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Research Article

BEHAVIOR OF R/C FRAMES WITH CONCRETE PLATE BONDED INFILLS

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ABSTRACT
A practical, economic and effective as well as occupant friendly seismic strengthening technique had been developed for reinforced concrete (RC) framed buildings lacking sufficient lateral stiffness. In this technique, high strength concrete plates are bonded onto the existing plastered hollow brick infill walls using a thin layer of epoxy mortar in order that infill walls are converted into lateral load resisting shear walls resulting from the composite action of infill wall with the plates bonded onto it. By this way, the building gains sufficient lateral stiffness. To analyze the behavior of RC frames strengthened by the aforementioned technique, results of eight one-third scale, one-bay, one or two storey deficient RC frames tested under reverse-cyclic lateral loading until failure are given in detail. Three different types of plates were used to strengthen the frames. Test results showed that the proposed strengthening technique considerably increased the lateral load capacities as well as the initial stiffness and energy dissipation capacities of the strengthened specimens, for both types of frames. Additionally, present study focuses on the comparison results of one-storey specimens with those of equivalent two-storey specimens to well-understand the behavior of such strengthened frames under lateral load, and infill walls under compressive and shear forces as well as tensile forces.

Keywords: *Epoxy Mortar, High Strength Concrete Plate, Reverse-cyclic Lateral Loading, Seismic Strengthening, Shear Force*

1. INTRODUCTION

Although Turkey is a seismically active country, most of the existing RC residential buildings are non-engineered, lacking sufficient lateral stiffness. They endanger public safety as well as country's economical well-being in a possible future earthquake. So, they should immediately be strengthened after a vulnerability assessment process. Up to past few years, researches concentrated on studies in which new shear walls were added into the building's RC frame. Although this method has been proved to be very effective structurally, it has a disadvantage which cannot be undervalued. This method requires evacuation of the building since huge volumes of construction material have to be carried into the building as well as a long construction period. The technique presented in this study promise a non-evacuation strengthening, even without causing more disturbance to the occupants than a painting job where the idea is to convert the existing hollow brick infill wall into a load carrying system acting as a cast-in-place concrete shear wall by reinforcing it with relatively thinner high strength PC plates to be bonded to the plastered infill wall and connected to the frame members by epoxy mortar.

In the experimental researches conducted by various researchers (Yuzugullu, 1979; Kahn and Hanson, 1979; Hanson, 1980; Kaldjian and Yuzugullu, 1983; Phan *et al.*, 1995; Nakashima, 1995; Roberts, 1995; Frosch, 1996, 1999; Frosch *et al.*, 1996; Li, 1997; Matsumoto, 1998; Han *et al.*, 2003; Isao *et al.*, 1999; Kanda *et al.*, 1998; Kesner, 2003; Kesner *et al.*; 2001, 2003, 2005) plates had been used as strengthening elements. These studies showed that the use of plates is an effective and convenient method which increases strength and stiffness of RC frames considerably, whereas it saves cost and time.

In the content of the present project, nearly fifty tests had been conducted on RC frames strengthened by PC plates. Current study presents the comparison of behaviors of eight RC frames strengthened by bonding RC plates on to infills.

2. EXPERIMENTAL INVESTIGATION

2.1. Test Frames

One-third scale, one-bay, one-storey and two-storey RC frames with common structural deficiencies observed in real practice in Turkey were used as test frames. Such deficiencies are insufficient lateral stiffness, non-ductile members, bad detailing and low concrete quality. Dimensions and reinforcement of the test frames are illustrated in Fig. 1.

2.2. Materials

Low strength concrete was intentionally used in the test frames to represent the concrete commonly used in existing building structures whereas relatively strong concrete is preferred for the PC plates to provide the required load carrying capacity by using relatively thin layer of concrete, minimizing the plate weight. For the same reason, mild steel plain bars were used as longitudinal reinforcement in beams and columns. Typical properties of reinforcing bars used in this study are listed in Table 1.

