figure 2 - (a) Anchors; (b) location and (c) configuration of anchors for Specimen SP-3

In SP-4, both sides of the specimen were fully covered by two orthogonal layers of CFRP. The CFRP sheets on the exterior face were extended and anchored to the frame members. On the interior face, the CFRP sheets were bent and anchored to the beam and column face as shown in Figure 3a. The anchor dowels used in SP-3 contributed to the behavior of the specimen significantly. However, it was observed that, the number of the dowels used in SP-3 was insufficient. Thus, the number of dowels used was increased in SP-4, Figures 3b and 3c. In Specimen SP-4, CFRP sheets covering the two faces of the infill were connected to each other by using CFRP anchors. Holes were drilled through the infill to place these anchors as shown in Figure 3d.

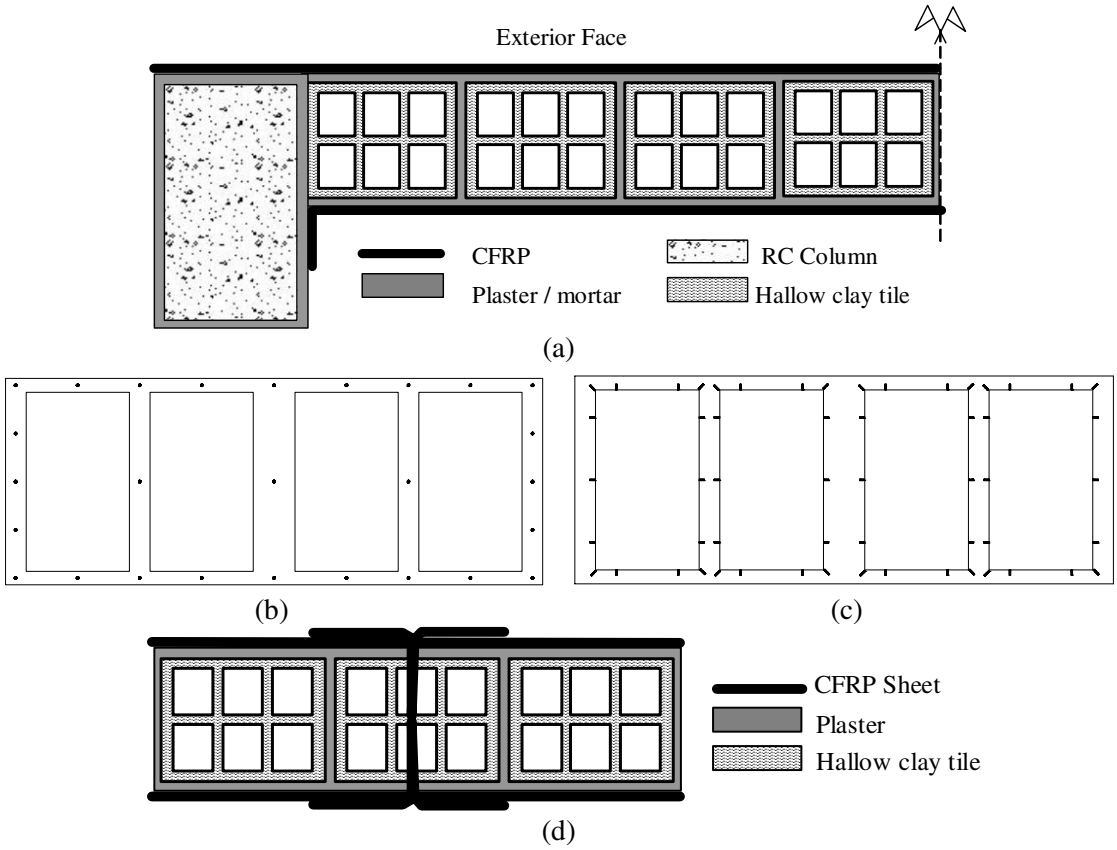
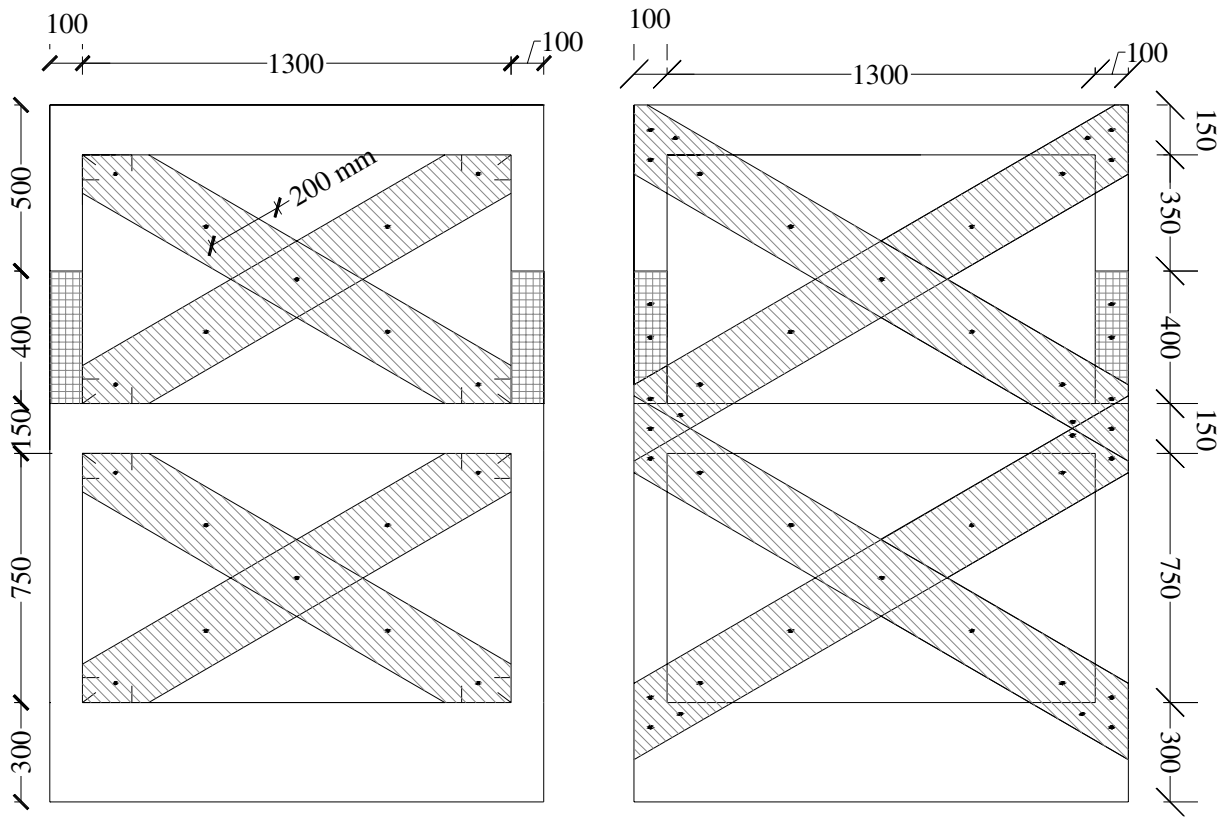


