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Strengthening of deficient RC frames with high strength concrete panels: an experimental study

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Abstract. An economic, structurally effective and practically applicable strengthening technique was developed for reinforced concrete (RC) framed buildings. The idea of the technique is to convert the existing hollow brick infill wall into a load carrying system acting as a cast-in-place RC infill wall by bonding relatively thin high strength precast concrete PC panels to the plastered hollow brick infill. For this purpose, a total of eight one-third scale, one bay, one story frames were tested under reversed-cyclic lateral loads. Test frames were designed and constructed with common deficiencies observed in practice. Four different panel types were used for strengthening. Test results showed that both strength and stiffness of the frames were significantly improved by the introduction of PC panels. Experimental results were compared with the analytical approaches suggested by the authors.

Keywords: strengthening; reinforced concrete; hollow brick infill wall; precast concrete panels; lap splice.

1. Introduction

Most of the existing RC residential buildings in many countries are seismically deficient since the load carrying systems of these buildings contain deficiencies such as flexible columns, soft stories, insufficient confinement, strong beam – weak column combinations and lap splices with insufficient lengths. These buildings suffer large lateral displacements under seismic loads due to low lateral stiffness. Therefore, a large existing building stock awaits seismic vulnerability assessment and seismic retrofitting. In the past, different techniques have been developed and applied for seismic strengthening of RC framed buildings. Among the available techniques, adding new cast-in-place RC infill walls to RC framed buildings was found to be the most appropriate and reliable method of system improvement in studies conducted by Ersoy and Uzsoy (1971), Altin *et al.* (1992), Miller and Reaveley (1996), Gregorian and Gregorian (1996), Turk (1998), Canbay *et al.* (2003) and Sonuvar *et al.* (2004). By constructing this type of infill wall, in some cases in the place of a partitioning wall, the building gains considerable strength and stiffness. Many buildings were repaired or strengthened with this method, especially after major earthquakes. However, there are

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some drawbacks of cast-in-place RC infill wall technique. The application of this method requires heavy construction work, so it is obligatory to evacuate the building. The workmanship in this retrofitting technique is difficult and time-consuming. It also necessitates acquiring and transporting large amounts of materials into the building. Recently, researchers all around the world have focused on developing economical, effective and practical strengthening techniques which do not require evacuation of the building. Zhou *et al.* (2007) proposed a strengthening technique by retrofitting one-third scale one story concrete frames using fiber-reinforced polymers (FRP). Among the newly proposed techniques, use of PC panels is an alternative technique for being inexpensive to produce, easy to apply and structurally effective after application.

Extensive experimental research has been conducted where panels have been used as infill materials. In the studies conducted by Yuzugullu (1979), Kahn and Hanson (1979), Hanson (1980), Higashi *et al.* (1980), Kaldjian and Yuzugullu (1983), Higashi *et al.* (1984), Phan *et al.* (1995), Frosch (1996), Frosch *et al.* (1996), Frosch *et al.* (1996), Li (1997), Matsumoto (1998), Frosch (1999), Isao *et al.* (1999), Frosch *et al.* (2003), Kesner and Billington (2005) and Cho *et al.* (2008), orientation and number, thickness, compressive strength, shear behavior, panel to panel and panel to frame were the main parameters studied. In addition, experimental results were compared with the results obtained by the application of other techniques, especially cast-in-place RC infill walls. These studies showed that use of panels for strengthening was an effective and convenient method which increases the strength and stiffness of the RC frames considerably, and saves cost and time.

The study presented in this paper concerns the success of an innovative retrofitting technique which is suitable for the hollow brick infilled RC framed residental buildings that constitute the majority of building stock in many countries. The idea is to convert the existing hollow brick infill wall into a load carrying system acting as a cast-in-place RC infill wall by bonding relatively thin high strength PC panels to the plastered hollow brick infills by epoxy mortar. A single panel would be unmanageable, too large to go through doors and too heavy to be carried by two workers. Therefore, the panels to be bonded have to be of manageable size and weight, and have to be assembled on the infill side by side with the other panels. In studies conducted by other authors, panel to frame connections and preventing panels' out-of-plane deformations were provided by different methods. Applying post-tensioning to panels was one of the methods in the studies conducted by Frosch (1996), Frosch *et al.* (1996), Frosch *et al.* (1996), Li (1997), Frosch (1999) and Frosch *et al.* (2003). In the present study, hollow brick infills and PC panels both form a stiff and composite infill where the panels provide stiffness and strength to hollow brick infill and the infill holds the panels against out-of-plane deformations. Panel geometry (full height strip or nearly square), panel to panel connections (shear keys, welding, only epoxy), panel to frame connections (welding, dowels at two or four sides) and effect of lap splice with columns' axial load level were the main parameters to be investigated.

2. Experimental investigation

2.1 Test specimens

The test specimens used in this experimental study were one-third scale, one-story one-bay RC frames with typical characteristics and common deficiencies of RC buildings such as low concrete strength, use of plain bars, short lap splice length ($20\phi = 160$ mm), insufficient anchorage, poor confinement and beam strength greater than columns. In columns which had 100 mm × 150 mm dimensions, 4-ϕ8 plain bars were used as longitudinal reinforcement whereas 6-ϕ8 plain bars were used in beams having 150 mm × 150 mm dimensions. Ductility of frame members was low, since insufficient ties were used in columns and the beam. Stirrups were ϕ4 bars and they were placed with a spacing of 100 mm, which is too large to provide any confinement. Also, beam-column joints were not confined. Confinement zones were not provided at beam and column ends. According to the Turkish Code, plastic hinge zones of reinforced concrete members should be well-confined. In addition, stirrups had 90º hooks, contrary to the Turkish Code specification for 135º

