Available online at www.sciencedirect.com

ScienceDirect

Cement & Concrete Composites 30 (2008) 612–621

Cement & Concrete Composites

www.elsevier.com/locate/cemconcomp

Strengthening of RC nonductile frames with RC infills: An experimental study

Sinan Altin, Özgür Anil *, Mehmet Emin Kara

Gazi University, Civil Engineering Department, Maltepe, Ankara 06570, Türkiye

Received 4 April 2006; received in revised form 14 July 2007; accepted 25 July 2007
Available online 8 August 2007

Abstract

The purpose of this study was to experimentally investigate the behavior of RC nonductile frames with RC infills under lateral loads. A total of six 1/3 scaled one bay, two story nonductile specimens were built and tested under reversed cyclic lateral load. Specimen frames were manufactured with the deficiencies commonly observed in practice. Test results showed that both the strength and the stiffness were significantly improved by the introducing infill. But the deficiency which affected the behavior of infilled frames most adversely was the presence of lap splices in column longitudinal reinforcement. In five of the test specimens' lap splices were introduced at just the floor level and their lengths were much shorter than code requirements. Three different local strengthening techniques were applied to the infilled test specimens to overcome this deficiency. These strengthening techniques included; (a) adding continuous longitudinal reinforcements along two stories in boundary elements of infills, (b) constructing new columns on both sides of infill walls, (c) welding column lap splices. It was observed that these local strengthening techniques prevented local failure in the splice region of the frame columns and improved the lateral strength and stiffness of infilled frame considerably.

Keywords: Strengthening; Reinforced concrete frames; Infill wall; Lap splice

1. Introduction

Most of the existing reinforced concrete residential buildings in many countries are seismically deficient because the load carrying system of these buildings contains flexible columns, soft stories, nonseismic reinforcement detailing and strong beam-weak column connections. Due to the low lateral stiffness of these kinds of frames, the buildings experience large lateral displacements under seismic loads. But the buildings can not sustain these large displacements often due to nonductile columns failure. In general lap splices in column longitudinal reinforcement are located at member ends, in potential plastic hinge regions and the lengths of splices are often shorter than those required by building codes. The confinement of spliced regions is also poor because of insufficient amount of ties with 90° hooks. Therefore, the failure of reinforced concrete columns usually occurs at the bottom where the column longitudinal bars are lap spliced. Consequently, the strengthening of these structures is required for public safety even in the case of a moderate earthquake.

The structural strengthening of a large number of reinforced concrete buildings becomes a challenging task in countries where seismic risk is high. In the past, different techniques were developed for the seismic strengthening of reinforced concrete buildings. Among the available techniques, the addition of reinforced concrete infill walls was found to be the most feasible strengthening technique for reinforced concrete medium rise buildings. In general, strengthening with reinforced concrete infill walls provides superior performance, considerable ease in construction, and economy. Reinforced concrete infill walls improve seismic behavior by increasing lateral strength, initial lateral stiffness, and energy dissipation capacity of reinforced con-

* Corresponding author. Tel.: +90 312 231 74 00/2215; fax: +90 312 231 92 23.
E-mail address: oanil@gazi.edu.tr (Ö. Anil).

0958-9465/$ - see front matter © 2007 Elsevier Ltd. All rights reserved.
doi:10.1016/j.cemconcomp.2007.07.003

Nomenclature

f_c	specified compression strength of concrete	ρ_h	horizontal reinforcement ratio of infill
f_{sy}	yield strength of reinforcing bars	ρ_v	vertical reinforcement ratio of infill
f_{su}	ultimate strength of reinforcing bars		

crete buildings, and limit both structural and nonstructural damages caused by earthquakes. The performance of retrofitted frames may be limited by the premature failure of column splices, unless they are strengthened to develop tensile forces. As a result, expected performance can not be obtained from strengthening with RC infill walls.

In the past extensive experimental research was conducted on reinforced concrete infilled frames. Many different types of infill walls, infill reinforcement arrangements and infill wall frame connections were studied [1–12]. In most of these studies one bay one story infilled frames and one bay two story infilled frames were tested under monotonic or lateral loading. Test results indicated that the infill walls increased the lateral load capacity of the frames and reduced their lateral drift. The research showed that; (a) properties of infills, such as infill material type (masonry brick or reinforced concrete) and reinforcement arrangement, (b) properties of frames, such as the ratio of column flexural reinforcement, column and beam stirrups ratios, and, (c) type and effectiveness of connections between the infill and frame members, affected the behavior of infilled frames significantly. In addition, the test results indicated that the behavior of frame-infill systems was controlled by the failure of splices in existing columns, which acted as boundary elements for infills. Studies showed that the benefits of strengthening nonductile RC frames might be limited due to the failure of splices in existing columns [13]. Feasibility studies on different methods of seismic upgrading were also performed [14,15]. It was concluded that the simplest and the most effective way of improving the behavior of such buildings, in which unsatisfactorily seismic behavior was inherent in the structural system, is to provide an adequate number of structural infill walls. Such infill walls not only increase the lateral stiffness significantly but also relieve the existing frames from carrying large lateral forces.

First, experimental study on strengthening of RC frames with infill walls was carried out in seventies in Turkey. In that study, researches tested nine one-bay one story frames with RC infills under monotonically increasing lateral loads [16]. It was found that the lateral load capacity of the test frames increased by 700% and lateral drift at failure reduced by 65%. Subsequently, one bay two story RC infilled frames were tested under reversed cyclic lateral loading [17]. The optimum infill reinforcement pattern and the connection details between the infill and frame members were determined based on experimental observations. Test results indicated that the infills improve the behavior of reinforced concrete frames significantly under lateral loads. In addition, the test result showed that specimens strengthened with infill walls having orthogonal reinforcement behaved better than others. Site surveys conducted after the 1999 Marmara and Adapazarı earthquakes in Turkey revealed that poor confinement and inadequate splice length at the bottom of the columns were the most important weaknesses causing structural damage and collapse. These studies were followed by additional research on the effectiveness of rehabilitation of damaged nonductile RC frames with RC infill walls [18,19]. In these studies researches tested one bay two story and three bay two story frames. The frames were damaged prior to the application of RC infills, under reversed cyclic lateral loading. Test results showed that the level of damage in the frames had an insignificant effect on the behavior of RC infilled frames. The success of RC infilled frames mainly depended on the connection provided between the frame and RC infill walls, and the confinement of concrete, especially within the splice region of columns. One study indicated that the applied local strengthening technique for preventing local failure in the spliced region of boundary columns, improved the strength and stiffness of infilled frame considerably [20].