Figure 3 – Specimen SP-4: (a) CFRP pattern; anchors on the (b) exterior face; (c) interior face; (d) infill

Since a premature failure was observed in testing SP-3 due to the presence of lapped splices in column longitudinal bars at the second floor level, the lapped regions in SP-4 were confined by wrapping CFRP sheets extending along the lap length. The confinement was provided to prevent a premature local failure resulting from lapped splices was eliminated.

The CFRP strengthening in Specimen SP-4 resulted in significant strength increase as compared to the reference specimen SP-1. However the amount of CFRP used in this specimen would make this type of strengthening uneconomical for practical use. Therefore in Specimen SP-5 it was decided to use CFRP strips placed in two diagonal directions instead of fully covering the infill. This type of strengthening was called, “Strut Type”. The width of the strips used as diagonals was 200 mm. The configuration of the CFRP strips and the anchor dowels used are shown in Figure 4. The lapped spliced region of this specimen was also confined with CFRP sheets similar to SP-4.



(a) Interior face (a) Exterior face

Figure 4 – CFRP arrangement and location of the anchor dowels in Specimen SP-5 (dimensions are in mm)

The CFRP detailing used in SP-5 resulted in a significant increase in lateral load capacity and energy dissipation capacity. The failure of SP-5 was initiated by delamination of CFRP strips at the foundation level. Yielding of longitudinal reinforcement and crushing of concrete were also observed at this region. To prevent this local failure, in Specimen SP-6 the bottom of the first story columns were also confined using two layers of CFRP. The length of this confined zone at the base was 150 mm. The configuration of diagonal CFRP strips and the number and location of anchors in this specimen were identical with those of SP-5.

In SP-6, shear failure in the first story beam-column joints was observed. This failure was due to the sudden failure of the anchors used in these regions. Specimen SP-6 reached its capacity when shear cracks at beam-column joints widened following the failure of anchors at this region. To overcome this type of failure, the size of anchors used on the first story beam-column joints were increased in Specimen SP-7. The properties of the test specimens are given in Table 1.

TEST SET-UP AND INSTRUMENTATION

The loading system and the test specimen are shown in Figure 5. The test set-up is similar to one which was developed by Smith [11]. Twin specimens with a common foundation beam were constructed and laid upon the steel plates resting on the ball bearings. The foundation beam was properly constructed and heavily reinforced to prevent the local failures.

Table 1 – Properties of the Test Specimens

			SPECIMEN						
			SP-1	SP-2	SP-3	SP-4	SP-5	SP-6	SP-7
Longitudinal Reinf. (8 mm bars)		Yield Strength, (MPa)	388	388	388	388	388	388	388
		Ultimate Str. (MPa)	532	532	532	532	532	532	532
Transvers Reinf. (4 mm bars)		Yield Strength, (MPa)	279	279	279	279	279	279	279
		Ultimate Str., (MPa)	398	398	398	398	398	398	398
Number of Bars		Beams and Columns	4	4	4	4	4	4	4
		Foundation Beam	6	6	6	6	6	6	6
Detailing of Transverse Reinf.		Spacing (mm)	100	100	100	100	100	100	100
		Hook Angle (º)	90	90	90	90	90	90	90
Compressive Strength of Concrete, (MPa)			19.5	15.3	12.9	17.4	12.0	14.7	17.5
Compressive Strength of Mortar, (MPa)			4.3	4.3	3.1	2.9	4.1	4.2	4.3
CFRP	Infill	Application Side	None	Both	Ext.	Both	Both	Both	Both
		Type	-	Blanket	Blanket	Blanket	Strut	Strut	Strut
		Anchors	No	No	Yes	Yes	Yes	Yes	Yes
	RC Frame	Application Side	None	None	Ext.	Ext.	Ext.	Ext.	Ext.
		Anchors	No	No	Yes	Yes	Yes	Yes	Yes

