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EXPERIMENTAL AND ANALYTICAL STUDIES ON THE STRENGTHENING OF RC FRAMES

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SUMMARY

Due to various deficiencies and inadequate lateral stiffness many reinforced concrete buildings are highly damaged or collapsed in Turkey during the last major earthquakes. To improve the behavior of such buildings and to prevent total collapse, necessary amount of strengthening must be provided. To investigate and to compare the two types of strengthening techniques of reinforced concrete frames, namely introducing an infill RC wall, and CFRP strengthened hollow clay tile wall, two 1/3 scaled, 2-story, 3-bay test specimens are prepared. The frame of the specimens is detailed such that it has the common deficiencies of existing buildings in Turkey. The test specimens are subjected to reversed cyclic quasi-static loading. By means of special transducers, axial force, shear force and bending moment at the base of the exterior columns are measured. Strength, stiffness, and story drifts of the test specimen are determined. Analytical studies are also conducted. Top displacement response envelope curves of the both specimens are verified with the analytical pushover results obtained using Ansys 7.0

INTRODUCTION

Many residential buildings suffer high damages during major earthquakes. One of the main reasons for this damage is the inadequacy in lateral stiffness of buildings. Extensive researches have been conducted to develop methodologies to provide additional lateral stiffness to existing buildings. Most commonly used techniques are infilling of selected frames, bracing and jacketing of frame members. Since most of the residential buildings in Turkey are reinforced concrete researches have been mainly focused on strengthening of RC frames.

Strengthening of RC frames by reinforced concrete infills was first suggested by Ersoy and Uzsoy [1] in Turkey after performing tests on one-story, one-bay infilled RC frames. The authors observed significant

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improvement in lateral strength and stiffness by introducing RC infills. The authors also carried out analytical studies to verify test results.

Higashi and Kokusho [2] investigated the difference between the effects of monolithically cast RC infills and precast RC infills to RC frame. The authors observed that the lateral strength of the two infilled frames were almost the same. The researchers emphasized the importance of the connection between precast infills to the frame. In the same context Sugano and Fujimara [3] investigated several kinds of strengthening methods by testing ten one-story, one-bay RC frames. The researchers compared the strength, stiffness and energy dissipation capacity of each frame.

Canbay et al. [4] investigated the effects of introducing RC infill on a pre-damaged 1/3-scaled, two-story, three-bay RC frame. To evaluate the contribution of the RC infills, the authors used two special force transducers at the base of the exterior columns. The authors stated the importance of steel anchors and the effect of possible slip of longitudinal bars in columns.

Although these studies have shown that introducing reinforced concrete infills is a very effective technique, its application to the inhabited existing structures seem to be not feasible. That is why the application of this technique in Turkey was limited to the lightly or moderately damaged buildings which were unoccupied after the earthquakes. However, in Turkey and elsewhere, it is known that most of the existing buildings have seismic deficiencies which make them vulnerable. Methods and techniques have to be developed which will enable strengthening with minimum disturbance to the occupants.

The application of CFRP, compared to inserting RC infill wall, is faster, more practical, giving minimum disturbance to the occupants. The strengthening using CFRP was initiated on bridge girders, piers, columns of structures and masonry walls. Pantelides et al. [5] investigated the improvement in the capacity of bridge piers strengthened with CFRP by performing several in-situ tests. The authors also observed a significant ductility increase. Triantafillou [6] studied in-plane and out-of-plane behavior of the masonry walls which were strengthened with externally bonded FRP laminates by performing several tests. The author reported the failure modes of the test specimens. FRP applied on unreinforced masonry walls were also investigated by Albert et al [7]. The researchers performed several tests by changing either the type of FRP or pattern of FRP or type of loading. Debonding of FRP sheets and slip of FRP anchors are the important problems encountered during the application of FRP. Ueda et al. [8] investigated bond strength characteristics of the continuous fiber sheet by performing pullout tests.

To develop a strengthening method using CFRP, a research was initiated at Middle East Technical University by Ozcebe et al [9]. Using CFRP, hollow clay tile infills of two-story, one-bay RC frames were strengthened. The researchers performed seven tests to investigate the effects of pattern and amount of CFRP sheets. The test results revealed that, CFRP strengthened infills increased the lateral capacity of the frame without significant increase in stiffness.

In this study two different strengthening techniques are investigated both experimentally and analytically. These strengthening techniques are strengthening with RC infill and with CFRP strengthened hollow clay tile infill. Two-story, three-bay, 1/3-scaled RC frames were used in this study to investigate the behavior of RC infilled frame and that of CFRP strengthened hollow clay tile infilled frame. Pushover analyses were performed to verify the test results.

EXPERIMENTAL PROGRAM

The two-story, three-bay RC frame specimens used in this study were same as those tested by Canbay et al. [4]. Only the middle bay of the frames was infilled. One of the frames was strengthened with RC infills and it was designated as Specimen S1. The second frame was strengthened with CFRP applied on the existing hollow clay tile infills and was designated as Specimen S2.

After introducing the infill in the middle bay, the two interior columns and the infill behaved like a structural wall. Therefore the specimen was composed of a structural wall and two exterior columns. To observe the distribution of lateral load between the structural wall and the exterior columns, special force transducers developed by Canbay et al. [4] were used at the base of the exterior columns.

Reinforced Concrete Frame

The reinforced concrete frame was designed to reflect the common deficiencies observed in many RC buildings in Turkey. Some of these deficiencies are: poor lateral strength, splices made above floor levels with inadequate lap length, insufficient lateral reinforcement at member ends, 90^o hooks at the end of ties and very poor concrete quality.

The RC frame was cast vertically on a rigid foundation. All the columns were 110×110 mm and all the beams were 110×150 mm. The schematic view of the RC frame is shown in Figure 1. In Figure 2, details of the columns and the beams are shown. Four eight-millimeter plain bars were used in the columns which resulted in a steel ratio of 1.7%. Four-millimeter plain bars spaced at 100 mm were used as ties. Longitudinal bars of the interior columns were lapped spliced above the floor level. Lap length was equal to 40 bar diameter. Longitudinal bars of the exterior columns were welded onto the force transducers. In the beams, four eight-millimeter plain bars used. As stirrups, four-millimeter plain bars were spaced at 100 mm in the beams. A compressive concrete strength of 10 MPa which is the common compressive strength of concrete in existing buildings in Turkey was aimed for both stories.