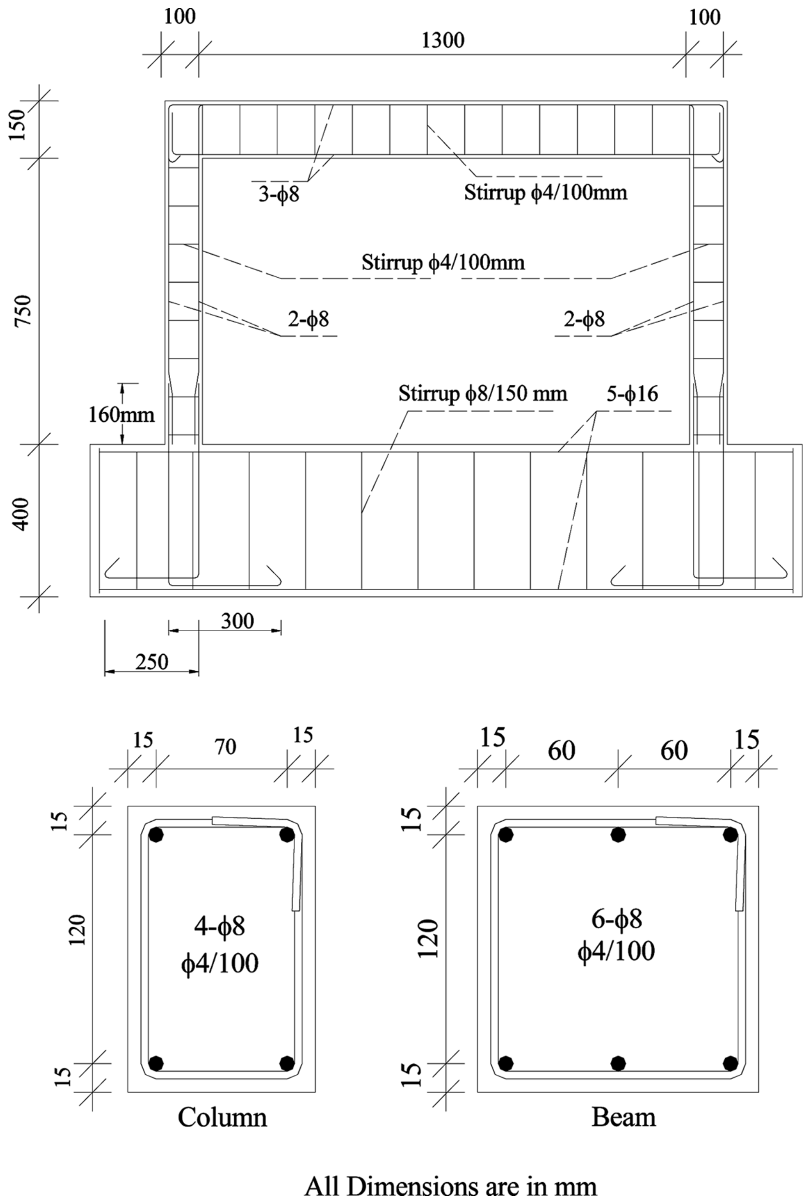


Fig. 1 Dimension and reinforcement of frames

hooks to provide effective confinement by anchoring tie ends to the core concrete. Dimensions and reinforcement details of the test frames are illustrated in Fig. 1. All frames were infilled with scaled (one-third) hollow bricks having 69 mm × 85 mm × 90 mm dimensions. Hollow bricks were laid such that their voids were oriented vertically. The thickness of the plaster on both sides of the infill was 10 mm. Since the hollow bricks were thinner than the frames, they were not located on the axis of symmetry of the frame to simulate exterior infill walls of a building, as is often in the practice case. All frames were white-washed for better observation of the tests.

2.2 Materials

The target concrete strength of the frame was 12 MPa, which is approximately the average grade for existing reinforced concrete buildings in Turkey. Target concrete grade of the panels was C40. In panels, relatively high strength concrete was used to provide high strength while minimizing panel thickness and weight. Workable concrete for the panels was produced using Sikament 300 admixture (specifications given in Product Data Sheet 2003). The same mix designs were used for the mortar in the infill construction and the plaster. Mild steel plain bars were used as reinforcement in both test frames and panels. Average compressive strength of the hollow bricks in the direction of the voids was calculated as 8.5 MPa based on gross area. Concrete compressive strengths of the frames, panels and mortar-plaster on the test date are listed in Table 1. Typical properties of reinforcing bars used are listed in Table 2. Mix proportions for frame and panel concrete are given in Table 3. Mortar-plaster mix proportion is given Table 4.

Table 1 Frame concrete, panel concrete and mortar strengths of the frames

Specimen Designation	Frame Concrete (MPa)	Panel Concrete (MPa)	Mortar (MPa)
CR	15.6	-	6.1
LR	9.7	-	4.9
CA	18.7	34.6	4.6
CB	12.2	46.5	3.4
CC	14.2	38.2	5.2
CD	11.1	45.1	5.2
LC	15.7	38.2	4.9
LD	10.1	45.1	3.3

Table 2 Properties of reinforcing bars

Bar Type	Property	Location	f_y (MPa)
ϕ3	Plain	Mesh steel for panel reinforcement	670
ϕ4	Plain	Stirrup for beam and column Panel reinforcement	220
ϕ6	Deformed	Dowel for frame-to-panel connection	378
ϕ8	Plain	Beam and column longitudinal bars	330
ϕ8	Deformed	Anchorage bar between adjacent panels Stirrup for foundation beam	330
ϕ16	Deformed	Foundation beam longitudinal bar	420

Table 3 Mix designs (Weight for 1 m^3 of concrete)

	Frame Concrete		Panel Concrete	
	Weight (kg)	Proportion by weight (%)	Weight (kg)	Proportion by weight (%)
Cement	267	12	501	19
0-3 mm Aggregate	422	19	994	38
3-7 mm Aggregate	844	38	857	33
7-15 mm Aggregate	444	20	-	-
Water	245	11	276	10
Sikament 300	-	-	4	0.15
Total	2222	100	2632	100

Table 4 Mortar-plaster mix proportion

Material	Sand	Lime	Cement	Water	Total
Weight (%)	65	10	10	15	100

Because of its superior compressive and tensile strength, Sikadur-31 (specifications given in Product Data Sheet 2001) was selected for the epoxy mortar, a compressive strength of 65 MPa was used for panel joints and between the panels and the plaster on the wall. According to the manufacturer's specifications, its tensile strength is nearly 20 MPa which is greater than the strength required in concrete applications. Its adhesive strength to steel and concrete is 30 MPa and 3.5 MPa, respectively. For the embedment of the anchorages to the frame members, Spit Epcon (specifications given in User Manual 2002) was selected because of its superior adhesive and flow properties.

2.3 Precast concrete panels

Two basic types of panels having different geometries and details were designed. They were intended to have reasonable size and weight, so that they could be carried and installed by only two workers. Also, they could not be too large to pass through door openings. Therefore, the panels were proposed to be made up of smaller separate units bonded side by side.

One approach was to arrange the panels in three rows and four columns, and another was to use several panels having the full height of the infill. Since the infill dimensions of the one-third scale test frames were 1300 × 750 mm, the first type of panels were 320 × 245 mm, and the latter type dimensions were 105 × 745 mm. The thickness of the bonding material and imperfections were taken into consideration. The two types of panel geometry and panel types are shown in detail in Fig. 2. The panel thickness was chosen as 20 mm for panels. Therefore, the one-third scale panels weigh about 3 kg. The corresponding weight for the full-scale panels would be about 80 kg, which is still manageable.