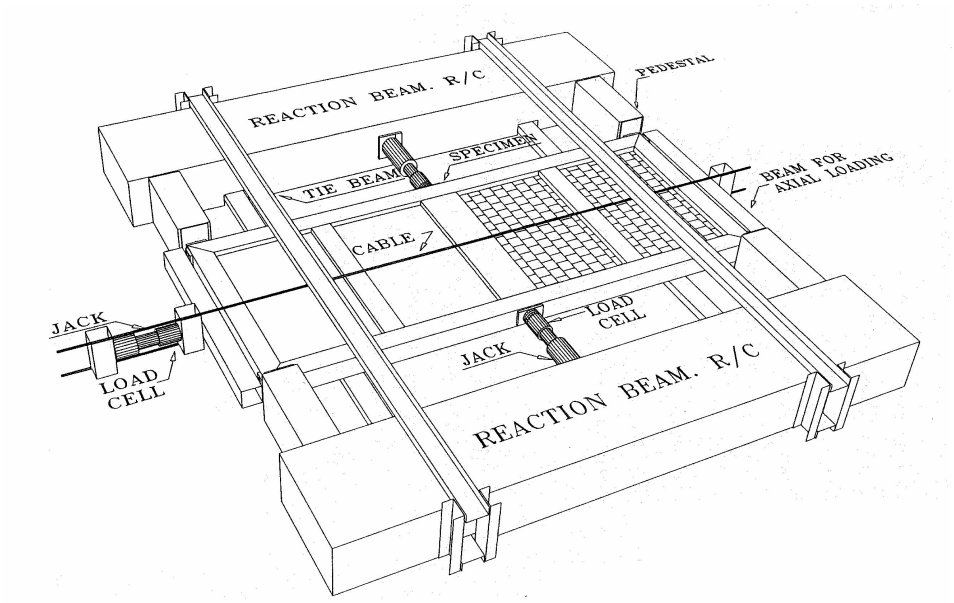


Figure 5 - Test Set-Up

The specimens were tested under reversed cyclic loading. The lateral load was applied through the foundation beam. Hence, the reaction forces at each end of the twin specimens become the lateral forces applied at the second story level. Axial load was applied to the columns by prestressing tendons as shown in Figure 5. The level of applied axial load on each column was about 25% of the nominal axial load capacity of the frame columns, $0.25N_o$.

Test specimens were instrumented to measure the applied loads, lateral displacements, rotations of the foundation beam and diagonal strains on the infill. Figure 6 shows the instrumentation.

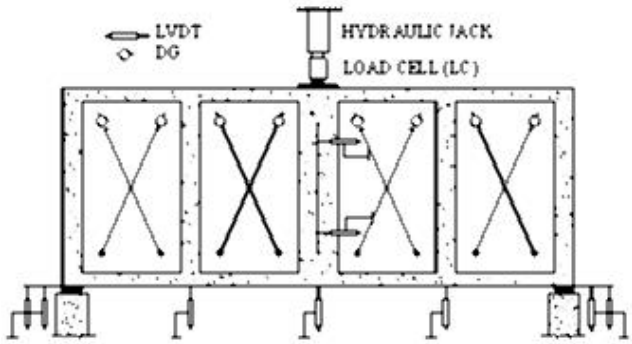


Figure 6 – Instrumentation

Lateral displacements and support settlements were measured by means of linear variable differential transducers (LVDT) which were mounted at each story and at the foundation beam levels. Two additional LVDTs were attached to the foundation beam to measure rotations. The infills were further instrumented with electrical dial gages (DG) placed diagonally to monitor the shear deformations. After each test, story displacements were calculated by making corrections considering both support settlements and rigid body rotations. Since each specimen consists of two parts, experimental results were presented only for the specimen where failure was observed.

BEHAVIOR OF TEST SPECIMENS

The first specimen tested (SP-1) was an unstrengthened, hollow clay tile infilled reinforced concrete frame. This specimen was constructed with the most common deficiencies observed in practice and it served as the reference specimen of this test series. As expected, it displayed a very poor behavior with limited lateral load and displacement capacities. The maximum lateral load resisted by this specimen was 55 kN. The specimen failed due to crushing of the first story beam column joint at the end of the seventh cycle.

The specimen SP-2 was strengthened by the application of two orthogonal CFRP sheets on each face of the infills. Since the CFRP sheets were neither extended nor anchored to the reinforced concrete frame members, delamination occurred at the early stages of the test. Therefore, no significant increase in the lateral load capacity was achieved as compared to specimen SP-1. Despite this unfavorable behavior, the inelastic displacement capacity and energy dissipation capacity of SP-2 were considerably higher than those of SP-1. The specimen failed due to crushing of the first story beam-column joints in the ninth cycle.

The exterior face of the masonry infills of the third specimen (SP-3) was fully covered by two orthogonal layers of CFRP sheets. Although, CFRP sheets were anchored to the frame members by special anchors, delamination of the CFRP layers on the second story infill panel was observed at the early stages of the

test. The failure of one anchor located at the mid-height of one of the second story columns had triggered this phenomenon. No delamination was observed on the remaining three panels. The delamination which took place on one infill out of four was probably due to poor workmanship in placing the anchors. This specimen resisted a maximum shear force of V = 65.4 kN. Although this level of lateral resistance was slightly higher than the one experienced by specimen SP-1, the strength increase was only 20 percent and it was not sufficient to ensure the required safety. The ninth cycle marked the end of the test. At the end of this cycle, the second story beam-column joints failed by crushing of concrete.