The objective of the current investigation was to develop a technique for strengthening poorly lap spliced columns of nonductile RC frames, while also providing RC infill walls. Three different local strengthening techniques were employed. These strengthening techniques included; (a) adding continuous longitudinal reinforcements along two stories in boundary elements of infills, (b) constructing new columns on both sides of infill walls, (c) welding column lap splices. Test of large-scale specimens were conducted under reversed cyclic loading. The results of these tests are presented and discussed in this paper.

2. Experimental program

2.1. Test specimens

A total of six one bay, two story specimens were constructed and tested as part of the experimental program. The properties of specimens are summarized in Table 1. The frames were manufactured to reflect seismic deficiencies observed in many countries of the world. Dimensions and reinforcement details of test frames are shown in Fig. 1. The test frame was a 1/3 scale model of a nonductile frame having weak columns-strong beams. Four 10 mm diameter deformed bars were used as longitudinal column reinforcement. Six 8 mm diameter deformed bars were used

Table 1
Properties of test specimens

Specimens	Frame			Infill[a]		
	f_c frame (MPa)	Splice length	Remarks	f_c infill (MPa)	Horizontal reinforcement #/diameter (mm)	Vertical reinforcement #/diameter (mm)
1	14.8	150 mm	–	Bare frame without infill		
2	15.0	No splice	–	29.8	6/6	9/6
3	15.2	150 mm	–	29.8	6/6	9/6
4	15.0	150 mm	+	29.6	6/6	9/6
5	15.3	150 mm	++	30.2	6/6	9/6
6	14.7	150 mm	+++	30.6	6/6	9/6

Notes: (+) – Infill wall with edge member; (++) – Infill wall with new edge column; (+++) – Welded RC frame column reinforcements.
[a] Infill reinforcement has placed at both of the infill faces.

as longitudinal beam reinforcement. Plain bars with 4 mm diameter were used for both column and beam ties. The ties had 90° hooks and were spaced at 100 mm. No confinement was provided at beam column joints. Only in specimen 2, the longitudinal bars of the columns were kept continuous along two stories. Other specimens had column longitudinal reinforcement lap spliced at story levels with a lap-length of 150 mm. Frame reinforcement was otherwise the same for all specimens.

The connections between the frame and infill walls were established with dowel reinforcements that were fixed with epoxy in to the frame. Deformed bars of 10 mm diameter were used as dowel reinforcement. The ratio of infill reinforcement was approximately the same as that for dowels. Dowel reinforcement details are shown in Fig. 2. The dowels were extended 300 mm into the infill wall at the foundation beam and 250 mm in to the other members on other sides. Specimen 4 had a different dowel arrangement, consisting of only vertical bars. Two continuous deformed rebars having the same diameter and properties as other vertical dowels were fixed into the infill using epoxy. The vertical dowels were staggered, and were extended 250 mm and 350 mm into the infill wall.

First, a set of holes were drilled on the inner faces of frame members. Then, the holes were cleaned and the dow-

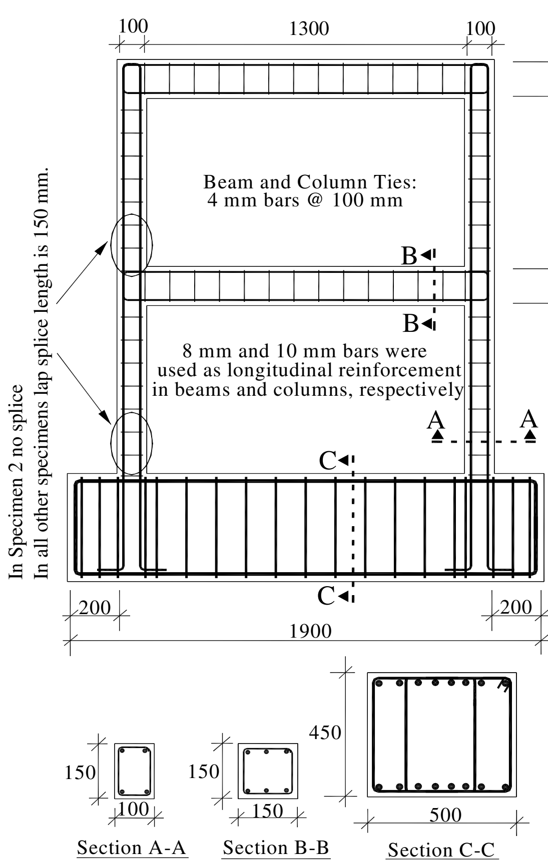


Fig. 1. Dimensions and reinforcement details of bare frame.