Connections between panels and bonding of panels to the frame were done carefully since the performance of the lateral-load-resisting wall depended on the interaction between the two components; panels and the plastered hollow brick infill wall. The panels provide stiffness and strength to the infill wall and the wall provide restraint for out-of-plane deformations. Also, the composite infill and frame must act as a unit. Separation of the infill from the columns, beam or the

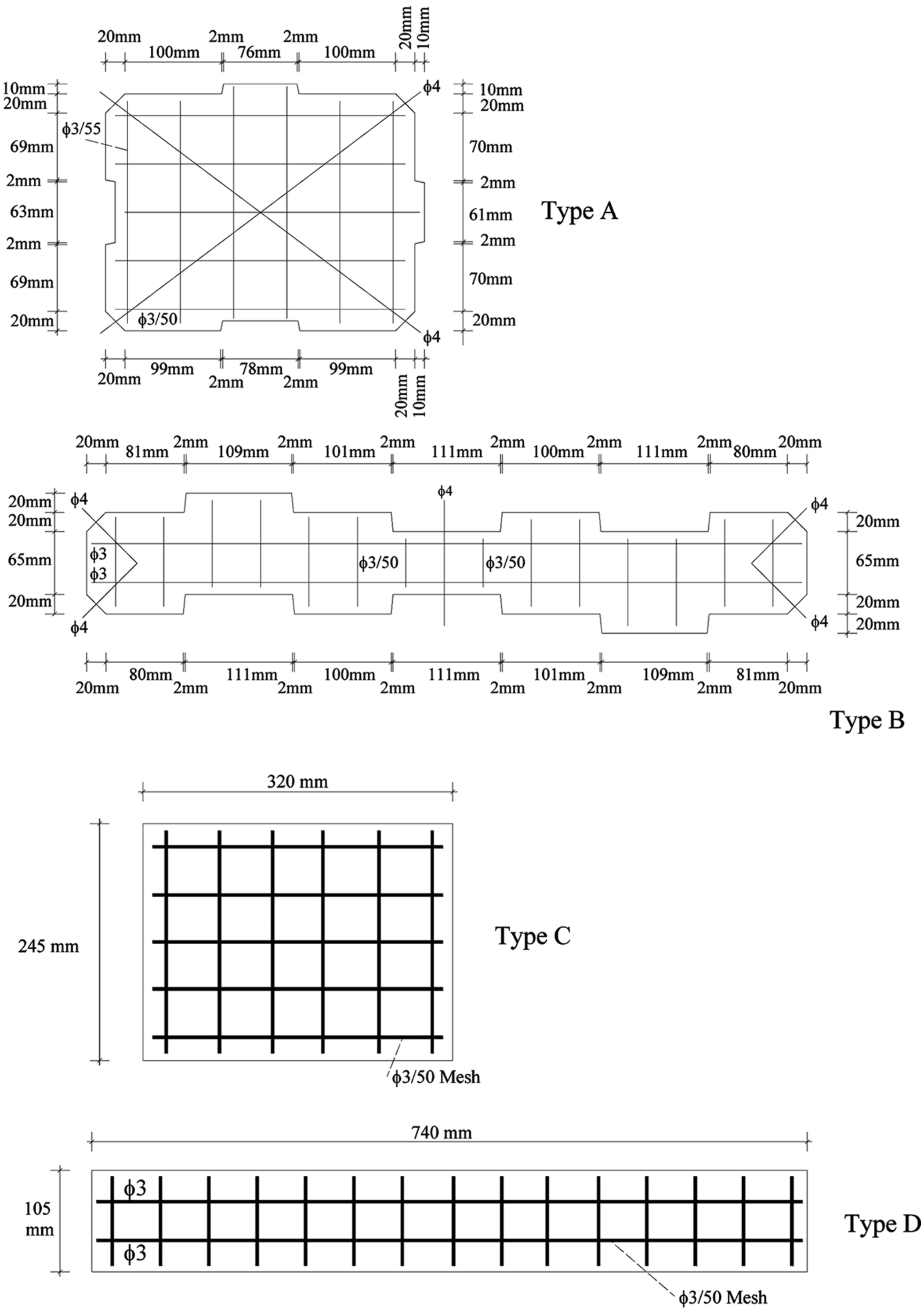


Fig. 2 Dimension and reinforcement of panel types

base should be prevented. As a result, epoxy mortar as well as shear keys, welding of bars that project from the panels, epoxy anchoring dowels in panel types A and B were implemented. Two other, panels with straight edges and without projecting bars were designed to improve the efficiency of the proposed technique. Type C and Type D panels had the same geometries with those of Type A and Type B panels. Eventually, tests of specimens strengthened by using Type C and Type D panels indicated that the epoxy mortar used in connecting the panels to each other and to frame members proved to be so successful that both shear keys and welded connections were

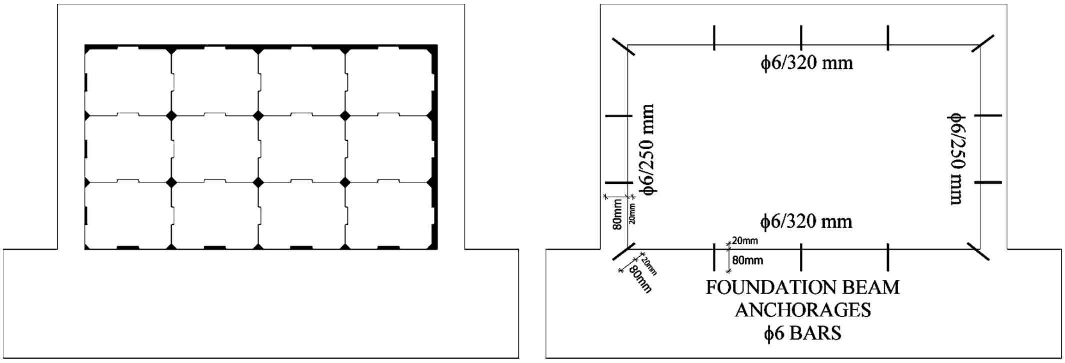


Fig. 3 Panel arrangement for Type A panels (Specimen CA)

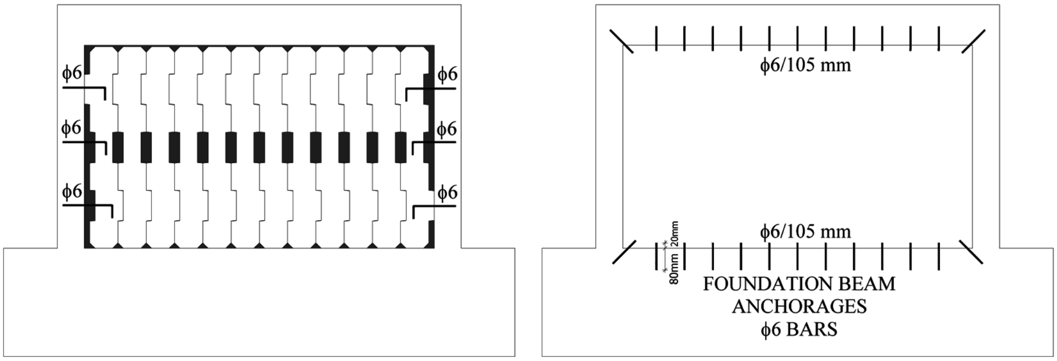


Fig. 4 Panel arrangement for Type B panels (Specimen CB)