In SP-4, the CFRP sheets were applied on both faces of the masonry infills in two orthogonal directions (blanket type). The CFRP sheets were also extended to the frame members. Homemade CFRP anchors were used to fasten the CFRP sheets to the frame members and to the hollow clay tile infills. As a result of this strengthening scheme, a significant improvement of behavior was achieved. The lateral load carrying capacity of SP-4 was almost 135 percent higher (V = 131.5 kN) than that of SP-1. This load level was achieved in the twentieth load cycle. At this load level CFRP buckled at the edge of the first story joint and the previously observed cracks at the bottom of the first story columns widened significantly. In the twenty-second cycle, the maximum load was 125 kN in the positive half cycle and 127 kN in the negative half cycle. At this load level, the anchors at the bottom of the first story column failed and the CFRP was completely delaminated at the foundation level, which was then followed by crushing of concrete at the bottom of the first story columns. In the last cycle (i.e. cycle 23), the maximum loads were 104 kN and 75 kN in the positive and negative half cycles, respectively. Longitudinal bars of the columns buckled at the foundation level. A tie at this level ruptured due to buckling of longitudinal bars.

The failure of SP-4 was a flexural failure and this specimen displayed more ductile behavior as compared to the reference specimen SP-1. This specimen dissipated nearly five times more energy than SP-1.

In the design of CFRP detailing of SP-5, the main aim was to obtain a similar response as that of the specimen SP-4 by using less amount of CFRP. For this purpose the CFRP strips were placed on the infills in a cross-bracing configuration. The strips were anchored to the infills and to the frame members. This strengthening pattern led to remarkable savings in the CFRP material. The amount of CFRP used in SP-5 was only one fourth of the CFRP used in SP-4. In spite of this reduction in the amount of CFRP used, SP-5 resisted a lateral load of 118.8 kN (113 percent increase as compared to SP-1). The lateral load capacity of SP-5 was about 11 percent less than that of SP-4. Moreover, the energy dissipation characteristics of specimens SP-4 and SP-5 were very similar.

The failure of Specimen SP-5 was triggered by buckling of the CFRP material in the compression strut. This phenomenon was then followed by failure of the anchor dowels at the foundation level. In the twentieth cycle, plastic hinges at the bottom of the first story columns were fully developed and the specimen failed due to crushing of concrete in these regions.

The strengthening pattern used in SP-5 was economically feasible and resulted in a considerable improvement in the behavior and strength of the frame. For this reason it was considered as being satisfactory and the strengthening patterns used in the remaining two specimens were slight modifications of this pattern. In SP-6 the only difference was the confinement of the plastic hinge zones at the base of the first story columns by CFRP. This modification did not improve the behavior and the strength of the specimen. While in the fifteenth cycle when the lateral load was 100 kN, the anchors of the tensile strut failed suddenly and the tensile load carried by this strut was released. As a result of redistribution of forces within the system, the load on the compressive strut increased significantly, which led to a sudden and brittle shear failure in the first story beam-column joint.

To prevent the shear failure observed in the first story beam-column joints of SP-6, these zones were strengthened in SP-7 by increasing the size of anchors. However, this modification did not improve the behavior of this specimen. The testing was stopped when the first story beam-column joints of the specimen failed in shear. In general, specimens SP-6 and SP-7 displayed similar strength, ductility and energy dissipating characteristics.

The experimental hysteretic load-deflection relationships of the test specimens and their views at the end of the tests are presented in Figure 7.