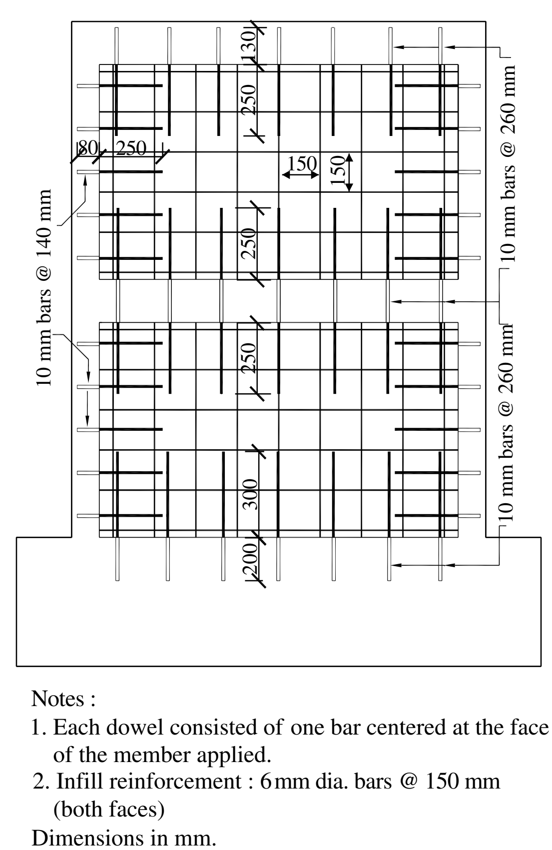


Fig. 2. Infill reinforcements and dowels details of specimen 2, 3, 5 and 6.

els were inserted into epoxy injected holes. Each dowel consisted of one bar centered at the face of the member. Mesh reinforcement was used for infill wall consisting of 6 mm plain bars. Bar spacing in horizontal and vertical directions were 150 mm. The reinforcement was placed on both faces of the infill wall. The ratio of infill wall reinforcement were $\rho_v = \rho_h = 0.009$ in both vertical and horizontal directions.

Specimens 1 and 2 were reference specimens. Specimen 1 was a bare frame and the other specimens were infilled. There was no strengthening of the lap spliced region for

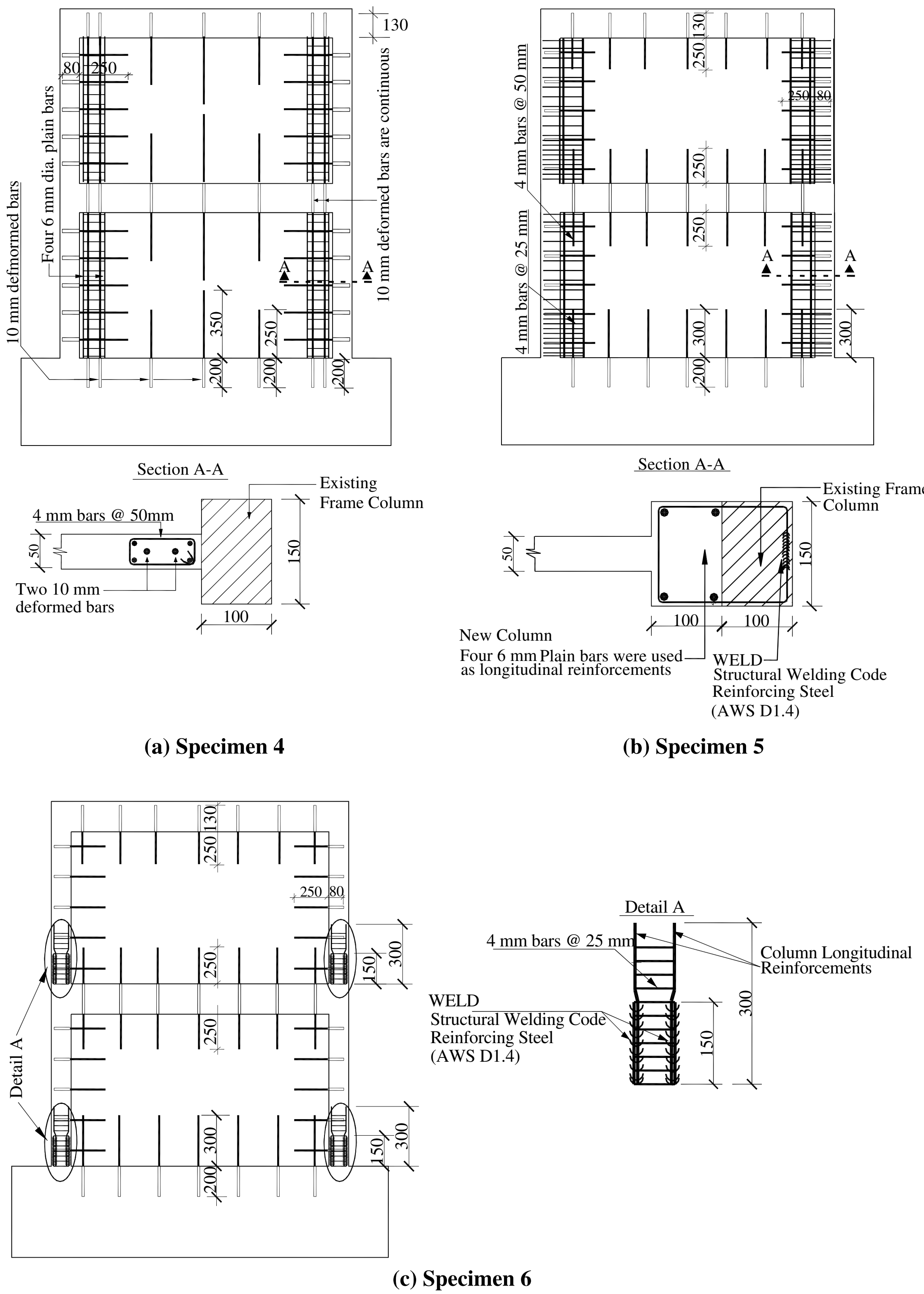


Fig. 3. Local strengthening technique details of specimen 4, 5 and 6 (Dimensions in mm.).