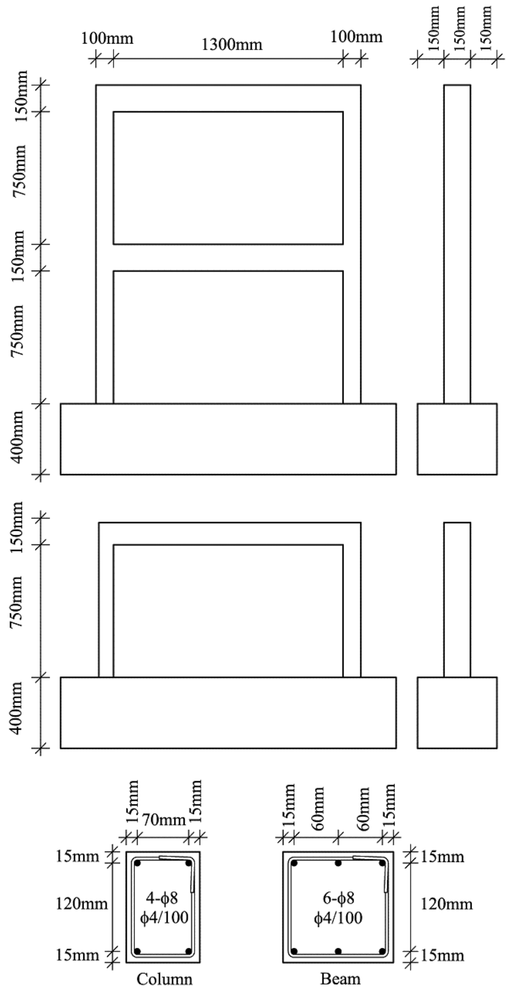

Fig. 1. Dimension and reinforcement of frames

Table 1. Properties of Reinforcing Bars

Bar	Property	Location	f_y (MPa)	f_{ut} (MPa)
ϕ3	Plain	Mesh steel for plate reinforcement	670	750
ϕ4	Plain	Stirrup for beam and column Plate reinforcement	220	355
ϕ6	Deformed	Dowel for frame-to-plate connection	580	670
ϕ8	Plain	Beam and column longitudinal bars	330	445
ϕ8	Deformed	Anchorage bar between adjacent plates	350	470
ϕ16	Deformed	Foundation beam longitudinal bar	420	580

Both the first storey and second storey bays were infilled with scaled (one-third scale) hollow brick infill covered with scaled layer of plaster at both faces. Ordinary cement-lime mortar with ordinary workmanship was used during the plaster application and wall construction, reflecting the usual practice. The hollow bricks used in this study is illustrated in Fig. 2.

Because of the superior compressive and tensile strength, Sikadur-31 was preferred to be used as epoxy mortar having a compressive strength of 65 MPa used in plate joints and between the plates and the plaster on the wall. According to the manufacturer's manual, its tensile strength is nearly 20 MPa, its adhesion strength to steel and concrete is 30 MPa and 3.5 MPa, respectively. For the embedment of the dowels to the frame members, Spit Epcon was used as epoxy in the present study.

2.3. Infill Wall Strengthening Plates

Within the study reported, three different types of PC plates having two basic shapes were designed and tested to observe their performances as infill wall strengthening elements. One approach was to have rectangular shaped plates by arranging them in three rows and four columns, and another was to have full height strip shaped plates by using the full height of the infill in several lines. Plate thickness for all types was chosen as 20 mm. Therefore, the plates came out to be about 3 kg in weight. This weight is for one-third scale plates, so the corresponding weight for the actual sized plates would be about 80 kg, which is not too heavy to be carried manually by two workers. The plates "I" and "III" have nearly square geometry whereas the plate "II" has full height strip geometry. The method of bonding plates on to infill is illustrated in Fig. 3.

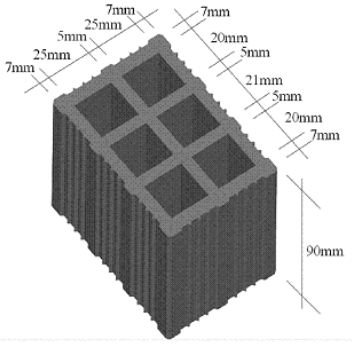

Fig. 2. Hollow Brick Dimensions

3. TEST SET-UP AND INSTRUMENTATION

3.1. Loading and Supporting System

As it can be seen in Fig. 4, the setup consists of a universal base, test specimen, loading system and a reaction wall. The main foundation was fixed to the strong floor and the specimens were fixed on top of the main foundation by steel bolts.

Specimens were placed inside a steel frame which was fixed to the main frame. The steel frame was also supported by the laboratory wall by L-section steel bars forming a steel frame which was intended to prevent out-of-plane deformations, i.e., torsion of the specimen by providing lateral support to the beam(s) with rollers.

During the tests, a constant axial load was applied on to the columns of the specimens. The load was applied by two hydraulic jacks on both sides of the specimens and the load was transferred to the cross beam by cables. The axial load applied on both columns, equal to 60 kN (~20% of column's axial load capacity), were continuously and carefully monitored and adjusted manually.