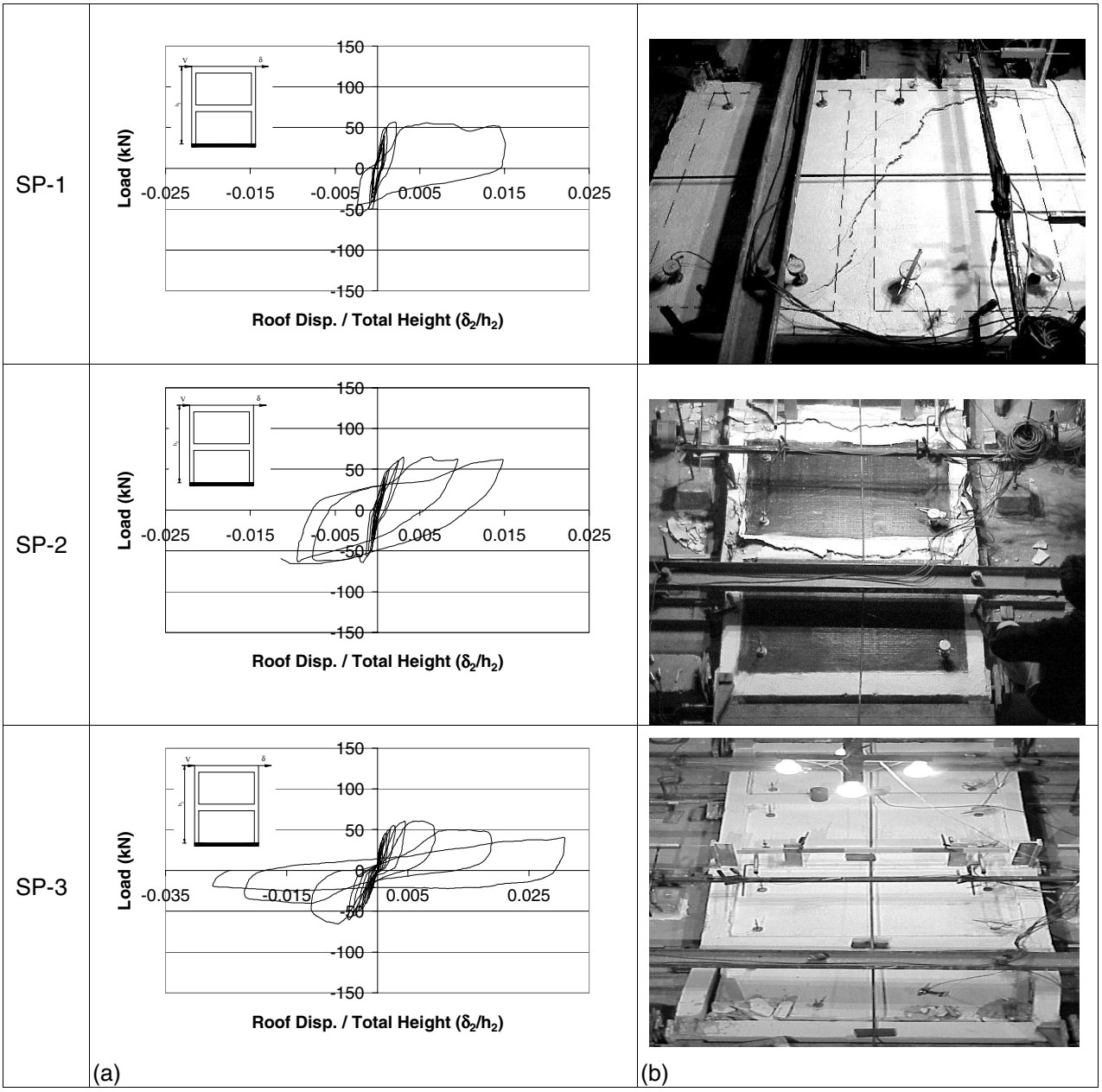


Figure 7 – (a) Load vs. roof drift ratio hysteretic relationships; (b) Specimens at the end of the test.