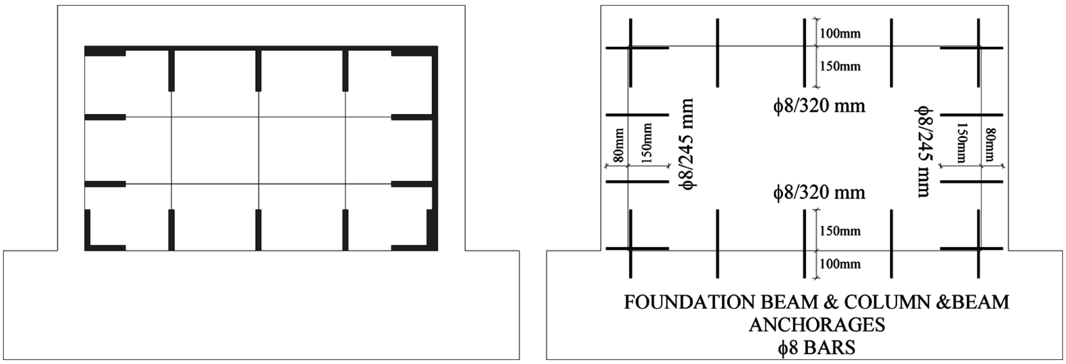


Fig. 5 Panel arrangement for Type C panels (Specimen CC and LC)

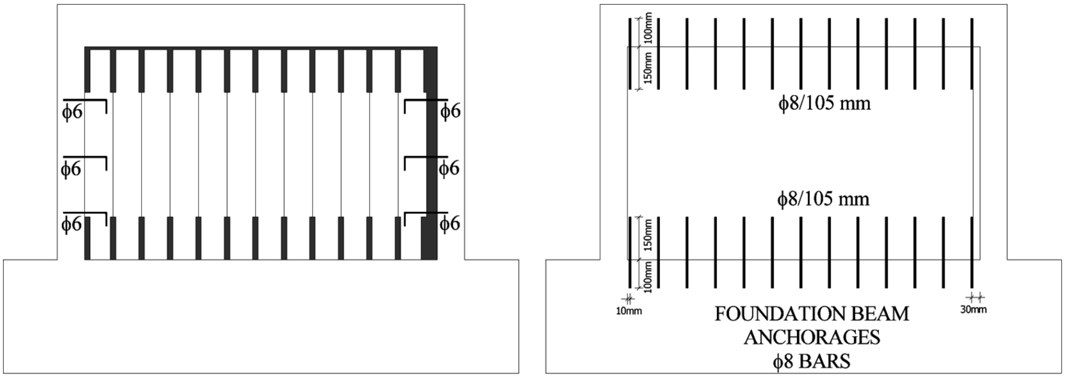
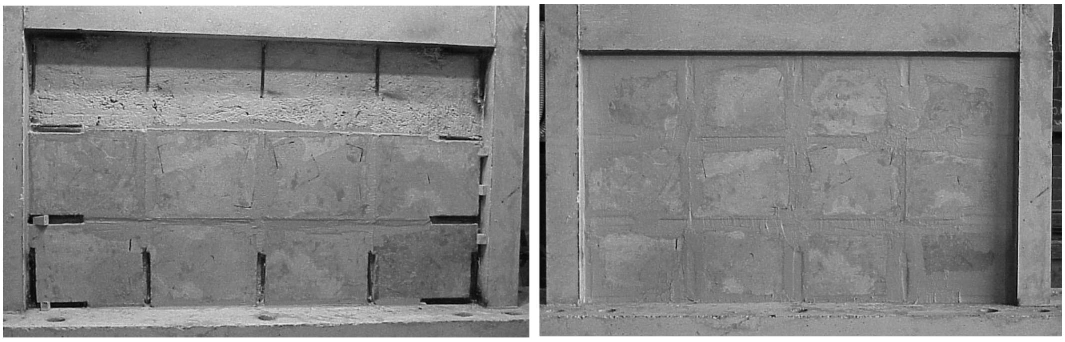


Fig. 6 Panel arrangement for Type D panels (Specimen CD and LD)



Bonding panels on to the hollow brick infill – filling the gaps with epoxy

Fig. 7 Panel application for Specimen CC

omitted in the studies conducted by Baran *et al.* (2003) and Susoy (2004). However, anchorage bars in between the panels at four sides of the infill were considered essential in order to get a monolithic panel strengthened infill-frame behavior.

After drilling holes in the existing frame members for anchorages, holes were cleaned with compressed air and infilled with epoxy resin. Anchorages were $\phi 6$ deformed bars for Type A and Type B panels and $\phi 8$ deformed bars for the remaining type panels. After fixing the anchorages and bonding panels to the plaster infill wall, panel bars were welded to each other and to anchorages for Type A and Type B panels. Then, gaps between the panels and between the panels and the frame members were filled with epoxy mortar. No surface finishing were normally required since the panels had sufficiently smooth surfaces. At room temperature, epoxy mortar and resin gains considerably enough strength in several hours, which shows the bonding effectiveness of the epoxy mortar.

The frames are grouped in pairs. The legend of each pair starts with the letter "C" to indicate that the frame has continuous column longitudinal reinforcement or "L" to indicate that the frame has lap splices (20ϕ bar diameter = 160 mm in length) in column longitudinal reinforcement at foundation level. The second letter "A", "B", "C" or "D" indicates the type of the panel used to strengthen the frame or "R" indicates that the specimen is a reference test.

Panel configurations for test frames are illustared in Fig. 3 to Fig. 6. Panel application for Specimen CC is illustrated in Fig. 7.

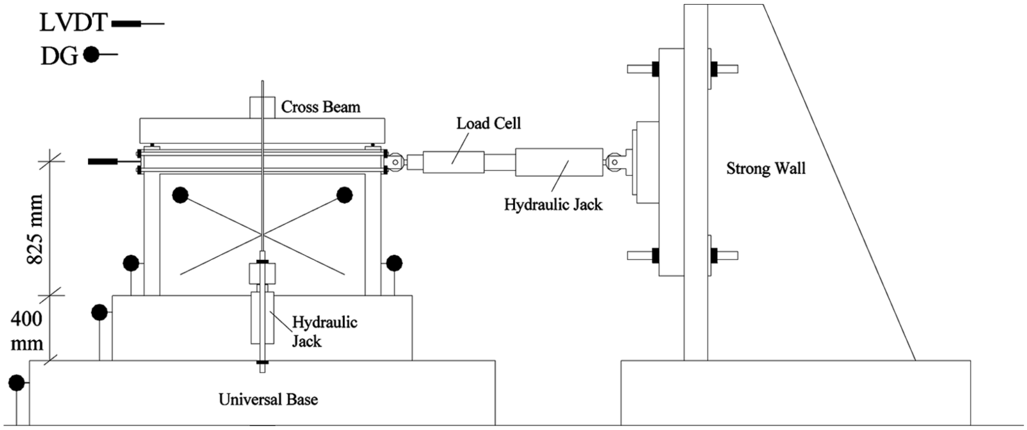


Fig. 8 Details of test set-up, loading system and instrumentation

2.4 Test set-up, loading system and instrumentation

As can be seen in Fig. 8, specimens were loaded against a reaction wall. Each test frame was cast with a rigid foundation beam, which was bolted to the universal base prestressed to the strong floor of the laboratory. Reversed-cyclic lateral loading was applied by using a double acting hydraulic jack (the capacities being 600 kN in compression and 420 kN in tension). A load cell was connected between the hydraulic jack and the test frame to measure the magnitude of the applied lateral load. In the tests, the lateral load was increased by 10 kN at each cycle up to peak and beyond that, deformation was controlled with increasing displacement cycles. Load-top displacements curves of the test frames were plotted.