specimen 3. Lap spliced regions of specimens 4, 5, and 6 were strengthened. Details of strengthening that was applied to infilled specimens are shown in Fig. 3(a)–(c). In specimen 4, boundary element of 130×50 mm dimensions were used at both ends, consisting of four 6 mm diameter plain bars. The boundary elements were confined with 4 mm diameter closed ties, spaced at 50 mm (Fig. 3(a)). Two new columns were built inside the frame of specimen 5. The new columns had the same dimensions as the existing columns (100×150 mm). Four longitudinal reinforcements with 6 mm plain bars were used in these new columns. Frame columns were confined together with the new columns by using 4 mm diameter plain ties. The spacing of ties was 25 mm and 50 mm at the lap splice region and outside of the lap splice region, respectively. The details of strengthening for specimen 5 are shown in Fig. 3(b). Cover concrete for the frame columns of specimen 6 were removed at lap splice regions and longitudinal reinforcement were welded together. These regions were confined with 4 mm diameter plain ties spaced at 25 mm along the bottom 300 mm segment of column. The details of the strengthening procedure for specimen 6 are shown in Fig. 3(c).

Table 2
Properties of reinforcing bars

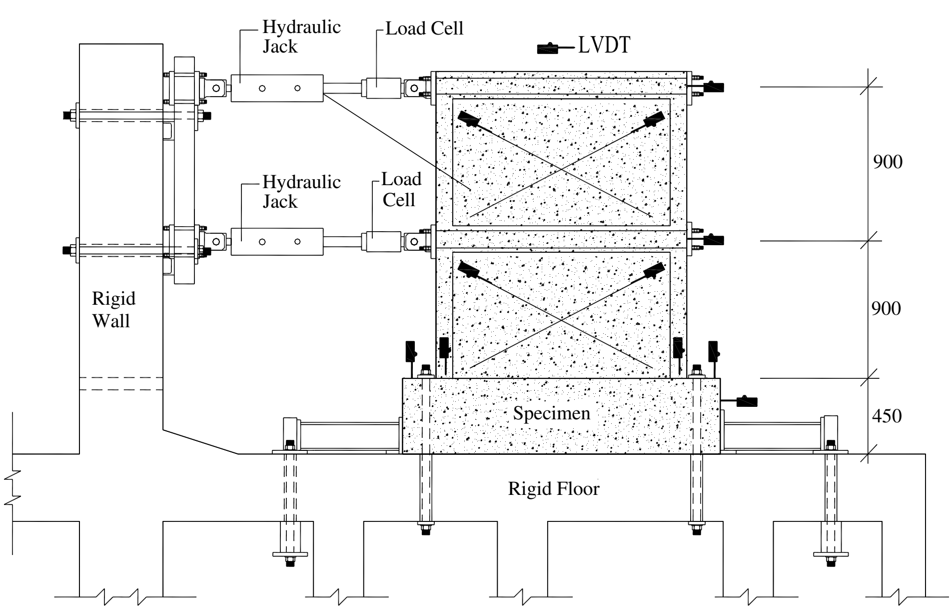
Bar diameter (mm)	f_{sy} (MPa)	f_{su} (MPa)	Type
4	326	708	Plain
6	427	489	Plain
8	592	964	Deformed
10	475	689	Deformed

2.1.1. Materials

The frames of specimens were cast from low strength concrete to represent the strength of concrete in existing buildings. The concrete strength was approximately 15 MPa. However, the concrete strength for infill walls was approximately 30 MPa on the day of testing. The properties of reinforcement used in this study are listed in Table 2.

2.2. Test setup and instrumentation

Details of test setup, loading system, and instrumentation are shown in Fig. 4. The specimens were fixed on stiff RC base beams and bolted to laboratory strong floor. A steel stability frame was constructed around the test specimen to prevent out-of-plane deformations. Specimens were tested under reversed-cyclic lateral loading applied to both floors. At every loading stage 1/3 of the base shear was applied to the first story and 2/3 to the second story. The lateral forces were applied by a hydraulic pump that controlled two hydraulic jacks. The forces were measured by the two load cells (with a capacity of 400 kN in compression and 200 kN in tension). The load level remained in the elastic range for the first few cycles and increased beyond yielding subsequently. During the test, the first and second story displacements and lateral loads were measured. Linear variable differential transformers (LVDTs) were used for displacement measurements. Average shear deformations of walls were measured by diagonally placed displacement transducers. Top displacements and lateral loads were monitored during testing. New initiated cracks and the propagation of cracks were marked on specimens and the mechanism of failure was observed during testing.



Dimensions in mm.

Fig. 4. Details of test setup, loading system, and instrumentation.