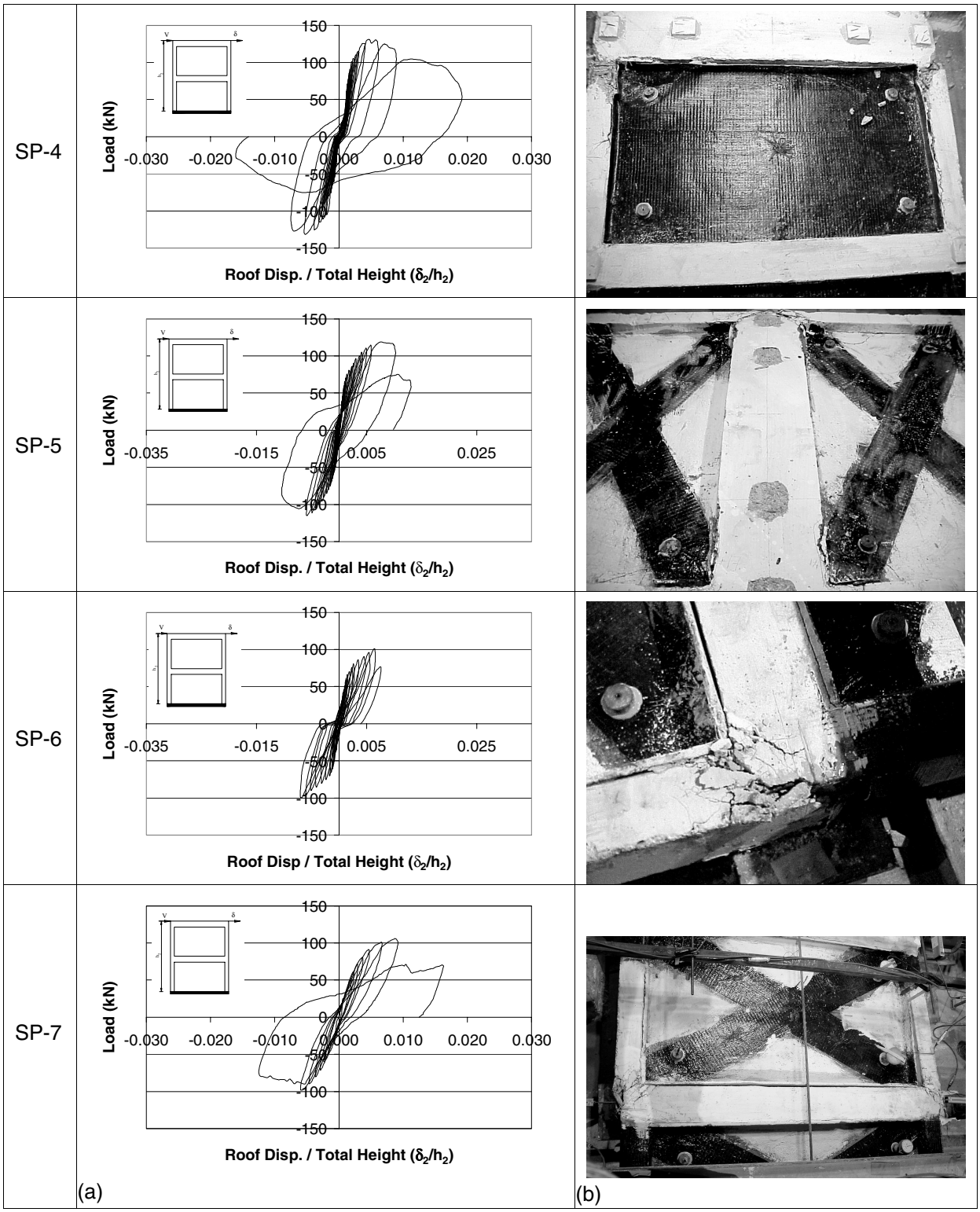


Figure 7 (cont'd) – (a) Load vs. roof drift ratio hysteretic relationships; (b) Specimens at the end of the test.

The envelope curves for all the specimens tested are presented in Figure 8. The test results are also summarized in Table 2. As can be seen in Figure 8 and Table 2, specimens SP-4 and SP-5 displayed significantly superior behaviors as compared to the other specimens. Although, SP-4 behaved better than SP-5, the amount of CFRP used in that specimen was not economical at all. Thus, it can be stated that, the CFRP detailing used in SP-5 seems to be the most efficient strengthening scheme tested within the scope of this study as far as economy and the ease in application are concerned.