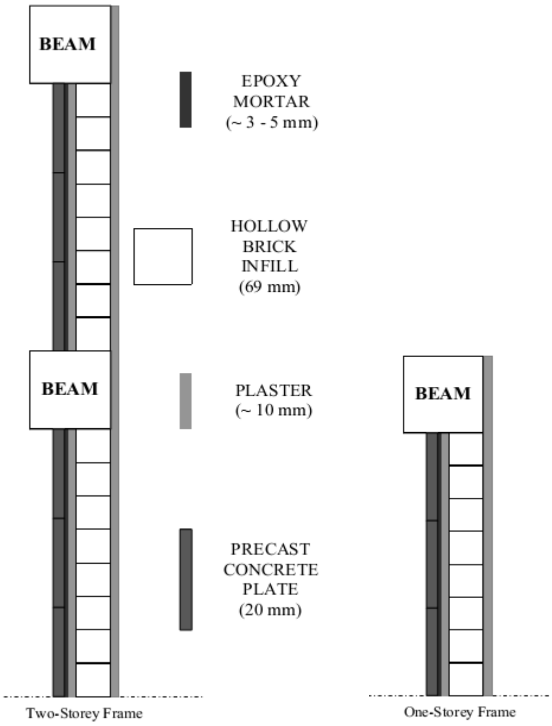

Fig. 3. Plate Bonding Method

The specimens were tested with hysteretic lateral loading for modelling ground motion effect. The system consisted mainly of an adjustable steel frame attached to the reaction wall, a load cell, a hydraulic pump and pin connections at either end of the loading column consisting of the jack and load cell. Loading was applied by the assemblage of a hydraulic jack and a load cell with pin connections at the ends. The pin connections at the ends provide the system to create axial stress only. The lateral load was planned to act in two directions; pushing and pulling. To achieve this aim by a loading system at one side, steel plates were attached to both ends of the beam with four steel bars. For one-storey specimens, lateral load was applied at first storey beam level. For two-storey specimens, lateral load was applied at one third span of the spreader beam to ensure that it was divided in 1:2 ratio between the first and second story beam levels. In the tests, the loading was applied in cycles. Each load level was repeated in reverse direction before proceeding to the next load level.

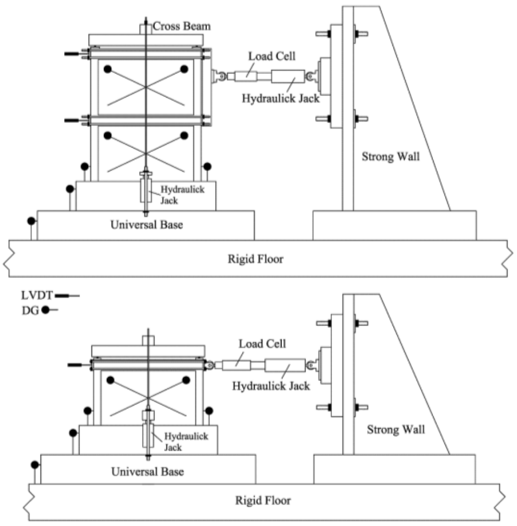

Fig. 4. Test Set-ups and Instrumentation

3.2. Deformation Measurement System

In order to record deformations, linear variable displacement transducers (LVDT) and dial gauge type measurement devices were used. The positions of the gauges are shown in Fig. 4. The lateral load was being recorded by a load cell throughout the tests.

Lateral displacement of stories was measured with respect to the universal base. The readings from the LVDTs were used to construct load-displacement, load-story drift and load-infill shear displacement curves.

Shear deformations were measured on infill walls by means of two diagonally placed dial gauges. Transducers were located 130 mm away from the corner of the infill walls. The reason for choosing this location was to avoid localized effects like crushing of concrete during experiment.

4. EXPERIMENTAL RESULTS

4.1. Behavior of Test Frames

In this section, test results of one-storey specimens are compared with the test results of equivalent two-storey specimens. Behavior of equivalent pairs are compared with respect to ultimate strength and interstorey drift ratio characteristics to analyze the similarities and differences between the pairs.

4.1.1. Reference Frames, CR

CR denotes unstrengthened specimens which had continuous reinforcement and only plastered hollow brick infill. The two test frames had similar concrete strength. From Table 2, it can be seen that the one-storey frame had a lateral load capacity of 86.6 kN which was greater than that of equivalent two-storey frame with a value of 76.8 kN. Corresponding first story drift ratio values at the maximum forward load were 0.0037 and 0.0042 for one-storey and two-storey CR frames respectively. But when loading in the negative direction is considered, one-storey and two-storey frames had lateral load capacities of 79.1 kN and 78.8 kN, respectively. Corresponding first story drift ratio values at the maximum backward load were 0.0033 and 0.0030 for one-storey and two-storey CR frames respectively. Both frames exhibited typical masonry infilled frame behavior. The plastered hollow brick infill considerably contributed to the lateral load capacity at the initial stage, however, this contribution decreased rapidly as crushing started in the infill leading to a behavior similar to that of the bare frame where significant deformations took place under rather small lateral loads. The closeness of the capacity and corresponding first storey drift ratio values can be observed in the comparison of load-displacement curves and comparison of response envelopes, as shown in Fig. 5.

In the tests, diagonal cracking started earlier on the first storey infill of the two-storey frame than that of the one-storey frame. First hairline cracks on columns were observed together with the cracks on the infill at a lateral load capacity of 50 kN. This situation was observed in the test of one-storey frame at a lateral load capacity of 80 kN. For both frame types, infill started crushing from the corners and the lateral load started to be carried by the frame members. Transformation of behavior into bare frame action occurred about the eighth cycle for both frames, at a lateral load capacity of nearly 60 kN, where half cycle loadings were controlled by the top storey level displacement. The behavior was very similar considering the first stories and the failure of the frames occurred by crushing at column bases. Photographs of both first story infills of both reference frames are given in Fig. 6., where the upper one is of two-storey frame and the lower is one-storey.