The axial load on columns was approximately equal to 10% or 20% of the column axial load capacity and was provided by steel cables post-tensioned by hydraulic jacks, as shown in Fig. 8. The load was continuously monitored and readjusted during the test. A rigid steel guide frame was constructed around the test frame to prevent out of plane deformations.

All deformations were measured by displacement transducers; using either Linear Variable Differential Transducers (LVDT) or electronically recordable Dial Gauges (DG) as shown in Fig. 8. Sway displacement was measured at the story level by an LVDT. Infill wall shear deformation was measured by dial gauges placed on the infill diagonally and located 130 mm away from the corner of the infill walls. Displacement measurements at the bottom of both columns were taken to be able to calculate rotations of the entire frame as well as providing data for monitoring the critical column section deformations. All cracks on the frame were marked during the test and the mechanism of failure was observed during testing.

3. Experimental results

3.1 Behavior of test specimens

Lateral load-storey drift ratio curves of specimens are given in Fig. 9. As indicated in this figure, bonding PC panels to the plastered hollow brick infills increased strength and stiffness significantly.

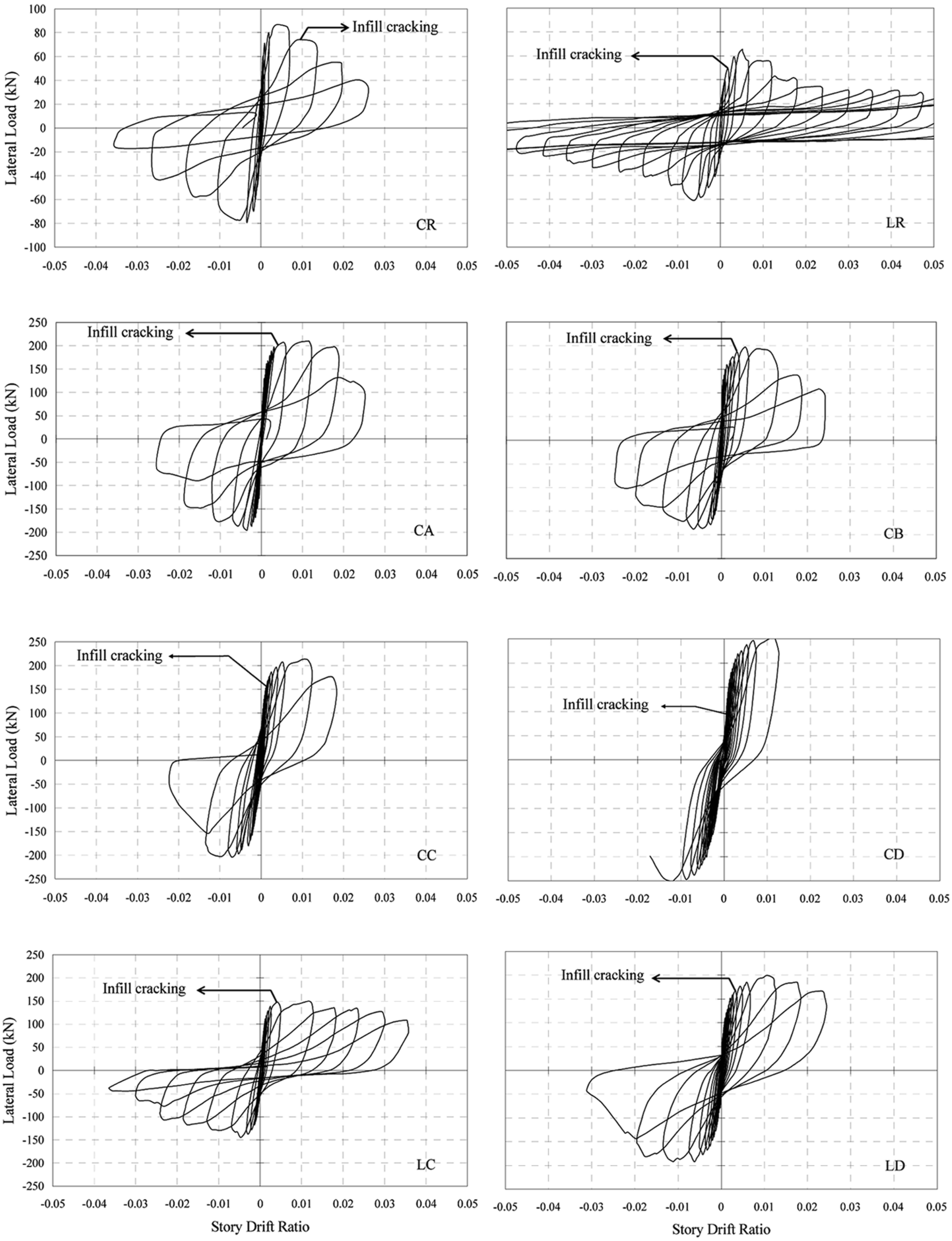


Fig. 9 Lateral load-story drift ratio curves

Reference specimens CR and LR reached 0.36% and 0.53% lateral drifts at their ultimate loads. After the infill of Specimen CR crushed at the upper corners due to diagonal compression, the specimen lost its lateral load carrying capacity. Crushing of the infill and failure of both columns of Specimen LR happened simultaneously, thus the experiment was ended due to excessive damage.

Specimens CA, CB, CC and CD experienced some infill damage, however failure occurred in the columns just below column-beam joint regions. As it can be seen in Fig. 9, Specimen CB lost its lateral load carrying capacity and stiffness more abruptly than Specimens CA, CC and CD after infill cracking occurred. Specimen CC also exhibited infilled frame behavior rather than monolithic