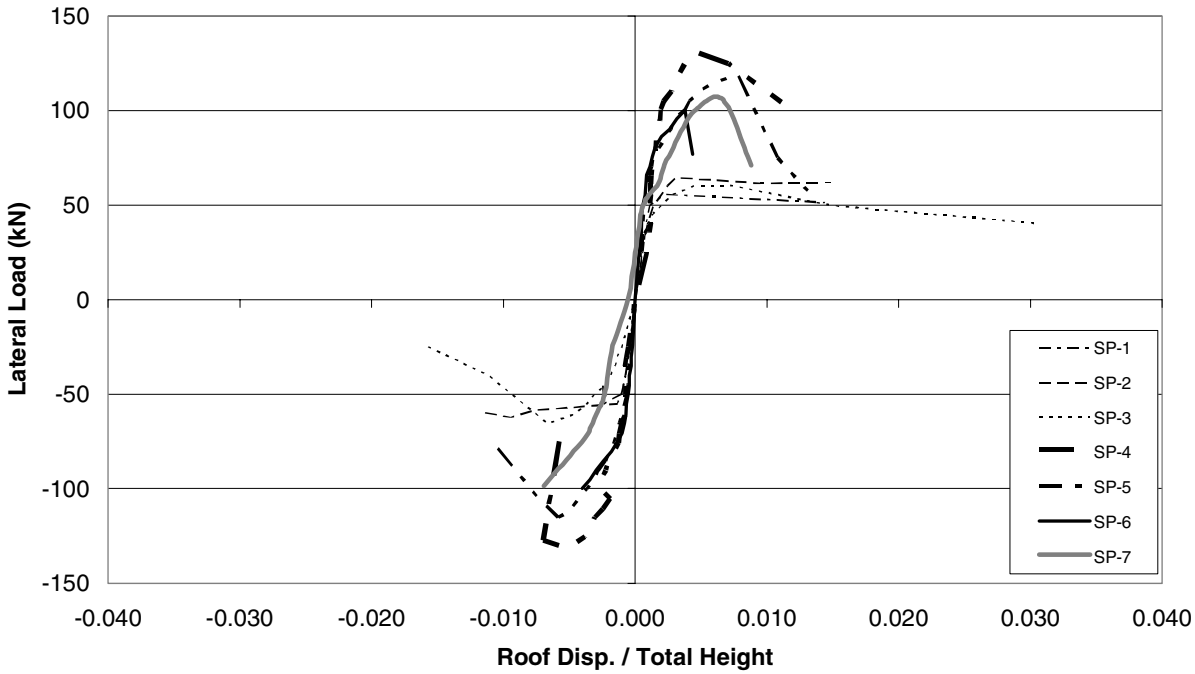


Figure 8 – Response Envelope Curves

Table 2 – Summary of Test Results

Specimen	Max Load Lateral (kN)	Initial Stiffness (kN/m)	Total Energy Dissipation (kJ)
SP-1	55.8	29 660	2.5
SP-2	64.6	29 520	6.1
SP-3	65.4	21 820	8.7
SP-4	131.5	36 430	11.1
SP-5	118.8	39 604	7.8
SP-6	100.4	32 624	4.2
SP-7	105.7	24 392	4.0

CONCLUSIONS

Conclusions summarized below are based on seven 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 of the frame members.
- Covering both faces of the infills by CFRP sheets anchored to the frame members (blanket type) increased the strength significantly. The lateral load capacity of SP-4 was more than twice that of the unstrengthened specimen (SP-1).
- Strengthening made using diagonal CFRP strips connected to the frame members seems to be a feasible and economical solution (SP-5). Although the amount of CFRP used in SP-5 was about ¼ of that of SP-4, the strength was only slightly less. The ratio of lateral strength of SP-5 was about twice that of the unstrengthened specimen SP-1.
- Test results indicated that the CFRP strengthening increased the energy dissipation capacity of the infilled frames significantly.
- During the tests on strengthened specimens, no damage was observed on the infill even at high interstory drift ratios (about 0.01).
- Among the common deficiencies observed in existing buildings, lapped splices in column longitudinal bars made at the column base seem to have the must adverse effect on behavior. Tests indicated that wrapping the lapped region with CFRP strips eliminated this adverse effect.
- Observation made during the tests showed that the effectiveness of CFRP strengthening highly depends on the anchors which connect the CFRP to the frame members. Workmanship is extremely important in making and placing the CFRP anchors.
- A comparison 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.

ACKNOWLEDGEMENTS

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