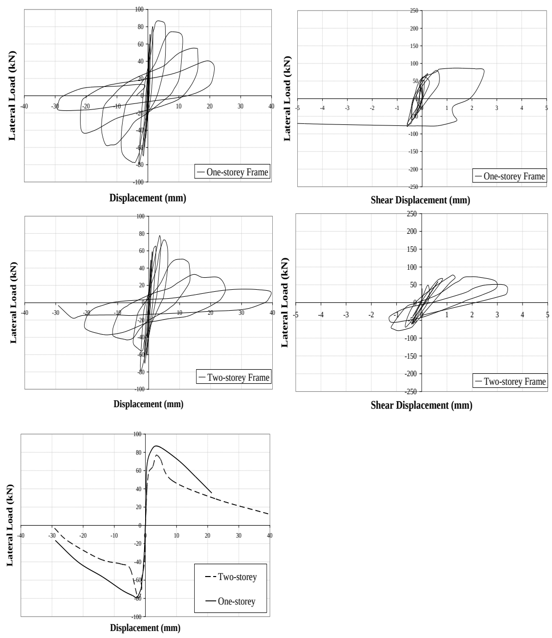

Fig. 5. Comparison of Lateral Load vs. First Storey Displacements of both CR Specimens

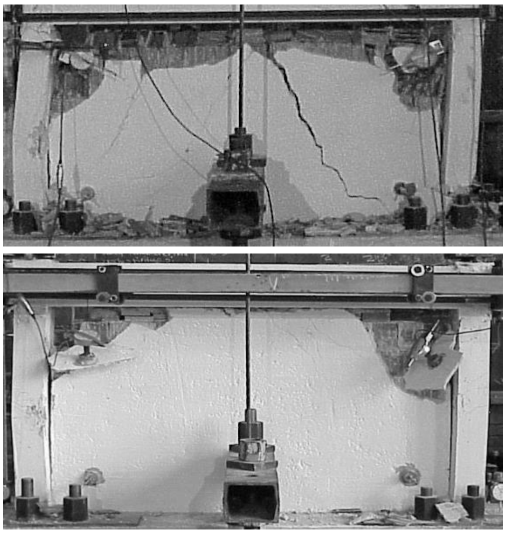

Fig. 6. Photographs of first storey infills (CR Specimens)

4.1.2. Frames Strengthened by Using Type I Plates, CI

Both CI Specimens were strengthened by using Type I PC plates The plate arrangement for both CI specimens is shown in Fig. 7. The lateral load capacities cannot be considered as close values since they were 209.6 kN and 186.2 kN for one-storey and two-storey frames, respectively. However, they were especially close for negative direction, which were 196.0 kN and 192.5 kN, respectively for one-storey and two-storey frames. For forward loading, first storey drift ratio values at the maximum load were 0.0120 and 0.0037, respectively for one-storey and two-storey frames. These ratios were 0.0038 and 0.0069 for backward loading. During testing of two-storey CI specimen, the entire test unit behaved nearly as a monolithic cantilever where failure took place at the base level in terms of yielding of the steel in the tension side column and concrete crushing and buckling of longitudinal steel in the compression side column. However, at the end of one-storey frame test, it was observed that some damage occurred in the infill; however, eventual exhaustion came with the failure of columns.

For both frames, the proposed technique led to a significant increase in the capacity and thus to a significantly better seismic performance meaning an effective behavior of plate strengthened hollow brick infill. In addition, one storey frame had more ductility as can be observed from the comparison of envelope curves given in Fig. 8. The difference in ductility is significant in the forward direction, but it does not seem to be much different in the backward direction. One-storey test frame have an envelope curve which is more wider, possibly caused by the pinching effect resulting from the large cracks developed on the infills. The more wider loops of one-storey specimen can easily be seen in the lateral load-top displacement graph given in Fig. 8. Photographs of both first story infills of both specimens are given in Fig. 9, where the upper one is of two-storey frame and the lower is one-storey.