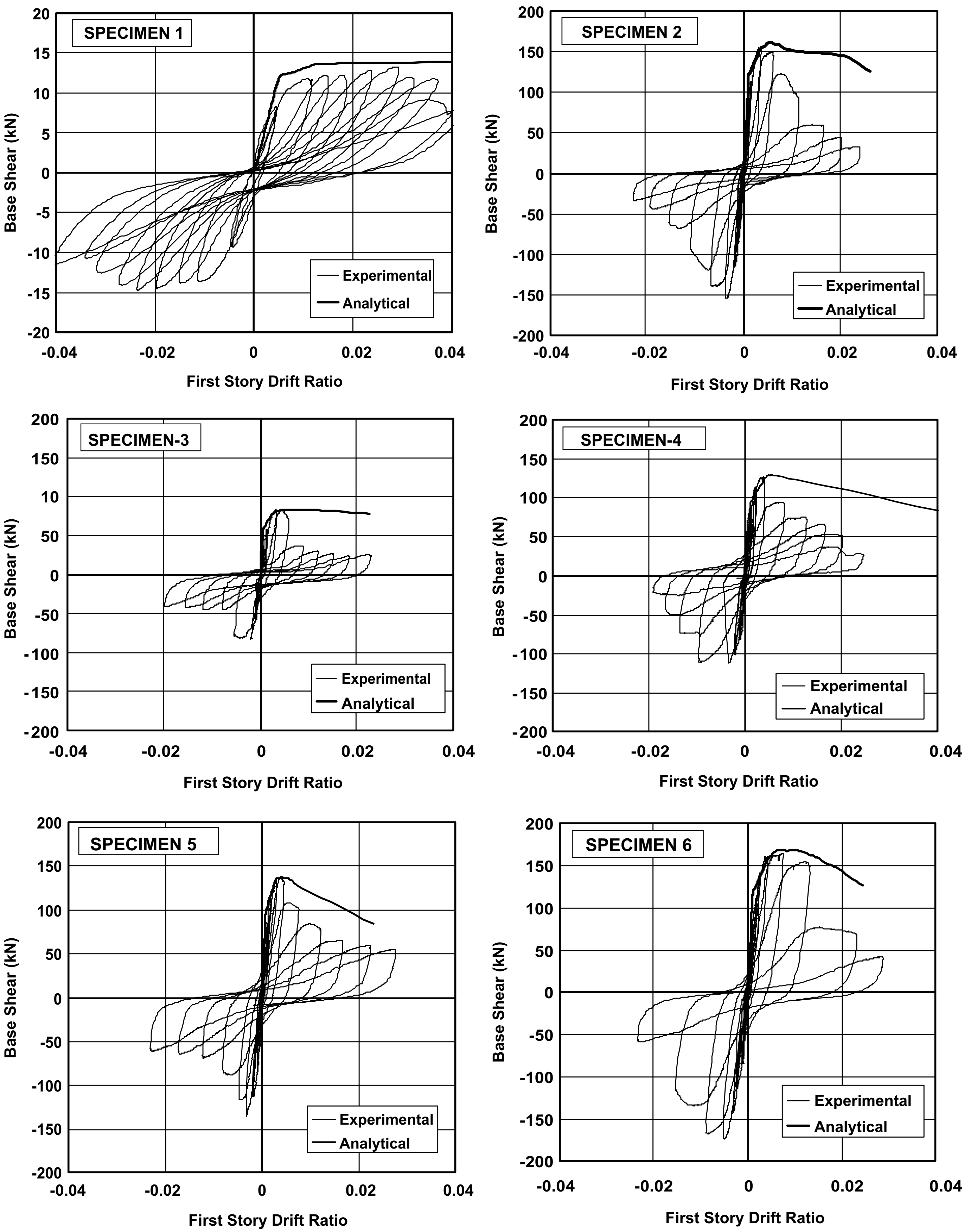


Fig. 5. Base shear-first story drift ratio hysteretic curves of specimens.

3. Experimental results

3.1. General behavior of test specimens

Base shear versus first story drift hysteresis curves for the test specimens are shown in Fig. 5. As can be seen from these curves, adding infill walls into the bare frame decreased the story drift, and increased strength, stiffness and energy dissipation capacity significantly. Specimen 3 developed the lowest strength among the infilled specimens. This specimen lost 50% of its lateral load carrying capacity immediately after reaching its ultimate load capacity. Local strengthening of lap splice regions of other specimens improved strength and stiffness significantly as compared with specimen 3. Specimens that had local strengthening at lap splices regions lost lateral load carrying capacities gradually when compared with specimen 3. The highest improvement was obtained in specimen 6. Test results are summarized in Table 3. All infilled specimens reached their ultimate lateral load capacities at 0.3% first story drift ratio, approximately.

No separation and slippage of dowels were observed at the connection between frames and infill walls. Flexural cracks initiated and concentrated at column lap splice regions. Fig. 6 shows specimens after testing. Bare frame specimen 1 showed flexible column mechanism, and failed due to the slippage of first story column longitudinal reinforcement at foundation beam level. Infilled specimen 2 with continuous column reinforcement failed, because a large number of diagonal shear cracks occurred in the first story infill wall and concrete in this region crushed. Infilled specimen 3 with no local strengthening of column lap splices failed due to the slippage of frame column longitudinal reinforcement. Infill wall slid along a large horizontal crack that formed at the tip of anchor dowels. The main shear crack in the first story wall joined the horizontal sliding shear crack that formed at the tip of dowels. Among the retrofitted specimens, only specimen 4 exhibited the slippage of longitudinal column reinforcement. specimen 4 eventually failed by sliding along the shear keys that formed in the first story. Specimens 5 and 6 with seismic retrofitting failed due to sliding along the horizontal crack at the tip of anchor dowels. This was attributed to the shifting of the critical section to the location where the dowels were terminated. The dowels were terminated 300 mm above the foundation beam.