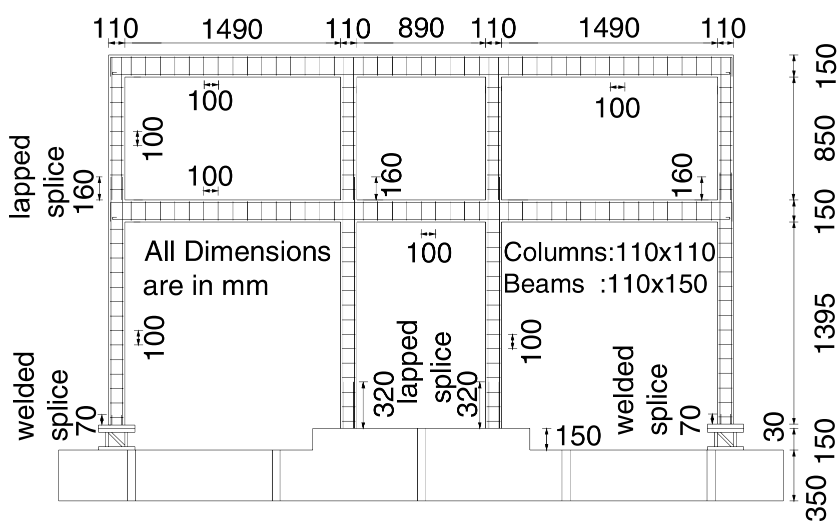

Figure 1. Dimensions and Reinforcement Details of the Frame (Dimensions are in mm)

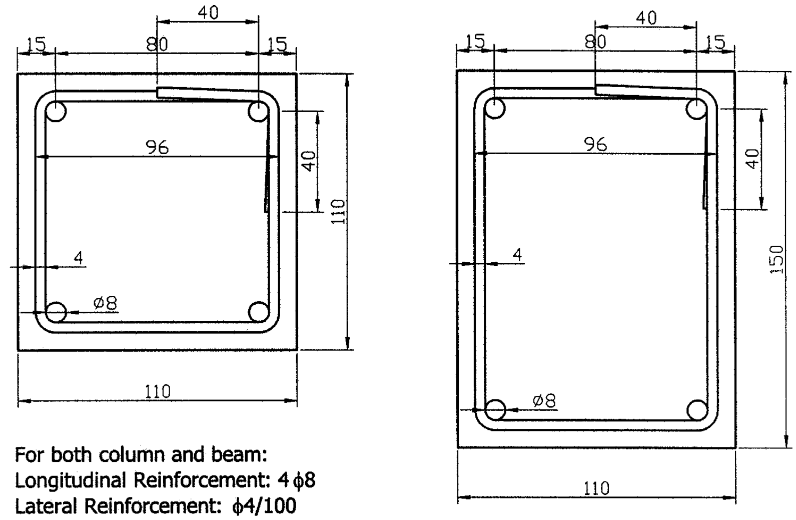

Figure 2. Column and Beam Cross-Section

Specimen S1

Specimen S1 was strengthened by introducing 70 mm thick RC infills in its middle bay. The connection of the infills to the frame members were achieved by steel anchor dowels. For this purpose 12 mm diameter holes were drilled in the foundation and in the frame members. Epoxy was injected to these holes and 10 mm diameter deformed anchor bars were inserted. The depth of the steel anchor dowels in the foundation was 150 mm and that in the frame members was 100 mm. The configuration of the steel anchor dowels is shown in Figure 3.

For the infills, two layers of reinforcement meshes were prepared using 6 mm diameter plain bars. The spacing of these bars was 150 mm in both horizontal and vertical directions. The reinforcement ratio was about 0.0055 in each direction. Concrete compressive strength of 30 MPa was aimed for the RC infills.

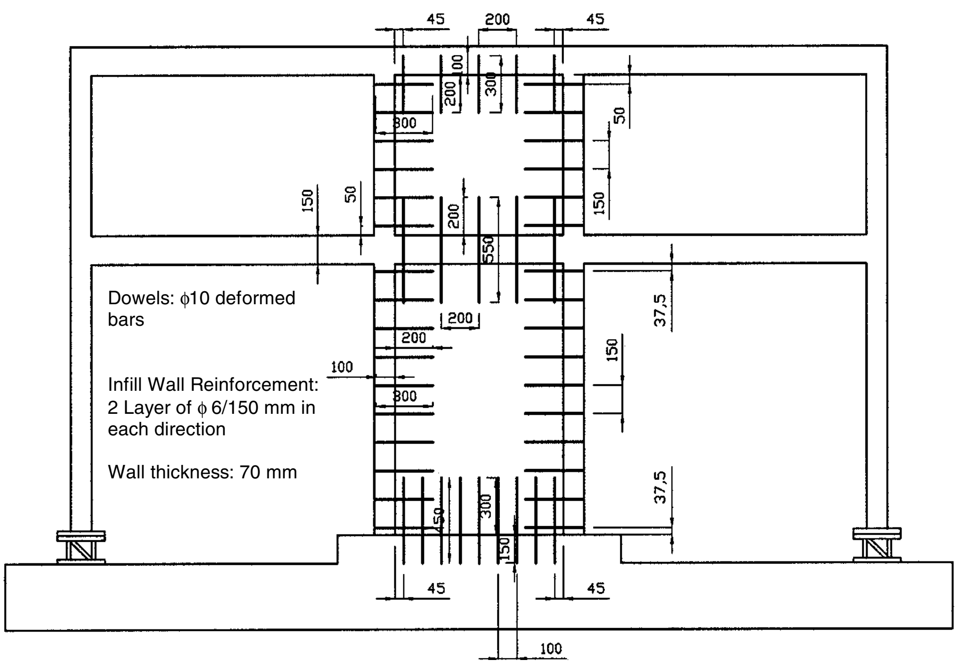

Figure 3. Steel Anchor Dowels (All dimensions are in mm)

Specimen S2

Specimen S2 was strengthened with diagonally placed CFRP on hollow tile infills. The hollow clay tile infills were constructed first. Due to the scale of test specimens, 1/3-scaled hollow clay tiles were used. CFRP sheets were applied diagonally on both sides of the infills. Since the bond between hollow clay tile infill to concrete is very weak, the CFRP sheets were extended and anchored to the frame members to provide better connectivity between them.

The configuration of the CFRP sheets is shown in Figure 4. The dots in this figure show the locations of the CFRP anchor dowels. CFRP anchor dowels were prepared from 50 mm wide carbon fiber sheets. The depth of the CFRP anchor dowels in the frame members was 70 mm. The CFRP anchor dowels were inserted into 8 mm diameter epoxy filled holes. As can be seen in Figure 4, CFRP anchor dowels were also used in the hollow clay tile infills to prevent debonding of the CFRP sheets. The CFRP anchor dowels in the infills connected the infill and the CFRP sheets on both sides. The CFRP anchor dowels used in the infills and frame members are shown in Figure 5.

Local strengthening was made at the spliced regions of the interior columns by wrapping this region by CFRP sheets. Before applying the CFRP sheets, corners of the columns in this region were rounded in order to minimize the stress concentration. Then these regions were wrapped with short CFRP sheets. The length of these wrapped portions were 500 mm and 300 mm in the first and second stories respectively. Since the CFRP used in this study is unidirectional, two orthogonally placed sheets were used to provide a proper confinement.