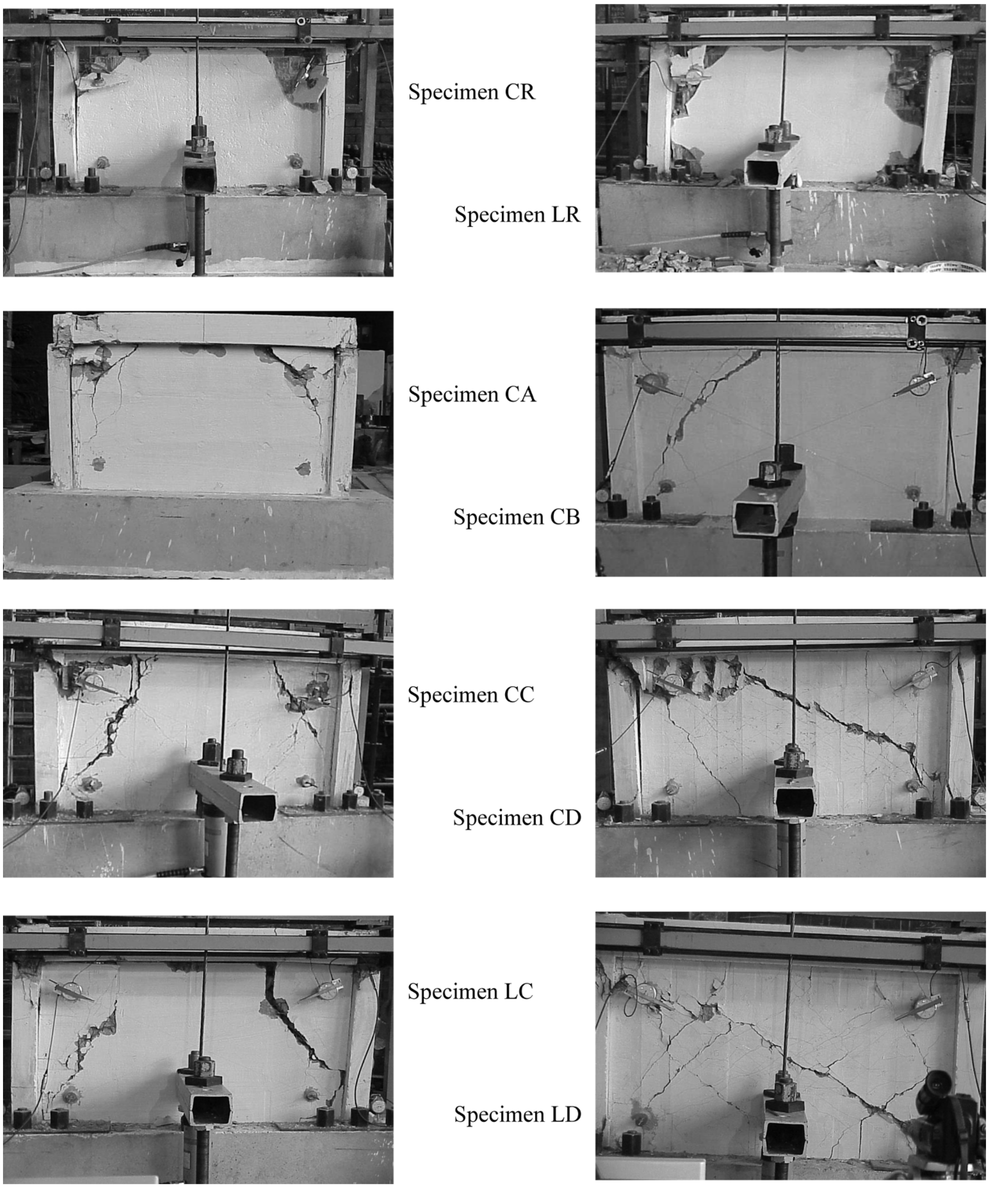


Fig. 10 Specimens after testing

shear wall behavior after diagonal cracking and crushing of its infill. However, the only difference was the lower ductility level, and this can be attributed to the simpler connection details between the panels and the frame members. Specimen CD behaved significantly different. Its behaviour was much less ductile, reached less than half of the maximum lateral displacements of other specimens. The infill panels of specimen CD had dowel connections to all frame members, and due to the strip shape of the panels, it had a greater number of dowels to the foundation and beam. That provided this specimen an infill of relatively high load capacity. When the infill crushed, the relatively weak frame members could not carry the excessive load in frame action. Except from Specimen CB, all strengthened specimens reached storey drift ratio of nearly 1.10% at their ultimate loads. Specimen CB reached this story drift ratio value at 96% of its ultimate load after peak.

Specimens LC and LD also experienced infill and frame member damage, especially at the joints and at the column bases. Lower axial load level led to widening of diagonal cracks on the infill. As a result of lower axial load and presence of lap splice, larger diagonal cracks on the infill increased deformations resulting with more ductile behaviour. Specimens LC and LD showed storey drift ratios of 1.24% and 1.09% at ultimate loads, which were nearly equal to the values of Specimens CC and CD. Fig. 10 shows specimens after testing. In this figure, crack pattern on specimen infills can be seen.

4. Discussion of test results

4.1 Strength and stiffness

Test results are summarized and presented in Table 5. This table was prepared to show the effect

Table 5 Summary of the test results

Specimen	Axial Load Level N/N_o[(1)]	Max. Forward Load (kN)	Ratio[(2)]	Drift Ratio δ/h	Initial Stiffness (kN/mm)	Ratio[(3)]	Cumulative Energy Dissipation (Joule)	Ratio[(4)]
CR	0.25	86.6	1.00	0.0036	96.0	1.00	5.7	1.00
LR	0.13	65.5	1.00	0.0053	59.9	1.00	8.6	1.00
CA	0.25	209.9	2.42	0.0112	312.4	3.25	15.5	2.72
CB	0.25	197.0	2.27	0.0056	308.0	3.21	15.1	2.65
CC	0.25	213.5	2.47	0.0106	294.0	3.06	9.2	1.61
CD	0.25	254.7	2.94	0.0119	275.8	2.87	8.4	1.47
LC	0.13	148.9	2.27	0.0124	159.3	2.66	14.3	1.66
LD	0.13	199.6	3.05	0.0109	280.4	4.68	14.4	1.67

(1) $N_0 = 0.85 f_c' A_c + f_y A_{st}$
(2) Ratio of max. forward load to that of reference specimen
(3) Ratio of initial stiffness to that of reference specimen
(4) Ratio of cumulative dissipated energy to that of reference specimen

of the proposed strengthening technique on ultimate strength, stiffness, energy dissipation capacity and interstorey drift ratio of RC test frames.

The ratio of the ultimate lateral strength of Specimens with continuous column longitudinal bars to that of the reference specimen ranged between 2.27 and 2.94. The increase in strength was more pronounced for Specimens CC and CD, which were strengthened with simplified types of panels, namely Type C and Type D panels. The ratios for Specimens with lap splices on column longitudinal bars were 2.27 and 3.05. Specimens with lap splices on column longitudinal bars showed similar strength increase as Specimens having continuous bars. As compared to specimens with continuous column bars, corresponding specimens with lap splices had relatively lower strength, as is normally expected.

The superior capacity of specimens strengthened with type D panels (CD and LD) over specimens strengthened with type C panels (CC and LC) is very significant. The most influential factor is the number of dowel bars. At the foundation level and beam level, 5 dowels were used for type C dowels, whereas 13 dowels were employed between panels of type D, due to different shape of the panels. Also, the shape of type D panels is more effective for load transfer between panels.