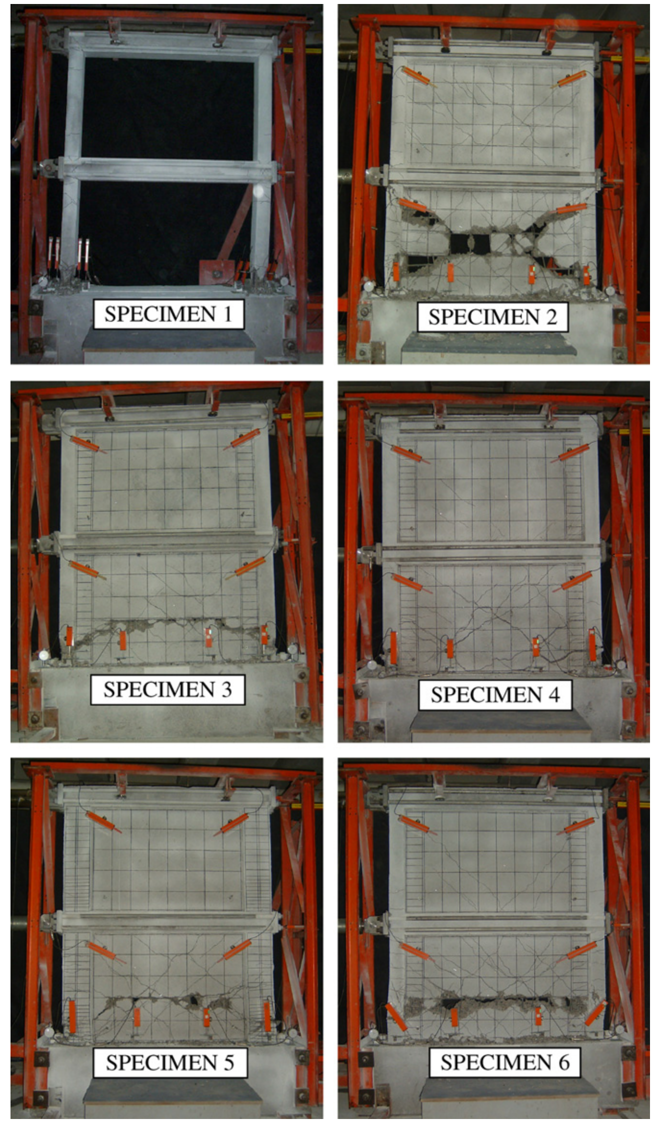


Fig. 6. Test specimens after failure.

3.2. Strength and stiffness

Seismic retrofitting non-ductile frames resulted in a significant increase in lateral load resistance. As can be seen from Table 3, maximum lateral load carrying capacities of infilled specimens were between 5 and 12 times greater those for bare frames. Lateral strengths of specimens with

Table 3
Summary of test results

Specimens	Maximum lateral load (kN)				Drift ratio at maximum load level (%)		Failure mechanism
	Forward	[a]	Backward	[b]	Forward	Backward	
1	13.94	1.00	−15.46	1.00	2.01	−2.01	Column mechanism
2	156.00	11.19	−155.00	10.03	0.27	−0.30	Shear
3	82.61	5.93	−82.54	5.34	0.33	−0.34	Flexure-sliding shear
4	127.02	9.12	−111.97	7.24	0.29	−0.21	Flexure-shear
5	136.53	9.79	−136.18	8.81	0.31	−0.25	Flexure-sliding shear
6	165.25	11.85	−174.25	11.27	0.32	−0.33	Flexure-sliding shear

[a] Ratio of maximum lateral load of infilled frame to bare frame in forward cycles.
[b] Ratio of maximum lateral load of infilled frame to bare frame in backward cycles.

retrofitted spliced regions were 10 times greater than those of bare frames. The largest strength increase was obtained in specimen 6 which showed twice the capacity of specimen 3. Specimens 4 and 5 had 1.6 times greater strength than specimen 3.

Strength envelopes were plotted by connecting the peak points of lateral load displacement hysteretic curves in Fig. 7. The strength envelopes are used for determining general behaviors and strengths of specimens. The addition of infill walls increased strength and stiffness of nonductile RC frames, and decreased drift ratios, as expected. Retrofitting lap spliced regions improved the strength and behavior of specimens significantly. The stiffness values are given in Table 4. The initial stiffness of specimens was calculated by using the slope of the linear part of the first load excursion. The failure stiffness was computed as the slope the line connecting 15% strength decay points in two directions. In general, the infilled specimens had 78 and 38 times larger initial and failure stiffnesses when compared with bare frames, respectively. Specimen 3 had the smallest stiffness among infilled specimens. Retrofitting lap splice regions significantly improved the initial and failure stiffnesses of specimens. Specimen 5 with two new columns on both sides of the infill wall had the largest stiffness among the specimens.

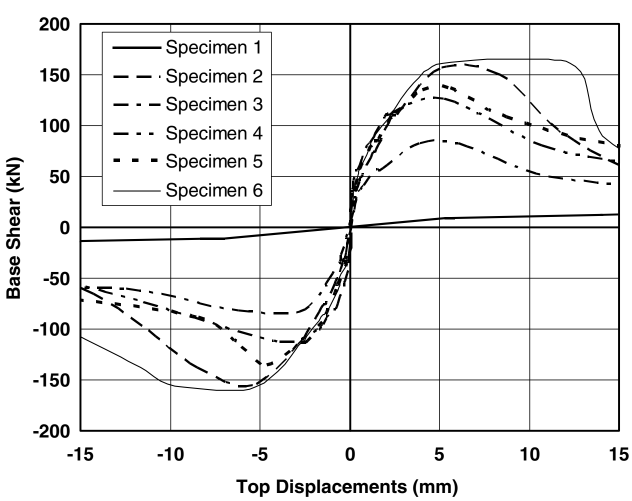


Fig. 7. Base shear-top story displacement envelope curves.

Table 4
Stiffnesses of test specimens

Specimens	Stiffness (kN/mm)		Ratio[a]
	Initial	At failure	
1	2.45	0.47	1.00
2	217.38	17.00	88.73
3	70.94	11.55	28.96
4	129.11	13.04	52.70
5	337.99	31.02	137.96
6	203.97	16.47	83.25

[a] Ratio of initial stiffness of infilled frame to bare frame.

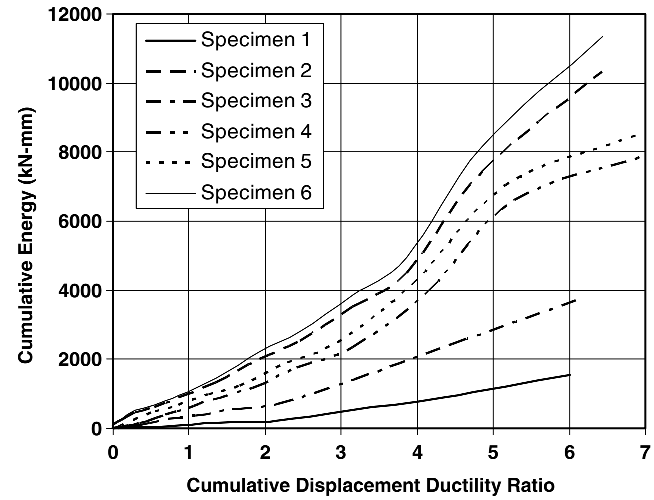


Fig. 8. Energy dissipation characteristics of specimens.