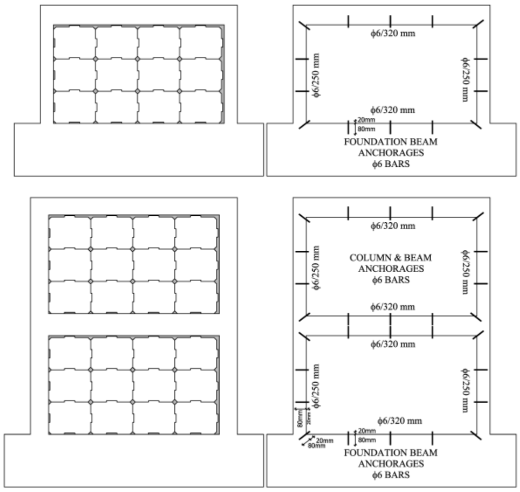

Fig. 7. Plate Arrangement for both CI Specimens

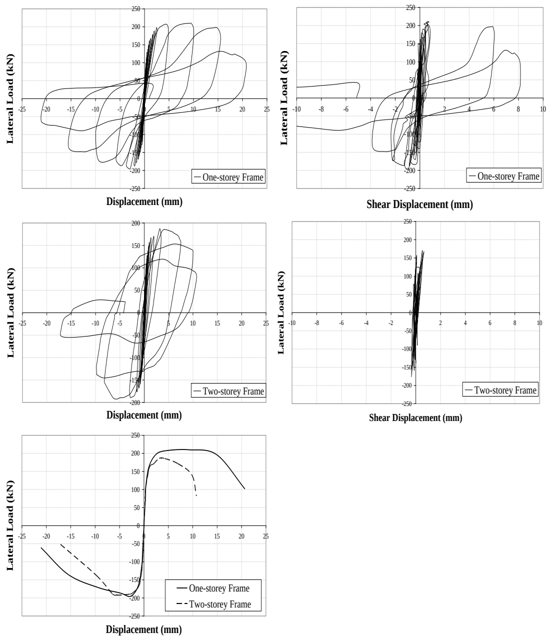

Fig. 8. Comparison of Lateral Load vs. First Storey Displacements of both CI Specimens

4.1.3. Frames Strengthened by Using Type II Plates, CII

One or two storey frames strengthened with Type II plates were denoted as CII. The plate arrangement for both CII specimens is shown in Fig. 10. When the values in Table 2 are observed, it can clearly be seen that both types of frames showed significantly similar behavior for both loading directions in the perspective of lateral load capacity and response envelopes. One-storey and two-storey CII specimens reached lateral load capacities of 197.0 kN and 201.3 kN, respectively for forward loading and 187.4 kN and 198.2 kN, respectively for backward loading. Corresponding first storey drift ratio values were 0.0056 and 0.0089, respectively for one-storey and two-storey test frames for forward loading and 0.0066 and 0.0070, respectively for backward loading. The response envelopes seem to be almost coinciding with each other. There is some difference between the curve shapes in the load-deformation plots of two frame types. As in the cases of both CI specimen tests, the entire two-storey specimen CII behaved nearly as a monolithic cantilever whereas it was observed that some damage occurred in the infill of the one-storey specimen CII. As compared to one-storey CI Specimen, this specimen has less narrow cracks and hence, less pinching effect most possibly stemming from this phenomenon. Lateral load-top displacement graphs for both specimens are given in Fig. 11.

Fig.9. Photographs of first storey infills (CI Specimens)

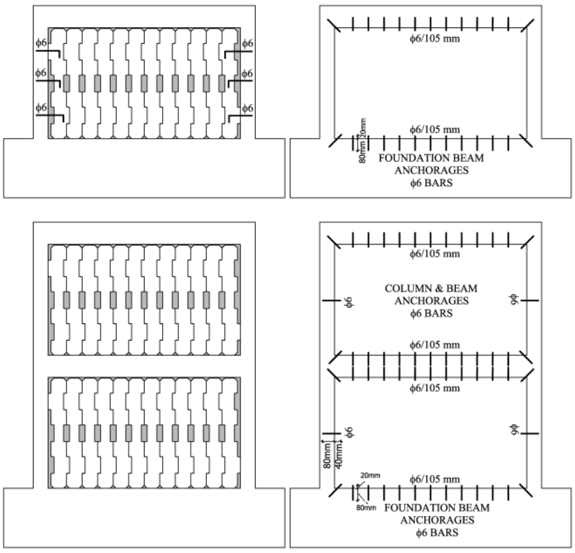

Fig. 10. Plate Arrangement for both CII Specimens

For both frames, cracking started at column bases at the same load level. Then, infill separation cracks were formed, a little later at the two-storey frame. Beam-column joint cracks appeared earlier and were more significant at the one-storey frame. At the two-storey frame, column flexural cracks were observed before joint cracking. At the one-storey frame, plate cracking started at later cycles, and the infill was separated from the frame. The two tests ended similarly by damages in the column bases, but the one-storey frame had much greater damage at the joint region whereas two-storey frame had greater damage at the base level. Test results for both frame types are given in Table 2. Photographs of both first story infills of both specimens are given in Fig. 12, where the upper one is of two-storey frame and the lower is one-storey.