Response envelopes shown in Figs. 11 and 12 are plotted by connecting the peak points of the load-displacement hysteretic curves for each specimen. Response envelope curves can be used for evaluating the strength and stiffness characteristics of the specimens and also general behaviors. As can be seen from this figure, strength and stiffness of strengthened frames were significantly higher than those of the reference specimen. According to Turkish Seismic Code 2006, maximum interstory drift ratio is limited to 0.0035 in the elastic analysis of the structure and this limit was marked on the curves given in Figs. 11 and 12. All Seismic Codes provide similar limits to prevent extensive structural and non-structural damage and to minimize second order effects. This value in Eurocode 8 regulations, for brittle nonsrructural infills in contact with the RC frame, the value is 0.5%. There was no significant drop in the lateral load carrying capacities of the strengthened specimens up to this limit. When stiffness is an issue, the drop in stiffness is more pronounced in specimens with lap splices in column longitudinal bars. This relatively early drop in stiffness and lower lateral strength can be attributed to the presence of lap splices together with the lower column axial loads in these specimens. Measured storey drift ratios for all strengthened specimens at

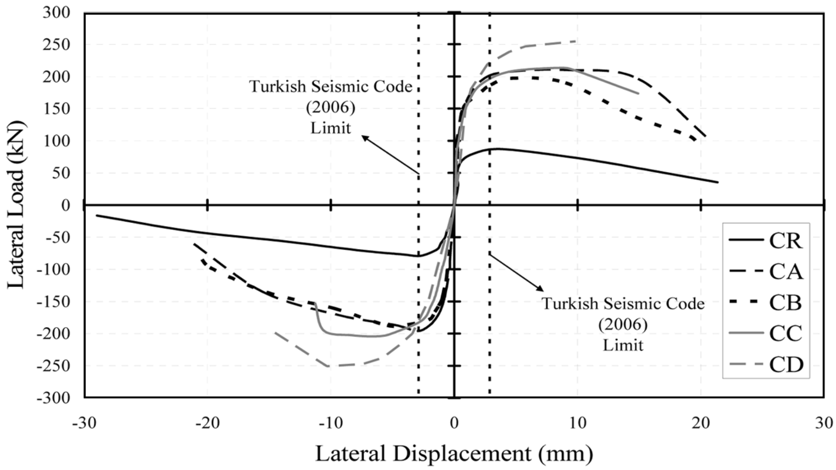


Fig. 11 Response envelopes of specimens having continuous column bars

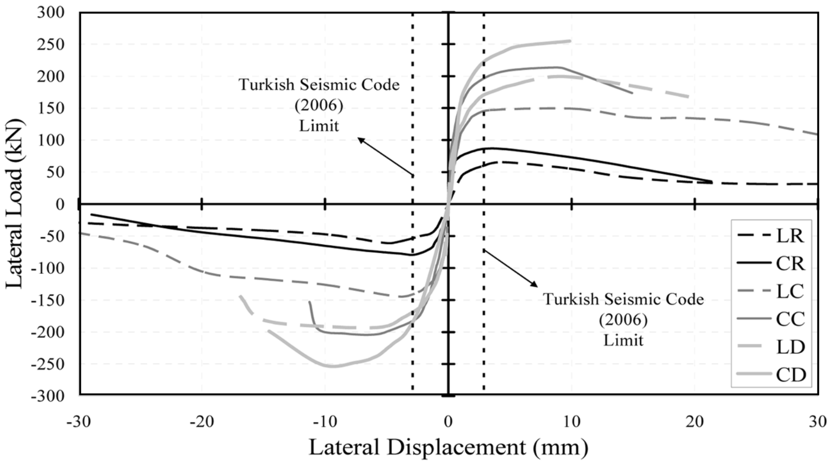


Fig. 12 Comparison for response envelopes of specimens having lap-splices with specimens having continuous column bars

ultimate load were greater than the required drift ratio suggested by the Turkish Seismic Code.

The initial stiffness values of specimens are given in Table 5. The initial stiffness of a specimen was calculated by using the slope of the linear part of the first forward load excursion and is by no means related to the actual initial stiffness of the test frame. It was used as a relative indicator in improvement of the rigidities of test frames. As it can be seen from the table, PC panels increased initial stiffness of specimens significantly. The increase ranged between 2.88 and 3.26 times greater than that of the reference specimen, 2.66 and 4.69 greater for specimens with lap splices on column longitudinal bars. The values show that initial values of the specimens with continuous column longitudinal bars increased about three times after strengthening. It was noted that Specimens CA and CB, which had welded connections between panels, showed the highest initial stiffness. Therefore, welding and shear keys prevent relative displacement between the panels and make the infill stiffer. Specimens CC and CD had less initial stiffness about 5.7% and 10.4% as compared to Specimen CA and CB since they do not have welding and shear keys between precast panels. Presence of lap splices and lower axial load seems to decrease initial stiffness from the comparison of CR and LR specimens. Specimen LC had provided good increase in initial stiffness with respect to the reference Specimen LR, but its stiffness was about 60% of the initial stiffness of Specimen CC, the corresponding specimen without lap splices. Specimen LD showed an initial stiffness almost the same as specimen CD and an increase with respect to the reference specimen. This unexpected high initial stiffness of Specimen LD can be attributed to the quality of workmanship of the hollow brick infill construction and good interaction between the frame and Type D panels.

4.2 Energy dissipation capacities of test specimens

Improving energy dissipation capacity of a frame is one of the major aims of the strengthening technique. It is also an important indicator of the improved seismic behavior. The amount of dissipated energy was determined by adding the areas under the lateral load-second story level displacement curves for each cycle. It is important to note here that the energy dissipation characteristics of the test frames depend on the loading history. The loading histories of the test frames were intended to be the same, but when the response of a test frame became non-linear, both backward and forward half cycle loadings were controlled by second story level displacements. The same second story level displacements were intended to be reached for the forward and backward half cycles.

The increase in the energy dissipation capacities of the strengthened specimens ranged between 1.44 and 3.42 times which means that the proposed technique improve the energy dissipation characteristics of the test frames. It is important to note that Specimens CC and CD dissipated less energy as compared to the remaining strengthened specimens and this behavior can be attributed to the number of inelastic displacement cycles with large amplitude.

The total amount of dissipated energy of each specimen is tabulated in Table 5. For all panel types, strengthening increased the total dissipated energy considerably. The increase is about 1.5 to 2.5 times with respect to the reference test. The highest energy was dissipated by specimens strengthened with type A and type B panels. The effective energy dissipation of these specimens was due to the presence of welding between the panels. Specimens CC and CD dissipated much less energy compared to CA and CB specimens, since they were not welded or had shear keys. The lowest energy dissipation was obtained from CD, which is even less than that of lap-spliced reference specimen. Low energy dissipation of this specimen resulted from crushing of the infill. Specimens LC and LD dissipated more energy than Specimens CC and CD, equivalent specimens with continuous column longitudinal reinforcement. The dissipated energy is nearly as much as Specimens CA and CB. Lower column axial loads was the main reason for relatively high dissipation capacities.

4.3 Comparison of experimental and analytical results

In the analytical studies, plastered hollow brick infills and PC panels were modeled firstly by means of two different equivalent diagonal compression struts as elastic-brittle bars with no tensile resistance. Axial load capacity of the compression strut, F_{strut}, modeling the infill strengthened by bonding PC panels can be computed using Eq. (1)

$$F_{strut} = F_{infill} + F_{panel} \tag{1}$$

where F_{infill} is the axial load capacity of the diagonal compression strut to model the plastered hollow brick infill wall and F_{panel} is the axial load capacity of the diagonal compression strut to model the whole panel made of smaller separate panel units.

Material properties of the plastered hollow brick infills, used in the analytical calculations, were obtained from testing of infill panels under diagonal compression, conducted in the laboratory. From these tests, strength and modulus of elasticity for the plastered hollow brick infills were obtained as 5.0 MPa and 7,500 MPa with low variability, respectively. Dimensions of the strut, namely thickness, width and axial rigidity of the compression strut were determined according to FEMA 1998. Representative values for the axial load capacity and axial rigidity of the first strut were taken as 70 kN and 70 kN/mm.