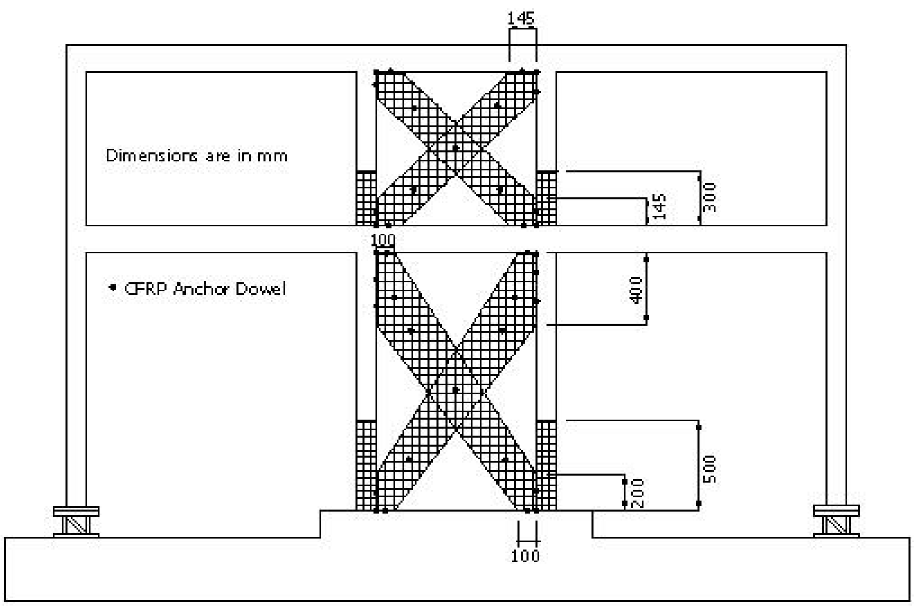

Figure 4. Configuration of the CFRP and Location of Anchor Dowels

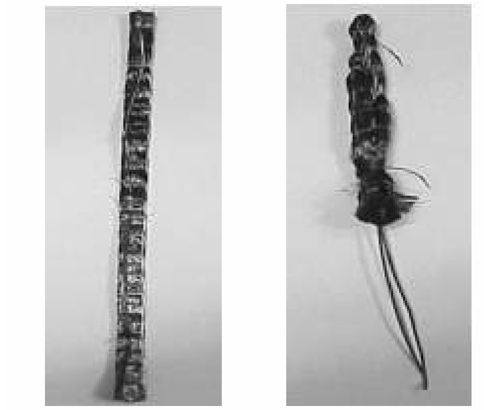

Figure 5. Anchor Dowel in the Infill (Left), Anchor Dowel in the concrete members (Right)

Materials

Plain bars which are the common type of reinforcing steel in most of the existing buildings in Turkey were used as reinforcing steel. The properties of the reinforcing steels are given in Table 1. The achieved average compressive strengths of the concrete used in the specimens are given in Table 2. Average compressive strength of the mortar used in the construction of the masonry walls of Specimen S2 was found to be 2.6 MPa. The average compressive strength of the hollow clay tile in the direction of its holes was calculated as 7.8 MPa considering the gross area of the clay tile. The ratio of net area to the gross area of the 1/3-scaled clay tiles was about 0.5.

CFRP sheets were composed of an epoxy-based matrix and carbon fiber reinforcement. The carbon fiber used in this study was unidirectional. The characteristic tensile strength and elasticity modulus of the carbon fiber are 3,430 and 230,000 MPa, respectively. To determine the tensile strength of CFRP, coupon tests were conducted at METU (Ozcebe et al. [9]). The average tensile strength of the CFRP sheet was found to be 800 MPa, and its ultimate strain was obtained as 1.25 percent.

Table 1: Properties of Reinforcing Bars

Type	Diameter (mm)	f_y (MPa)	f_{su} (MPa)	Property
Column & Beam Longitudinal	8	407	488	Plain
Column & Beam Transversal	4	322	422	Plain
RC Infill Mesh *	6	378	484	Plain
Steel Anchor Dowels *	10	455	568	Deformed

* Used in Specimen S1

Table 2: Concrete Strength

Strengthening technique	Specimen	28 day f'_c (MPa)	Test day f'_c (MPa)
RC Infill	S1 1st Story	10.9	13.6
RC Infill	S1 2nd Story	10.7	11.8
RC Infill	S1 Infill Wall	30.4	32.2
CFRP	S2 1st Story	9.8	11.2
CFRP	S2 2nd Story	9.1	9.6

Test Setup and Instrumentation

Both test specimens were tested in the vertical position. A two-way hydraulic ram mounted to a rigid reaction wall was used to apply the lateral load. Quasi-static reversed cyclic lateral load was applied only at the second story level. Out of plane displacement of the test specimens was prevented by an auxiliary steel frame. Additional vertical load of 9 kN was applied on the top of each second story column prior to testing the specimen.

LVDT displacement transducers were used to determine the lateral displacement at each story levels. Two force transducers were placed at the base of the exterior columns to measure axial force, shear and moment. In addition, 12 electronic displacement gages were mounted on the specimens to evaluate the shear deformations of infills and to measure the curvatures at the base of the exterior columns. A load cell was used to measure the applied lateral load. The instrumentation and the test setup are shown in Figure 6.