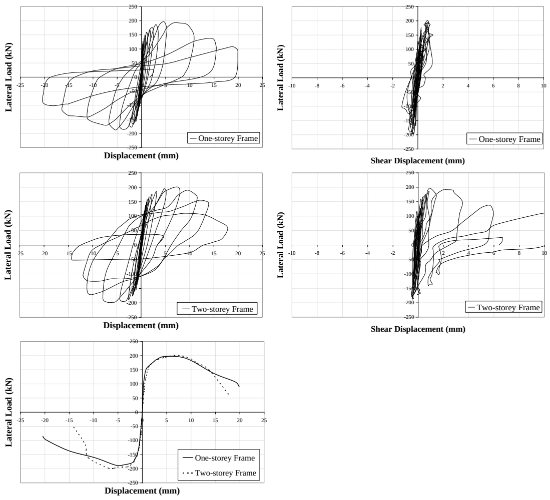

Fig. 11. Comparison of Lateral Load vs. First Storey Displacements of both CII Specimens

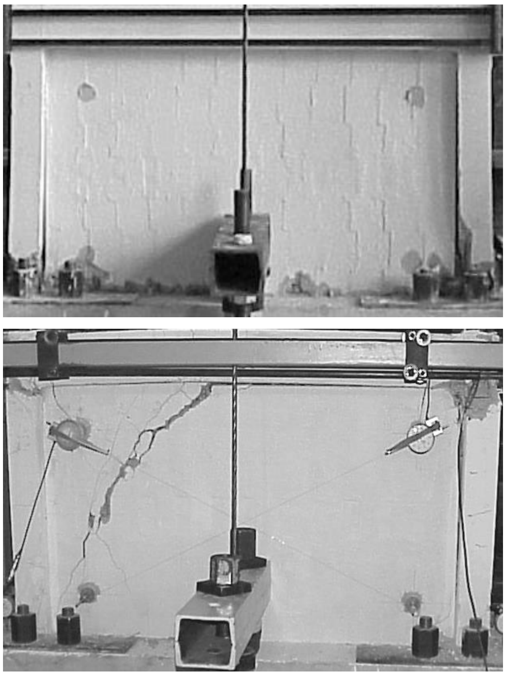

Fig.12. Photographs of first storey infills (CII Specimens)

4.1.4. Frames Strengthened by Using Type III Plates, CIII

CIII specimens were Type III plate strengthened frames with anchorage to all frame members in the first storey. The plate arrangement for both CIII specimens is shown in Fig. 13. Two frame types showed similar response as can be observed from the values given in Table 2 and response envelopes given in Fig. 14. For forward loading, one storey and two-storey test frames reached a lateral load capacity of 213.5 kN and 212.9 kN when the first storey drift ratio values were 0.0103 and 0.0055, respectively. For backward loading, they reached maximum load values of 204.0 kN and 218.5 kN at drift ratios of 0.0071 and 0.0062, respectively. One-storey specimen has higher ductility than the two-storey specimen, especially in the positive direction. In the negative direction, maximum lateral displacement of two specimen types seems to be similar. It can be noted here that, one-storey specimen has a little wider response envelope than the equivalent two-storey specimen which can possibly be attributed to the pinching effect stemming from the cracks occurred on the infill of the one-storey specimen.

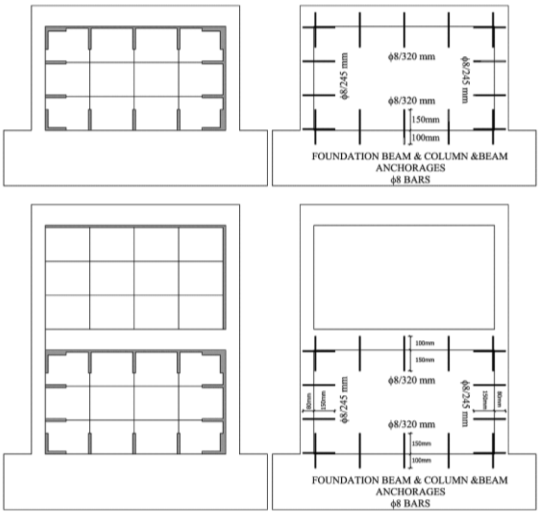

Fig. 13 . Plate Arrangement for both CIII Specimens

Separation of the infill from the columns and cracking at the column bases started in early cycles of both tests. Also, diagonal cracking on the plates started at the same load level for both cases. However, beam-column joints cracked at the one-storey frame before plate cracking, but at the two-storey frame, beam-column joint cracks occurred after plates started cracking. In the last cycles of both tests, column bases started to crush and the cover concrete dispersed. The two-storey frame failed from the column base crushing. On the other hand, the one-storey frame, also having significant damage at the column bases, failed by crushing at the beam-column joint suddenly. One-storey frame had much more wider cracks on the plates. Test results for both frame types are given in Table 2. Photographs of both first story infills of both specimens are given in Fig. 15, where the upper one is of two-storey frame and the lower is one-storey.