3.3. Energy dissipation capacities of test specimens

The energy dissipation was determined by calculating the areas inside the hysteretic load–displacement loops for each cycle, disregarding displacements beyond which lateral strength reduction reached 15%. Fig. 8 depicts the variation of cumulative dissipated energy as a function of cumulative displacement ductility ratio. Displacements at ultimate load were considered as yield displacements for the calculation of ductility values. Specimen 3 dissipated the smallest amount of energy among the infilled specimens. The energy dissipation capacity of infilled specimens was significantly improved by the introduction of strengthening of column lap splice regions. Specimen 6 dissipated the largest amount of energy among those with lap splice retrofits. It also dissipated a bit more energy than specimen 2. Specimen 6 dissipated 3 times as much energy as specimen 3, and as an average specimens 4 and 5 dissipated 2 times as much energy as specimen 3.

4. Analytical study

The load displacement behavior of test specimens was evaluated by using nonlinear push over software IDARC-2D [21]. Pushover analyses simulated the nonlinear lateral load–displacement relationship of the test specimens analytically. Analytical model for specimen 3 is given in Fig. 9 as an example. There was no special element in the software for modeling column lap splices. For this reason, the member capacities were calculated considering maximum forces developed in members using the actual anchorage length of column bars. The maximum stress in spliced bars was calculated using nominal bond stress, as suggested by an earlier researcher [22]. Beam of the frame was modeled as two pieces and was connected to middle axis of infill wall. Parts of the beam that was connected to infill wall were modeled as rigid elements. The RC infills were simulated as shear walls. The three parameter Park model was

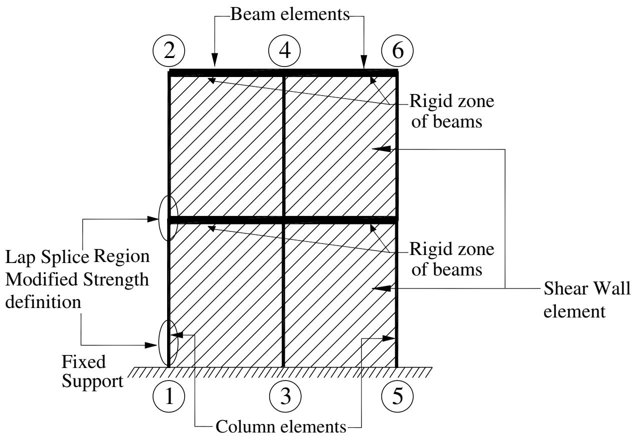


Fig. 9. IDARC-2D model of specimen 3.

used as a hysteretic model. Incorporated stiffness degradation and strength degradation, parameters were taken into account in the hysteretic model. It was assumed that there was low level strength deterioration, and high level stiffness degradation. The lateral loads were increased step by step until the system reached failure. The experimental hysteretic base shear load versus first story drift curves and analytical pushover curves for the specimens are shown in Fig. 5. The analytical models adequately simulated the behavior of infilled test specimens until the ultimate load was attained. The analytical response envelopes showed that the initial stiffnesses of infilled specimens were evaluated slightly greater than those observed experimentally. However, the decrease in stiffness due to cyclic loading could not be well simulated.

Investigation of the analytical load displacement curves showed that they successfully simulated the experimental load carrying capacities. But analytically calculated initial stiffnesses were slightly greater than the experimental results. In addition there were differences between the part of analytical and experimental load displacement curves at which after the ultimate load was reached and load was started to drop. In our opinion one of the main reasons for the difference between the analytical and experimental initial stiffnesses is the difference in the method of load application. While cyclic loading was applied during experiments, analytically the load was increased monotonously up to failure. This difference in application of loading affected the initial stiffnesses of analytical and experimental results. IDARC-2D software does not include elements that were crushed under compression and crack under tension for modeling concrete members. Due to this reason, the loss of load carrying capacity after ultimate load can not be simulated well with the analytical model. The loss of load carrying capacity after ultimate load was modeled by applying empirical coefficients to the hysteretic model of IDARC-2D software.

The values obtained from pushover analysis and the ratios of experimental ultimate load and initial stiffness values to analytical ones are summarized in Table 5. The lateral load capacities of infilled specimens were simulated with great success. The ratios of ultimate loads measured in infilled specimens to analytical values varied between 0.97 and 1.03. Initial stiffnesses calculated from pushover analysis were greater than the experimental values for all specimens. The ratio of experimentally obtained initial stiffnesses to those computed analytically varied between 0.83 and 0.92 for infilled specimens.

5. Summary and conclusions

This paper report on an experimental study about strengthening of nonductile RC frames by introducing infill walls. Nonductile RC frames with deficient longitudinal column reinforcement lap splices were strengthened with three different splice strengthening techniques, and tested under cyclic lateral loads. These strengthening techniques included; (a) adding continuous longitudinal reinforcements along two stories in boundary elements of infills, (b) constructing new columns on both sides of infill walls, (c) welding column lap splices. A common precaution that was taken in all splice strengthening techniques was extra confinement in these regions externally with closely spaced ties. The results are summarized below:

- Strength, stiffness and energy dissipation capacity of infilled specimens were significantly higher than those for bare frames.
- Frame specimens with deficient column lap splices showed improved strength and stiffness when retrofitting with infill walls. However, in the absence of splice retrofits, the specimens failed prematurely because of local

Table 5
Comparison of analytical and experimental results

Specimens	Ultimate load capacity (kN)			Initial stiffness (kN/mm)		
	Experimental	Analytical	Ratio[a]	Experimental	Analytical	Ratio[a]
1	15.46	13.57	1.14	2.45	2.44	1.00
2	156.00	161.43	0.97	217.38	239.50	0.91
3	82.62	83.28	0.99	70.94	82.67	0.86
4	127.02	128.94	0.99	129.11	155.45	0.83
5	136.53	137.60	0.99	337.99	368.25	0.92
6	174.25	168.60	1.03	203.97	232.23	0.88

[a] Ratio of experimental results to that of analytical values.

weaknesses of deficient column lap splices. Strength and stiffness of specimens with spliced column reinforcement were significantly lower than those with continuous column reinforcements.