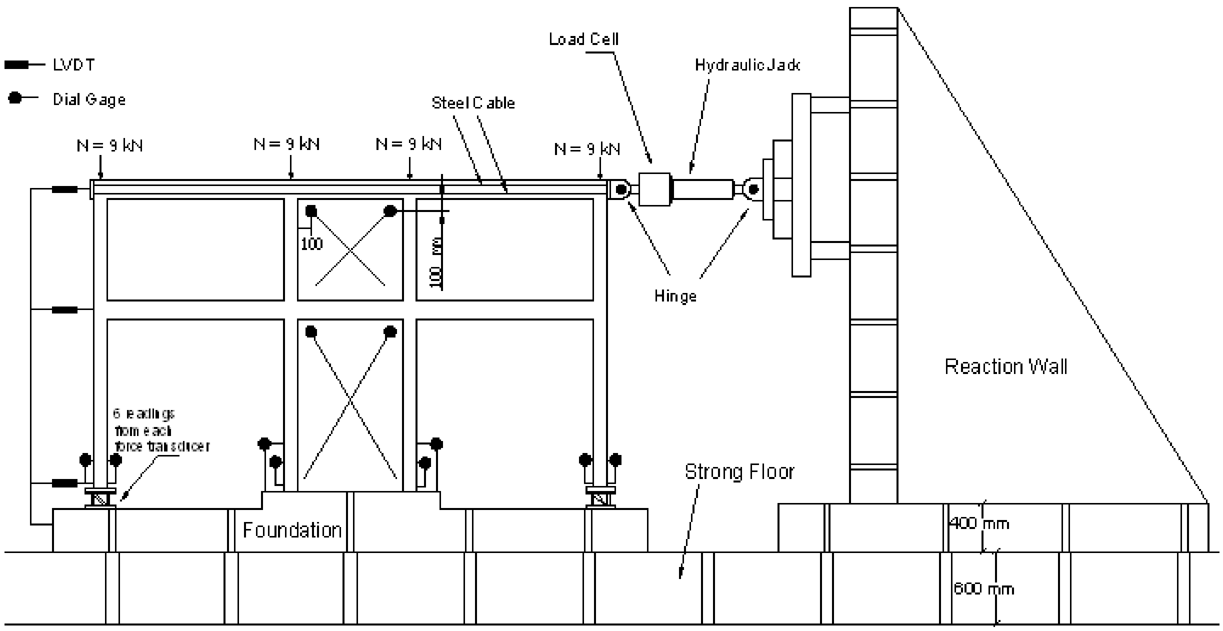

Figure 6. Instrumentation and Test Setup

BEHAVIOR OF TEST SPECIMENS

Specimen S1

The behavior of the specimen was dominated by the flexural action of the infill wall. Almost total lateral load (about 90 percent) was carried by the RC wall. Therefore, the behavior of Specimen S2 was very similar to that of a cantilever beam. Since almost total lateral load was carried by the infills, the most significant damage was concentrated on the infills at the end of the lapped splice region. The test was terminated when the decrease in strength was 24 percent due to heavy damage in the RC infill. Pinching of hysteresis loops was observed towards the end of the test due to high deformations causing slip of the longitudinal bars of the boundary columns. At the end of the test, the first story had reached a drift ratio of 2.83 percent. The ratio of the top displacement at the end of the test to the top displacement at the maximum load was measured as 5.7 in the positive direction and 4.8 in the negative direction.

Specimen S2

The behavior of Specimen S2 was similar to that of Specimen S1. The diagonally placed CFRP sheets on the hollow clay tile infill made it to behave like a structural wall. The failure of the specimen was due to failure of the CFRP anchor dowels at the foundation level. The lateral load at which the CFRP anchor dowels failed was the maximum lateral load reached during the test. After the failure of the CFRP anchor dowels, a few more cycles were applied. When the strength of the specimen reduced considerably (40 percent) the test was terminated. At the end of the test, the first story had reached a drift ratio of 0.9

percent. The ratio of the top displacement at the end of the test to the displacement corresponding to the maximum lateral load was calculated as 2.22 in the positive direction and 1.43 in the negative direction.

DISCUSSION OF EXPERIMENTAL RESULTS

The comparison of the behavior of test specimens is made in terms of lateral strength, stiffness and story drifts. To see the improvement provided by the strengthening, the results of the bare frame test will also be discussed briefly. Bare frame test was conducted by Canbay et al. [4].

Table 3: Comparisons of the Test Results

Characteristics	Bare*	S1	S2	S1/Bare	S2/Bare	S1/S2
Lateral Strength (+) (kN)	14.0	70.6	64.7	5.0	4.6	1.1
Lateral Strength (-) (kN)	13.0	70.6	70.3	5.4	5.4	1.0
Initial Stiffness (kN/mm)	2.1	30.0	22.5	14.3	10.7	1.3
Drift Ratio At Max. Load (%)**	1.9	0.9	0.7	0.5	0.4	1.3

*Taken from [4]
**Displacement divided by the story height

To evaluate the lateral strength and the stiffness of the test specimens, envelope curves were constructed. Response envelope curves of Specimens S1 and S2 are given in Figure 7 together with the response envelope curve of the bare frame. As can be seen from this figure, strength and stiffness of both strengthened frames were significantly higher than those of the bare frame. It is seen that Specimen S2 had strength and an initial stiffness close to Specimen S1. However the strength degradation in Specimen S2 beyond the peak was faster and more significant as compared to Specimen S1. This is attributed to the failure of the CFRP anchor dowels at the foundation. After the failure of the CFRP anchor dowels, CFRP sheets were no longer as effective and therefore a great portion of lateral load was transferred from the infill to the exterior columns. As a result, the lateral load capacity of Specimen S2 decreased drastically.

In Figure 8, first story drift ratio envelope curves of the strengthened frames and the bare frame are given. On the same figure, the drift limit for elastic analysis specified by the Turkish Seismic Code [10] is also shown in order to enable comparison. The Turkish Seismic Code suggests the interstory drift limit as 0.0035 for the type of RC systems used in this study. Both specimens had insignificant strength and stiffness degradation till this limit. After exceeding this limit, drift ratios increased more rapidly without significant increase in the lateral load. The strength degradation beyond the peak was much more significant in Specimen S2 as compared to Specimen S1.

As can be seen from Table 3, which summarizes the test results of strengthened frames and the bare frame, both the strength and the stiffness of the strengthened specimens (S1 and S2) were significantly higher than those of the bare frame. The ratio of the strength of the Specimens S1 and S2 to bare frame was about five. The ratio of the initial stiffness of the strengthened frame to bare frame was about fourteen in Specimen S1 and eleven in Specimen S2.

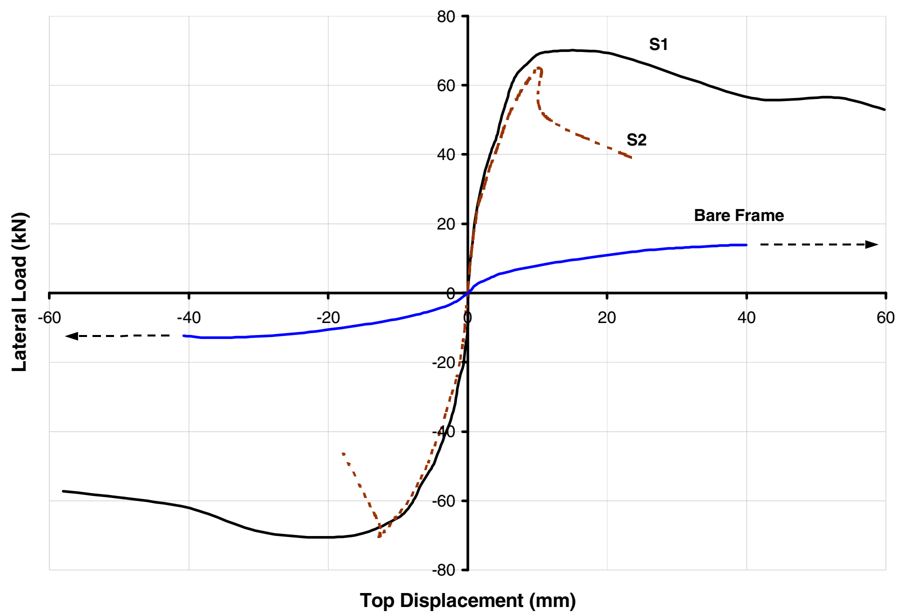

Figure 7. Envelope Curves of the Specimens (Bare Frame is taken from Canbay et al. [4])