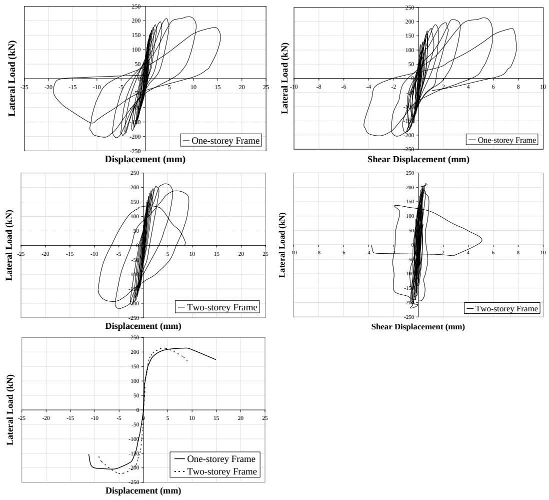

Fig. 14. Comparison of Lateral Load vs. First Storey Displacements of both CIII Specimens

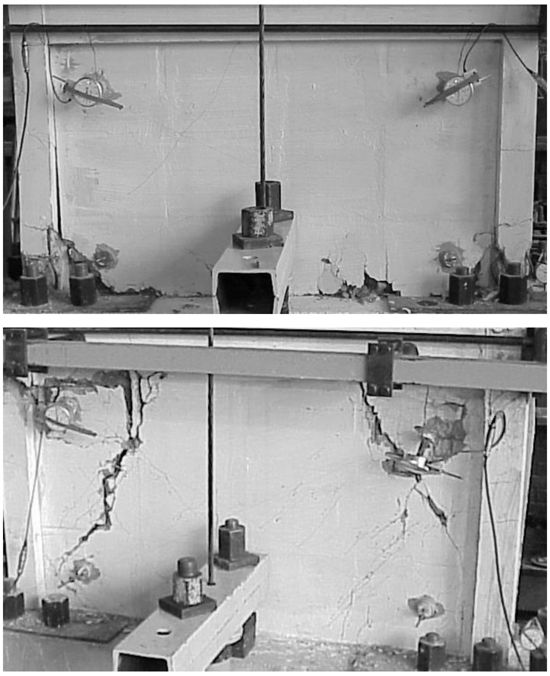

Fig. 15. Photographs of first storey infills (CIII Specimens)

4.2. Initial Stiffness Values of Test Frames

The initial stiffness of a specimen was calculated by using the slope of the linear part of the first forward load excursion. It was used as a relative indicator in improvement of the rigidities of test frames. As it can be seen in Table 3, all one-storey specimens have higher initial stiffness values than the equivalent two-storey specimen. This situation can be owing to the fact that more compressive and shear stress occuring in the infill of the one-storey test frame leading to more stiffer infill. Instead, more tensile stress occurs at the tension side column of two-storey frames resulting from the more overturning effect of the greater moment arm leading to less rigid specimen in early cycles.

Table 2. Test Results of all Specimens (Ratio of the value of one-storey test frame to that of two-storey test frame)

Sp.		One-St.	Two-St.	Ratio
CR	f_{ck} (MPa)	15.6	16.6	-
	Max. (+) Load	86.6	76.8	1.13
	(δ_1/h_1)	0.0037	0.0042	0.88
	Max. (-) Load	79.1	78.8	1.00
	(δ_1/h_1)	0.0033	0.0030	1.10
CI	f_{ck} (MPa)	18.7	18.2	-
	Max. (+) Load	209.6	186.2	1.13
	(δ_1/h_1)	0.0120	0.0038	3.16
	Max. (-) Load	196.0	192.5	1.02
	(δ_1/h_1)	0.0037	0.0069	0.54
CII	f_{ck} (MPa)	12.2	13.0	-
	Max. (+) Load	197.0	201.3	0.98
	(δ_1/h_1)	0.0056	0.0089	0.63
	Max. (-) Load	187.4	198.2	0.95
	(δ_1/h_1)	0.0066	0.0070	0.94
CIII	f_{ck} (MPa)	14.2	19.4	-
	Max. (+) Load	213.5	212.9	1.00
	(δ_1/h_1)	0.0103	0.0055	1.87
	Max. (-) Load	204.0	218.5	0.93
	(δ_1/h_1)	0.0071	0.0062	1.15

Table 3. Initial Stiffness Values of all Specimens (Ratio of the value of one-storey test frame to that of two-storey test frame)

Sp.	Initial Stiffness (kN/mm)		
	One-St.	Two-St.	Ratio
CR	95.8	64.7	1.48
CI	312.4	275.9	1.13
CII	308.0	197.6	1.56
CIII	294.0	196.1	1.50

4.3. Shear Deformations in the First Storey Infill Walls of Test Specimens

Lateral load-first storey shear displacement curves of all specimens are presented in Fig. 6, Fig. 9, Fig. 12 and Fig. 15. As seen in these figures, there was a visible shear deformation on the first storey of Reference Specimens CR. After introducing PC plates, the shear deformation due to base shear reduced in first storey plates. PC plates behaved rigidly so that they prevented excessive shear deformations. In the case of comparing the equivalent strengthened specimen pairs, shear displacement in the infills of two-storey specimens are less with the respect to that of one-storey specimens, as expected. This is more visible especially in the case of Specimens CI and Specimens CIII. In the case of Specimens CII, less crack formation on the first storey infill of one storey specimen may lead to the less shear deformation difference between this equivalent pair.