For the determination of the lateral load carrying capacity of the second strut modeling the RC panels, the formulation given by Eq. (2), obtained from drawing best-fit curves by nonlinear push-over analysis carried out by Baran *et al.* (2010), was used

$$F_{panel} = 7 \cdot \sqrt[4]{f_{c,panel}} \cdot b_w \cdot w \qquad (2)$$

where $f_{c,panel}$ is the concrete compressive strength of PC panel, b_w is the thickness and w is width of the second strut. In this study, b_w and w are calculated by using a method proposed by Smith (1962, 1966, 1967, 1968) and Smith and Carter (1969). According to this method, b_w is taken as 20 mm (real thickness) for all PC panels.

A second analytical model, as an alternative to the to the equivalent diagonal compression strut method, was also developed. In this alternative method, the whole strengthened frame section is defined as a single column. An equivalent thickness was taken into account instead of the thickness of the whole reinforced hollow brick infill wall for each strengthened specimen to form the interaction curves. For interaction curves, the equivalent thickness was calculated by using Young's Modulus of each layer. In the calculations, modulus of elasticity of the plastered hollow brick infill was taken as 7,500 MPa and modulus of the elasticities of the frame and panel concrete were calculated according to Eq. (3)

$$E_c = 4750 \cdot \sqrt{f_c} \text{ (MPa)} \qquad (3)$$

While forming the interaction curves, the mesh steel used for panel reinforcement was taken into account. For push-over analysis, the Young's Modulus of each infilled wall section was decreased by a factor of 0.70, in order to consider the effects of cracks in the early cycles, Also, reduced yield stress for column longitudinal bars with lap splices, as calculated by using Eq. (4) proposed by Canbay and Frosch (2005), was used

$$f_y' \cong f_y \cdot \sqrt{\frac{20\phi}{40\phi}} = 0.7071 \cdot f_y \qquad (4)$$

The values obtained from push-over analyses were compared with the experimental results in Table 6. Lateral load capacities of strengthened specimens were calculated within acceptable closeness. The ratio of experimental to analytical ones varied between 0.76 and 1.28 for equivalent compression strut method and between 0.93 and 1.24 for equivalent column method. Initial stiffnesses calculated from pushover analyses with the first method were close to the experimental

Table 6 Comparison of experimental and analytical values

Specimen	Ultimate Load (kN)			Initial Stiffness (kN/mm)	
	Experimental	Analytical$^{(1)}$	Analytical$^{(2)}$	Experimental	Analytical$^{(1)}$
CA	209.9	195.7	225.8	312.4	291.6
CB	197.0	202.3	218.1	308.0	296.5
CC	213.5	199.1	239.5	294.0	291.7
CD	254.7	199.6	236.8	275.8	292.9
LC	148.9	195.1	184.8	159.3	202.1
LD	199.6	195.1	202.9	280.4	238.8

$^{(1)}$Ratio of the experimental data to the analytical data (Equivalent Diagonal Compression Strut Modeling)
$^{(2)}$Ratio of the experimental data to the analytical data (Equivalent Column Modeling)

values for all specimens. The ratio of experimentally obtained initial stiffnesses to those computed analytically by using equivalent compression strut method varied between 0.79 and 1.17. Initial stiffness values obtained by using the equivalent column method overestimated the experimental ones, as expected.

5. Conclusions

In the study presented, 1/3 scale, one-bay, one-story nonductile RC frames with hollow brick infills were strengthened by bonding high strength PC panels and experimentally investigated under reversed-cyclic lateral loading. The following conclusions can be drawn in the light of the eight experiments conducted;

1. The proposed technique developed for seismic strengthening of RC framed buildings, namely converting the existing hollow brick infill wall into a load carrying system acting as a cast-in-place RC infill wall by bonding relatively thinner high strength PC panels on to the plastered hollow brick infills significantly increased the lateral load capacity and rigidity and improved the seismic behaviour of the test specimens. The proposed method also does not interfere with the function of the building and have little influence on occupants. Therefore, it can be defined as "occupant-friendly".
2. Test results obtained from specimens strengthened by panels connected only by the use of epoxy mortar were so successful that both shear keys and welded connections were not needed. Therefore, Type C and Type D panels can be used instead of Type A and Type B panels which have shear keys and an intensive labor. Hence, the method was much more practical and economical.
3. The use of anchorages between PC panels at four sides of the infill is essential.
4. The strengthened infill failed by excessive diagonal cracking of the panels, and the frame failed by crushing or failure at the column bases or at the beam-column joints. After the failure of the infill, the behavior of the system became similar to that of a frame. Stronger infills provided higher lateral load capacity, but hampered frame action, thus, limiting the ductility.
5. Lateral strengths of specimens with continuous column longitudinal bars increased by 2.3 and 2.9 times compared to reference specimen. The increase in lateral stiffness for the same specimens were 2.9 and 3.3 times.
6. Ultimate lateral load increase for Specimens LC and LD were 2.3 and 3.1 times and stiffness increased 2.7 times and 4.7 times, respectively. Lower axial load and presence of lap splices in column longitudinal bars created a negative effect on the lateral strength. Bar slip problems were observed in specimens with lapped-spliced column reinforcement.
7. Story drift ratios of all specimens at ultimate load were obtained to be greater than the 0.35% limit value suggested by the Turkish seismic code. There was no significant degradation in lateral load carrying capacities of all specimens up to this limit value. Furthermore, stiffness degradation is more pronounced for specimens with lap splices and lower column axial loads.
8. Hollow brick infills strengthened by PC panels were modeled by means of equivalent diagonal compression struts and by equivalent columns for analytical studies. From lateral load carrying capacity calculation point of view, push-over analysis made by both methods gave sound results. In addition, acceptable results were obtained analytically by calculating initial stiffness values of strengthened specimens using equivalent diagonal compression strut modeling. However, initial

stiffness values obtained by using the alternative method, namely equivalent column method, overestimated the experimental values. The authors believe that the proposed analytical approaches provide valuable results and help designers in simulating the behavior of a complicated composite structure.

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Notations

A_c	:	Gross cross-sectional area of the section
A_{st}	:	Total cross-sectional area of the longitudinal reinforcements in the section
f_c'	:	Specified compressive strength of the concrete
f_y	:	Yield strength of reinforcing bars
h	:	Height of the first story
δ	:	First story level displacement at a specified load level
N	:	Axial load applied to each column
N_0	:	Axial load capacity of the column section
ϕ	:	Diameter of the column longitudinal bars (8 mm in this study)
F_{strut}	:	Axial load capacity of the diagonal compression strut to model the PC panel strengthened infill
F_{infill}	:	Axial load capacity of the diagonal compression strut to model the plastered hollow brick wall
F_{panel}	:	Axial load capacity of the diagonal compression strut to model the whole panel made of smaller separate panel units
$f_{c,panel}$	:	Concrete compressive strength of PC panel
b_w	:	Thickness of the equivalent diagonal compression strut to model the whole panel made of smaller separate panel units
w	:	Width of the equivalent diagonal compression strut to model the whole panel made of smaller separate panel units