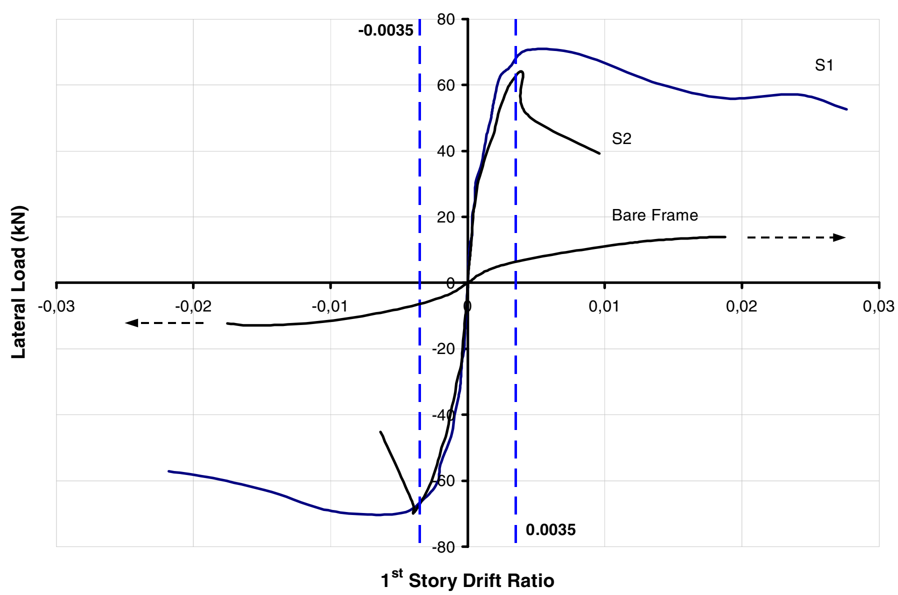

Figure 8. First Story Drift Ratio Envelopes of the Specimens

The shear force measured at the base of exterior columns by the force transducers were used to evaluate the portion of the lateral load carried by the infill wall. Figures 9 and 10 shows that at least 90 percent of the lateral load was carried by infill. Since the contribution of the frame members outside the infill seem to be insignificant, one can conclude that the strengthening of the frame members which have various deficiencies is not needed if adequate infill walls are provided.