4.4. Energy Dissipation Capacities of Test Specimens

When a structure deforms, the work done is stored as strain energy. Part of this energy is released in the unloading process, whereas the remaining energy is dissipated through different mechanisms.

Energy dissipation capacity is an important indicator of the structure's ability to withstand severe earthquakes. It is also an important indicator of the improved seismic behavior. For both type of test frames, the amount of dissipated energy was determined by adding the areas under the lateral load vs. load application 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.

Total amount of dissipated energy of each specimen is tabulated in Table 4. As it can be seen in this table, all one-storey test frames dissipated less energies with respect to their equivalent pairs except from Specimens CI, for which the dissipated energies by these specimens can be accepted as equal. More pinching effect observed in one-storey test frames seems to be one of the causes for this less energy dissipation.

All strengthened two-storey test frames behaved as a monolithic cantilever, meaning that heavy damage due to reversed cyclic lateral loads concentrated through the column bases together with the infill base. In addition, damage concentrated on the infill and beam-column joints for one-storey specimens, meaning that one-storey strengthened test frames exhibited frame behavior rather than monolithic cantilever.

Table 4. Energy Dissipation Capacities of all Specimens (Ratio of the value of one-storey test frame to that of two-storey test frame)

Sp.	Energy Dissipation (Joule)		
	One-St.	Two-St.	Ratio
CR	5.7	6.4	1.12
CI	15.5	15.3	0.99
CII	15.1	21.8	1.44
CIII	9.2	13.4	1.46

5. SUMMARY OF THE EXPERIMENTAL RESULTS

5.1. Effectiveness of the Proposed Strengthening Technique

Lateral load vs. displacement curves given in previous section indicate a significant increase in the load carrying capacity and a delayed strength degradation, leading to a better ductile behavior when precast concrete plates are bonded on to the plastered infill wall. An overall interpretation of the test results are summarized in Table 5 in terms of load carrying capacity, lateral rigidity, ductility and energy dissipation capacity points of view. Table 5 shows the effectiveness of the proposed strengthening technique.

Table 5. Performance improvement by the proposed technique (for both frame types)

	Relative to reference frame
Lateral load capacity	~2.5 times
Lateral stiffness	~3 times
Ductility	~2 times
Energy dissipation	~3 times

5.2. Comparison of Infill Behavior

During the tests, the behavior of the one-storey frame is very similar to the first storey of the two-storey frame, while the upper storey of the two-storey frame remains with minor damage. Cracking at the frame members and the infill starts and progresses similarly in all cases. After some cracks occur on frame members, diagonal cracks start on the infill. Then, heavy damage concentrates at the column bases and beam-column joints, and following the failure or damage of the infill, the frame members fail at these regions. The main difference of observed damage between the one-storey and the two-storey frames is that the first storey beam-column joint region of the one-storey frame receives much more significant damage than the same place of the two-storey frame.

One-storey and two-storey frames of same application showed very similar behavior. Lateral load capacities of two frame types are very close. One of the main differences is the application level of loading. In two-storey frames, the lateral load was applied at a greater height and therefore moment arm is greater. Greater moment arm of lateral load results in more overturning effect. So, more tensile stress occurs at the tension side column of two-storey frames. Compressive and shear stresses are more dominant in one-storey frames. This is the most possible reason for higher initial stiffness of one-storey frames. Ductility of the two frame types are not largely different, but generally one-storey frames showed higher ductility which can be a result of more efficient behavior of the infill, which can be positively influenced by the confining effect of compressive forces.

When the lateral lateral load-first storey displacement curve for one and two storey frames are compared, it is observed that there is a significant difference at the shape of the loops. Load-displacement curves for one-storey frames have a much more pinched shape than the curves for two-storey frames. This difference is small for the reference specimens. Pinching is the result of higher shear stresses causing larger crack widths, because when the loading is reversed, no stiffness can be observed while the cracks are closing. Therefore, the main reason for more pinching in one-storey frames is the higher level of shear action.

6. CONCLUSIONS

The conclusions presented below are based on the limited data obtained from eight tests conducted;

- The seismic rehabilitation technique reported in the present study significantly increased the lateral load capacity and rigidity as well as improving the seismic behavior of the test frames.
- The effectiveness of the rehabilitation technique can easily be observed from the tests of both one and two-storey frames showing that plate bonded hollow brick infill walls behave efficient and effective under compressive and shear stresses as well as tensile stresses.
- One-storey and two-storey frames of same application showed very similar behavior, especially lateral load capacities of two frame types are very close, indicating that they are equally acceptable as test units and one-storey frames may be preferred in the cases where there exits space, time and testing facility limitations.
- In the tests of one-storey specimens, failure occurred with significant damage in the beam-column joints which indicates that the effectiveness of the proposed technique depends not only on the strengthening plate properties but also on the properties of the existing frame. To have a satisfactory strengthening performance, strengthening of the frame members prior to the introduction of plates will obviously be needed.

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