- Infilled specimens with column lap splice retrofits showed significant improvements in strength, stiffness and energy dissipation capacity relative to infilled specimens with no splice retrofits.
- The strengthening technique in which longitudinal column reinforcement was welded and external confinement was provided at lap splice region by extra stirrups was the simplest, quickest and practically applied method among the others. In addition, the specimen with welded column lap splices showed the most successful behavior among the specimen with splice retrofits. Specimen 6 showed the largest story drift ratio with respect to other specimens without loosing its strength. For this reason Specimen 6 dissipated the largest amount of energy.
- Analytical studies were performed to understand the behavior of infilled specimens. IDARC-2D program successfully simulated ultimate strengths, although its push over analysis resulted in slightly higher initial stiffnesses for infilled specimens.

References

[1] Higashi Y, Kokusho S. The strengthening method of existing reinforced concrete buildings, US–Japan Cooperative Research Program in Earthquake Engineering, Honolulu, Hawaii, 1975.

[2] Higashi Y, Endo T, Shimuzu Y. Effects on behavior of reinforced concrete frames by adding shear walls, in: Proc of the Third Seminar on Repair and Retrofit of Structures, Ann Arbor, MI, 1982.

[3] Higashi Y, Endo T, Shimuzu Y. Experimental studies on retrofitting of reinforced concrete building frames, in: Proc of the Eight World Conference on Earthquake Engineering, San Francisco, CA, 1984, vol. 1, pp. 477–84.

[4] Klingner RE, Bertero VV. Infilled frames in earthquake-resistant construction, Report No. UCB/EERC-76/32, Earthquake Engineering Research Center, University of California, Berkeley, 1976.

[5] Klingner RE, Bertero VV. Earthquake resistance of infilled frames. ASCE J Struct Div 1978;104(ST6):973–87.

[6] Kahn LF, Hanson RD. Infilled walls for earthquake strengthening. ASCE J Struct Div 1979;105(ST2):283–96.

[7] Axley JW, Bertero VV. Infill panels: Their influence on seismic response of buildings, Report No. UCB/EERC-79/28, Earthquake Engineering Research Center, University of California, Berkeley, 1979.

[8] Axley JW. Modeling the stiffness contribution of infilled panels to framed structures by a constraint approach, in: Proc of the 7th WCEE, vol. 4, Istanbul, Turkey, 1980, p. 249–51.

[9] Sugano S. Aseismic strengthening of existing reinforced concrete buildings, in: Proc of the First Seminar on Repair and Retrofit of Structures, US/Japan Cooperative Earthquake Engineering Research Program, 1980.

[10] Aoyama H, Kato D, Katsumata H, Hosokawa Y. Strength and behavior of postcast shear walls for strengthening of existing R/C buildings, in: Proc of Eight World Conference on Earthquake Engineering, San Francisco CA, 1984, vol. 1, p. 485–92.

[11] Liauw TC, Kwan KH. Unified plastic analysis for infilled frames. ASCE, J Struct Div 1985;111(7):1427–49.

[12] Jirsa JO, Kreger ME. Recent research on repair and strengthening of reinforced concrete structures. In: Kircher CA, Chopra AK, editors. Seismic Engineering, ASCE, 1989, p. 679–88.

[13] Valluvan R, Kreger ME, Jirsa JO. Strengthening of column splices for seismic retrofit of nonductile reinforced concrete frames. ACI Struct J 1993;90(4):432–40.

[14] Miller JT, Reaveley L.D. Historic hotel Utah remodel and seismic upgrade. In: Sabnis GM, Shroff AC, Khan, LF, editors. Seismic Rehabilitation of Concrete Structures, SP-160, American Concrete Institute, Farmington Hills, MI, 1996, p. 115–30.

[15] Gregorian ZB, Gregorian GB. Seismic upgrading of hospital building No. 6 and 7, VA Medical Center, Bedford, Mass. In: Sabnis GM, Shroff AC, Khan, LF, editors. Seismic Rehabilitation of Concrete Structures, SP-160, American Concrete Institute, Farmington Hills, MI, 1996, p. 131–47.

[16] Ersoy U, Üzsoy S. The Behavior and Strength of Infilled Frame, TUBITAK MAG-205 Report, Ankara, Turkey, 1971, p. 95 [in Turkish].

[17] Altın S, Ersoy U, Tankut T. Hysteretic response of reinforced concrete infilled frames. ASCE J Struct Eng 1992;118(8):2133–50.

[18] Türk AM. Rehabilitation of reinforced concrete infill walls, Phd thesis, Bogazici University, Istanbul, Turkey, 1998, p. 213.

[19] Canbay E, Ersoy U, Özcebe G. Contribution of RC Infills to the seismic behavior of structural system. ACI Struct J 2003;100(5):637–43.

[20] Sonuvar MO, Özcebe G, Ersoy U. Rehabilitation of reinforced concrete frames with reinforced concrete infills. ACI Struct J 2004;101(4):494–500.

[21] Valles RE, Reinhorn A.M, Kunnath SK, Li C, Madan A. IDARC 2D Version 4.0: A Program for the Inelastic Damage Analysis of Buildings, Technical Report NCEER-96-0010, State University of New York at Buffalo, (1996).

[22] Mylrea TD. Bond and Anchorage. ACI J Proc 1948;44(3):521–52.