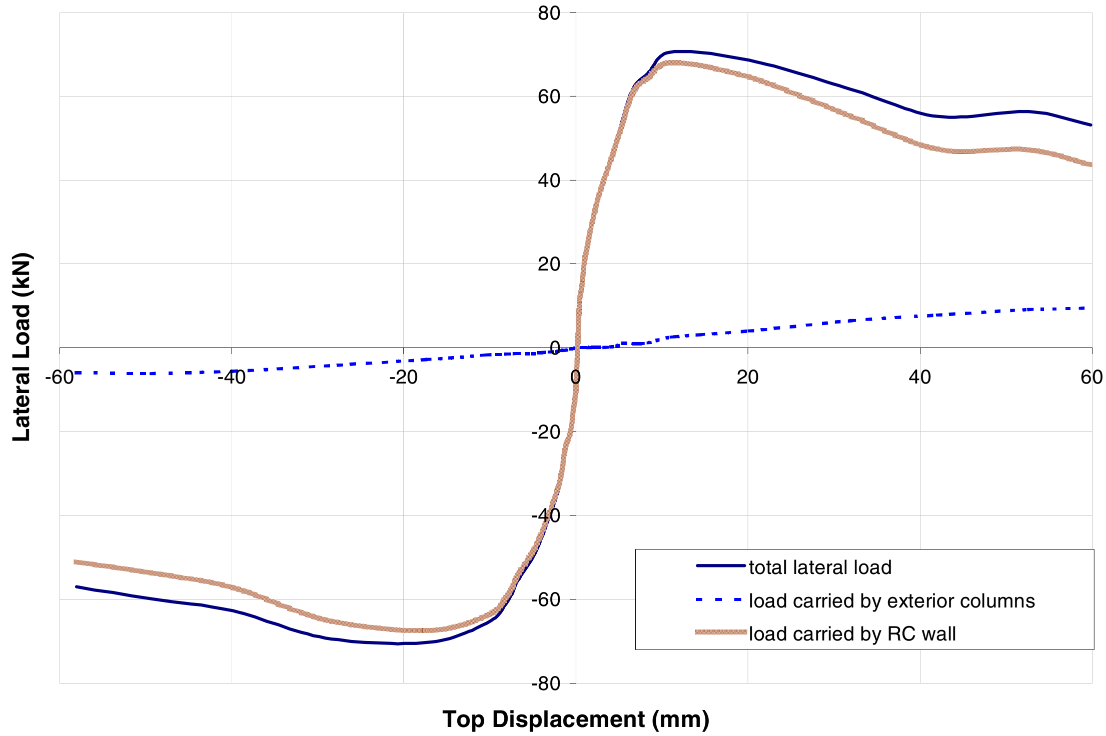

Figure 9. Load Sharing Envelopes of the Members of Specimen S1

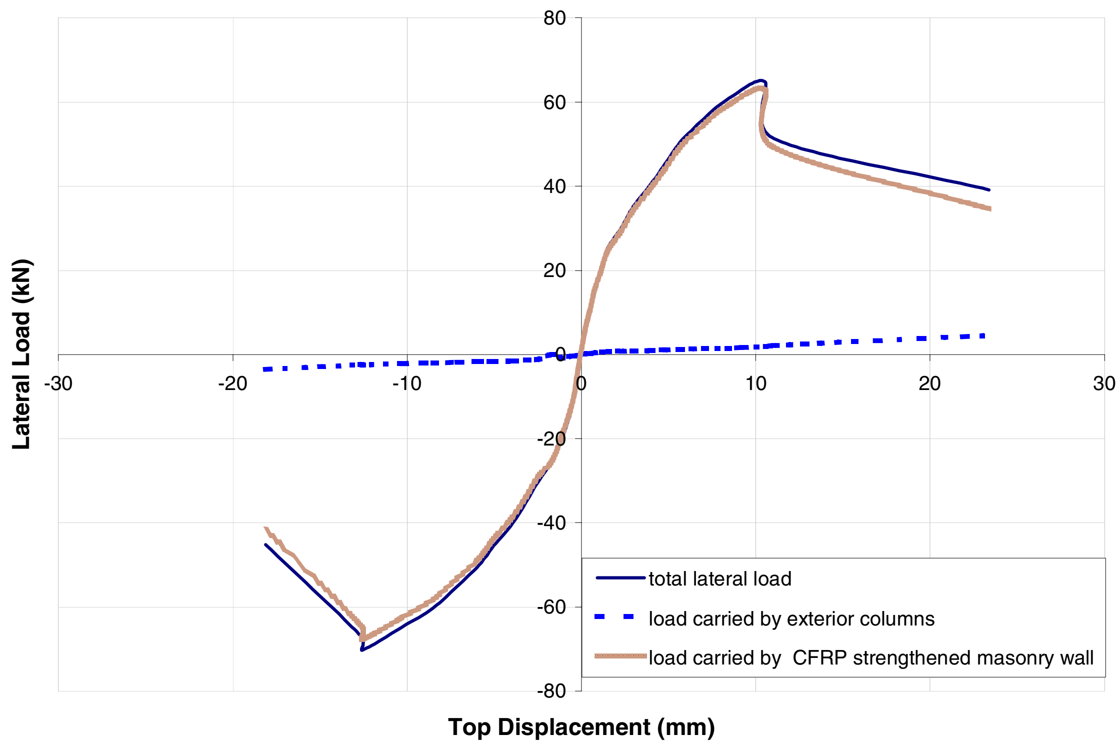

Figure 10. Load Sharing Envelopes of the Members of Specimen S2

PUSHOVER ANALYSES

Both Specimen S1 and Specimen S2 were modeled using commercial nonlinear finite element program ANSYS 7.0 [11]. To model RC frame members, *Solid 65* element was used. This element is used for 3-D modeling of solids with reinforcing bars. It has 8 nodes having 3 degrees of freedom at each node, translation in x, y and z directions. Considering the difference in the characteristics of the reinforcement (volumetric ratio, orientation and stress-strain relationship), different input sets were prepared for different members. Kent and Park Model [12] was used to define the stress-strain relationship of the concrete.

Specimen S1

In the modeling of Specimen S1, the middle bay of the frame was filled with 70 mm thick solid member representing the RC infill. Solid *65* elements were also used to represent the infill. Point load of 10 kN vertical load (own weight + additional dead load) was applied on top of each column. Since the specimen was tested under applying in-plane loading, out of plane translation of all nodes was prevented. All nodes at the foundation level were fixed in all directions.

Three pushover analyses were made; (a) considering the strain hardening of reinforcing bars, (b) with no strain hardening in reinforcing bars and (c) considering the slip of longitudinal bars with no strain hardening in the boundary columns. In all analyses 60 mm maximum lateral top displacement was applied incrementally to the nodes of the second story exterior joint where the displacements were measured during the test. At each step, applied displacement and the corresponding base shear were obtained. To include the bar slip in the analysis, the average bond strength of 1.7 MPa as suggested by Mylrea for plain bars was used [13]. The bond strength multiplied with the bond area (surface area of the 320 mm splice length) must be equal to the force in the reinforcing bar for the equilibrium. The force in the bar divided by the cross-sectional area of the bar would give the maximum stress in the bar as 272 MPa. This stress was used as the yield strength of the reinforcing bars of the interior columns.

The three pushover curves and the experimental curve are shown in Figure 11. In this figure, curve (a) represents the case in which strain hardening of the steel is considered, curve (b) represents the case in which strain hardening of steel is ignored and curve (c) represents the case in which slip of the longitudinal bars without strain hardening is considered. As it is seen, the analytical curve in which slip was considered (curve (c)) is very close to the experimentally observed behavior. Previous studies have shown that because of the poor bond between the plain bars and the concrete, strain hardening of the bars is hardly possible and slip of the plain bars is inevitable. This was also observed in the response envelope curves of Specimen S1 (Figure 11). However, the strength degradation beyond the peak point could not be predicted by the pushover analyses.

Specimen S2

Since Specimen S2 was strengthened with CFRP applied on hollow clay tile infills, new materials and members were defined in the input. Hollow clay tile infills strengthened with CFRP were represented as diagonal struts under compression or tension. Finite element model of Specimen S2 is given in Figure 12.

The lapped spliced regions of the interior columns were confined with CFRP. Therefore a different concrete model was used to model these regions. Although reinforcing steel confined concrete is similar to CFRP confined concrete there is a small difference between them. This model was derived by Karbhari and Gao [16] after testing CFRP confined concrete specimens.

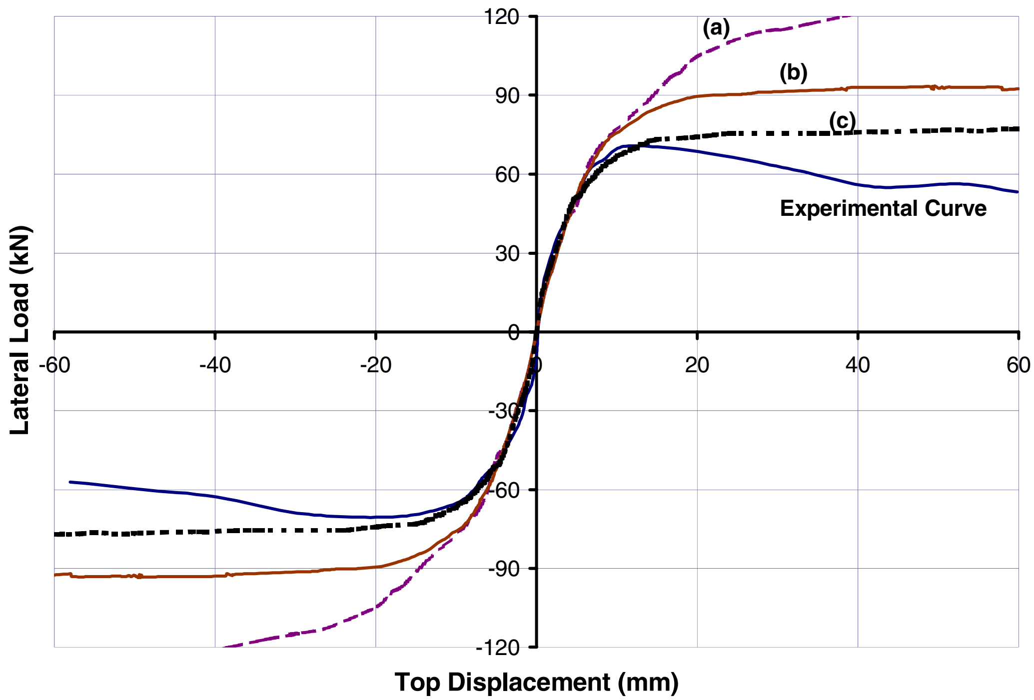

Figure 11. Response Envelopes of Specimen S1

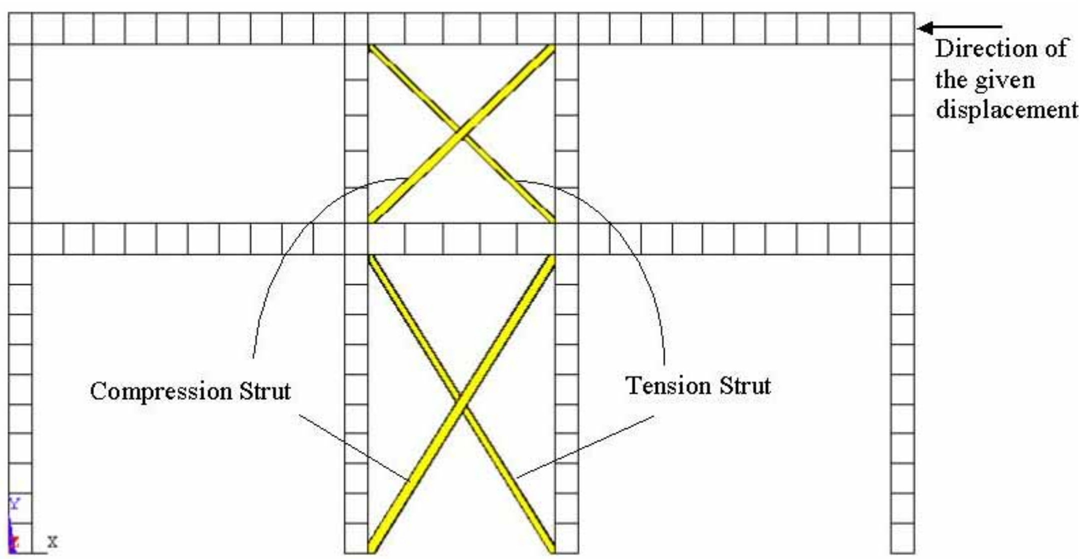

Figure 12. Finite Element Model of Specimen S2

Compression struts were defined using only the material properties of the hollow clay tiles, assuming CFRP has no significant strength under compression. Equivalent widths of the compression struts in both stories were determined by using FEMA 273 guidelines [14]. Relative stiffness of infill to frame, aspect ratio of infill are the parameters affecting equivalent strut width. Since the aspect ratios of the infill in the first story and in the second story were different, the width of the compression strut was taken as 638 mm and 480 mm for the first and second stories, respectively. The thickness of the compression strut was taken as the thickness of the plastered hollow clay tile infill (90 mm). The material model defined for the compression struts was bilinear with a modulus of elasticity of 1,000 MPa and an ultimate strength of 2 MPa [14].

As tension struts, first a pushover analysis, considering the CFRP sheets only, was made. In that analysis the stress-strain curve given in Figure 13 was used. The cross sectional areas of the tension strut which model CFRP sheet were 300×2 mm^2 in the first story and 200×2 mm^2 in the second story. The analytical result (curve (c)) is given in Figure 15 with experimental result. As it is seen there is no agreement between the two curves. Then, it was decided to include the effect of hollow clay tile infill in tension strut. For this decision an attempt to model CFRP with hollow clay tile was made considering the design necessities. The thickness of the tension strut was taken as the thickness of the composite material (CFRP + plaster + clay tile), which was 92 mm. There was no available data for the stress-strain relationship of such a composite material. Therefore some analyses were performed with different modulus of elasticity of the tension strut. Elasticity modulus of 64,000 MPa, which is the elasticity modulus of CFRP obtained by coupon tests, gave relatively good result under small lateral displacement. It is considered that CFRP and hollow clay tile infill work together up to cracking of the hollow clay tile infill. However, after cracking, only CFRP works, i.e., the area of the strut decreases to the area of CFRP only. Since the area reduction is not possible during an analysis with ANSYS 7.0, reduction in the area was converted to a reduction in the elasticity modulus of the composite material. Cracking strain of the infill was taken as the cracking strain of the mortar and it was calculated as 3.5×10^{-5} according to ACI-318 [15]. Therefore, at strain of 3.5×10^{-5} elasticity modulus of the composite material was decreased considering the reduction in the area. Since the strut area was reduced from 27,600 to 800 mm^2 approximately, elasticity modulus was decreased from 64,000 to 1,855 MPa. Pushover curve (curve (a)) was obtained using the material properties and the model described. As can be seen in Figure 15, the capacity of the test specimen drops sharply due to the failure of the anchorages. Therefore, elastic modulus of the material decreased to zero at a strain of 4.6×10^{-4} to include the failure of the CFRP anchorage dowels. The material model used for the composite material, which resulted curve (b) is shown in Figure 14. As it is seen from Figure 15, the result of the analysis in which failure of the CFRP anchor dowels is considered, agree better with the experimental curve.

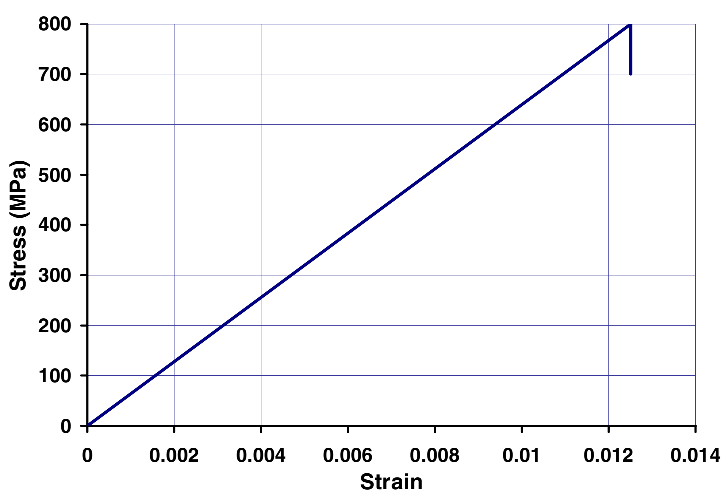

Figure 13. Tension Strut Model for CFRP

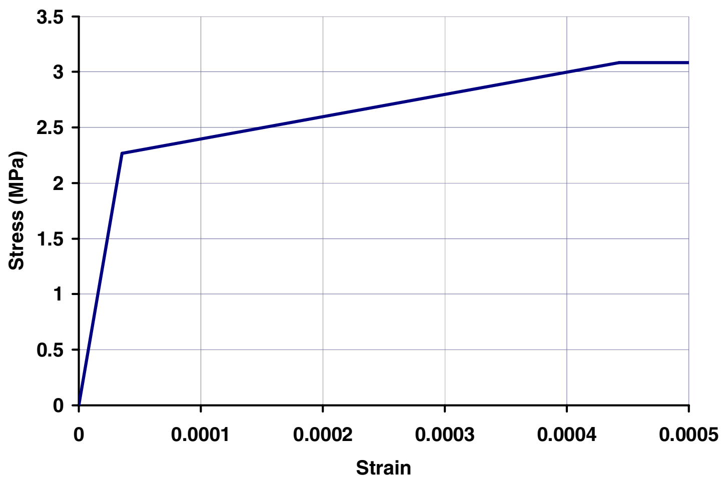

Figure 14. Tension Strut Model for Composite Material

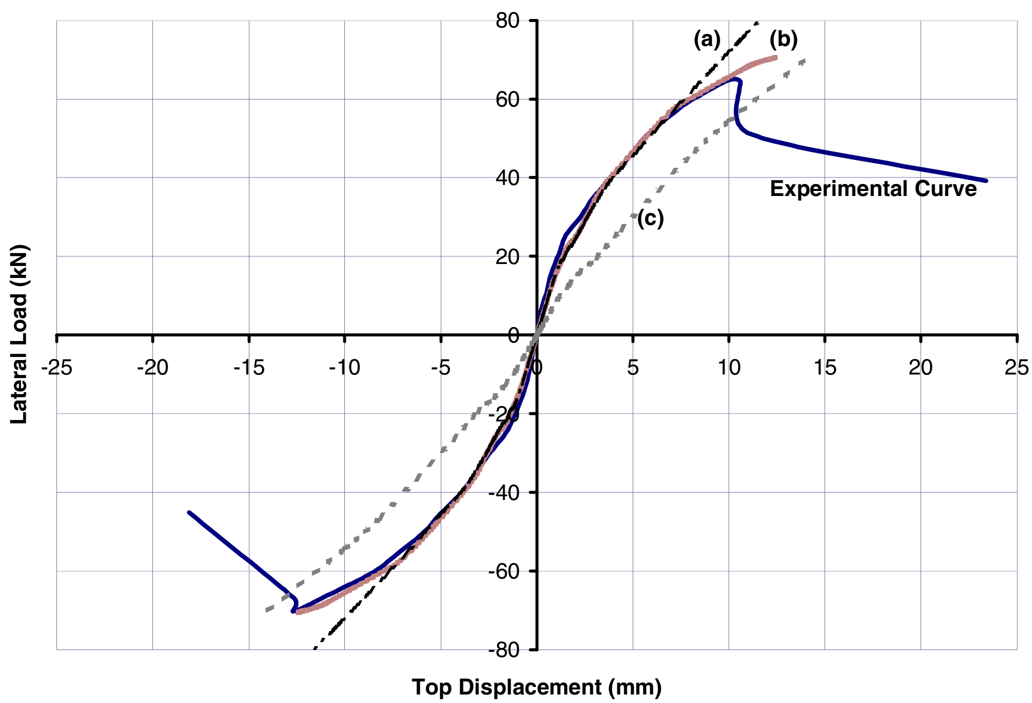

Figure 15. Response Envelopes of Specimen S2

CONCLUSION

Both strengthening techniques increased the stiffness and strength significantly. The lateral strength was increased by approximately 500 percent but the strength degradation of Specimen S2 beyond the peak was more pronounced. The stiffness of the strengthened frames was at least ten times that of the bare frame. The initial stiffness of Specimen S2 was slightly lower as compared to Specimen S1. The capacity of Specimen S2 depends on the effectiveness of the CFRP anchor dowels.

Axial force, shear and moment at the base of exterior columns were measured by specially designed transducers. These measurements indicated that at least 90 percent of the base shear was carried by the infill. Therefore the poor detailing of the frame members had no significant effect on the behavior and the capacity of the test specimens. It can be concluded that when strengthening is made by infilling selected bays of the framed structure or when existing infills are strengthened with CFRP sheets, strengthening of the frame members having various deficiencies seem to be not needed.

Initial stiffness of the finite element model of Specimen S2 highly depends on the cross-sectional area of tension struts. The behavior of Specimen S2 can be predicted quite accurately by a pushover analysis up to the maximum load. However, pushover analysis does not yield reasonable results beyond the peak

Plain bars were used as reinforcement since such bars have been used in most of the existing residential buildings in Turkey. Slip in the column longitudinal bars affected the capacity of both test specimens. The capacities could not be predicted by analysis unless bar slip was considered.

REFERENCES

1. Ersoy U. and Uzsoy S. "The Behavior and Strength of Infilled Frames." Report No. MAG-205, TUBITAK, 1971; Ankara, Turkey.
2. Higashi Y. and Kokusho S. "The Strengthening Method of Existing Reinforced Concrete Buildings.", U.S.-Japan Cooperative Research Program in Earthquake Engineering, 1975; Honolulu H1.
3. Sugano S. and Fujimura M. "Aseismic Strengthening of Existing Reinforced Concrete Buildings.", Proc. of the 7th WCEE, Istanbul, 1980; Vol.4.
4. Canbay E., Ersoy U. and Ozcebe G. "Contribution of Reinforced Concrete Infills to Seismic Behavior of Structural Systems." Structural Journal, ACI, 2003; 100(5): 637-643.
5. Pantelides C. P., Gergely J., Reaveley L. D. and Volnyy V. A. "Retrofit of RC Bridge Pier with CFRP Advanced Composites." J. of Structural Engrg, ASCE, 1999; 125(10): 1094-1099.
6. Triantafillou T. C. "Strengthening of Masonry Structures Using: Epoxy-Bonded FRP Laminates." Journal of Composites for Construction, ASCE, 1998; 2(2): 96-104.
7. Albert M. L., Elwi A. E. and Cheng J. J. R. "Strengthening of Unreinforced Masonry Walls Using FRPs." Journal of Composites for Construction., ASCE, 2001; 5(2): 76-83.
8. Ueda T., Sato Y. and Asano Y. "Experimental Study on Bond Strength of Continuous Carbon Fiber Sheet.", Fourth International Symposium on FRP Reinforcement for RC Structures, ACI International, 1999; SP 188-37, 407-416.
9. Ozcebe G., Ersoy U., Tankut T., Erduran E., Keskin R. S. O. and Mertol H. C. "Strengthening of Brick Infilled RC Frames with CFRP." SERU-Structural Engineering Research Unit, 2003; Report No. 2003/1, TUBITAK-METU, Ankara, Turkey.
10. Turkish Seismic Code, 1998; Turkey, 4-25.
11. ANSYS user's manual-version 7.0, 2002.
12. Kent D. C. and Park R. "Flexural Members with Confined Concrete." J. of Structural Engineering, ASCE, 1971; 97(7), 1969-1990.
13. Mylrea T. D. "Bond and Anchorage", J. ACI, 1948; V. 44.
14. FEMA 273 "NEHRP Guidelines for the Seismic Rehabilitation of Buildings." 1997.
15. ACI "Building Code Requirements for Reinforced Concrete and Commentary (ACI 318-R-95)." Michigan, 1995.
16. Karbhari V. M. and Gao Y. "Composite Jacketed Concrete under Uniaxial Compression-Verification of Simple Design Equations." Journal of Materials in Civil Engineering., ASCE, 1997; 9(4): 185-193.