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Effect of introducing RC infill on seismic performance of damaged RC frames

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Abstract. The main objective of this study was to investigate the seismic behavior of damaged reinforced concrete frames rehabilitated by introducing cast in place reinforced concrete infills. Four bare and five infilled frames were constructed and tested. Each specimen consisted of two (twin) 1/3-scale, one-bay and two-story reinforced concrete frames. Test specimens were tested under reversed-cyclic lateral loading until considerable damage occurred. RC infills were then introduced to the damaged specimens. One bare specimen was infilled without being subjected to any damage. All infilled frames were then tested under reversed-cyclic lateral loading until failure. While some of the test frames were detailed properly according to the current Turkish seismic code, others were built with the common deficiencies observed in existing residential buildings. The variables investigated were the effects of the damage level and deficiencies in the bare frame on the seismic behavior of the infilled frame. The deficiencies in the frame were; low concrete strength, inadequate confinement at member ends, 90 degree hooks in column and beam ties and inadequate length of lapped splices in column longitudinal bars made above the floor levels. Test results revealed that both the lateral strength and lateral stiffness increased significantly with the introduction of reinforced concrete infills even when the frame had the deficiencies mentioned above. The deficiency which affected the behavior of infilled frames most adversely was the presence of lap splices in column longitudinal reinforcement.

Keywords: reinforced concrete; infill; building frames; cyclic test; strength; stiffness; rehabilitation.

1. Introduction

In the last decade, six major earthquakes caused significant casualties and extensive structural damage in Turkey. In only the Marmara (August-17-1999) and Duzce (November-12-1999) earthquakes, 15 000 people were killed and more than 20 000 were injured. Observations made after these earthquakes indicated that the performances of low and medium rise RC residential and office buildings were generally very poor due to their flexible frame systems with limited ductility.

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The surveys revealed that these buildings had some common deficiencies. These deficiencies were; low concrete strength, inadequate lateral stiffness, inadequate ductility (ends of members and beam-column joints were not properly confined) and inadequate length of lapped splices in column longitudinal bars made above the floor levels. After these earthquakes, rehabilitation projects were launched to repair and strengthen the damaged buildings that had light and medium damage. These rehabilitation projects were financed by the Turkish Government.

Many different techniques have been tested and applied in the last 30 years for rehabilitation of existing reinforced concrete frame type of structures (Sugano 1996, Ersoy 1996, Jirsa and Kreger 1989, Moehle 2000). These methods can be utilized at the structural level or member level, such as jacketing of columns and beams, the addition of wing walls, addition of infill walls, addition of single or multiple precast reinforced concrete panel walls, addition of steel bracings, external steel framing and post tensioning (Higashi and Kokusho 1975, Kahn and Hanson 1979, Makino *et al.* 1980, Yuzugullu 1980, Bertero and Brokken 1983, Liauw and Kwan 1985, Jirsa and Kreger 1989, Badoux and Jirsa 1990, Miranda and Bertero 1990, Bush *et al.* 1990, Altin *et al.* 1992, Pincheira and Jirsa 1995, Phan and Lew 1996, Frosch *et al.* 1996, Turk 1998, Inukai and Kaminosono 2000, Lee and Woo 2002). In these studies, the loading histories were either monotonic or cyclic (Endo *et al.* 1980, Higashi *et al.* 1980, Sugano and Fujimara 1980, Ohki and Bessho 1980, Hayashi *et al.* 1980). Many different types of connections of the infill wall to the surrounding frame have been studied, such as the performance of different types of shear keys, dowels, and chemical anchors and wall reinforcement configurations (Sugano 1980, Aoyama *et al.* 1984, Jirsa 1988, Altin *et al.* 1992, Frosch 1996). Although many of the previous tests were performed on one-bay, one-story frames, there are some tests on one-bay, two-story or three-story frames (Higashi, Endo, and Shimizu 1982, 1984, Altin *et al.* 1992, Turk 1998). Nearly all of the previous tests were performed on ductile reinforced concrete frames.

Tests performed on non-ductile RC frames showed that the strengthened infilled frames show premature failures due to the failure of splices in the existing column made above the floor level (Valluvan, Kreger, and Jirsa 1993, Turk 1998, Sonuvar 2004).

As a conclusion, the simplest and the most effective way of improving the overall behavior of reinforced concrete buildings, i.e., placing an adequate number of structural walls into the strategic bays, eliminates the unsatisfactory seismic behavior due to the poor structural system. Such walls not only increase the lateral stiffness significantly but also relieve the existing non-ductile frames from carrying large lateral forces.

Results of previous tests and studies on Turkish buildings revealed that repair and strengthening of individual structural members would not be feasible, because almost all the columns and the beams of the reinforced concrete buildings would have to be rehabilitated. Moreover, inadequate lateral stiffness was a major problem. It was found out that seismic rehabilitation by introducing reinforced concrete infills to some selected bays would be a practical and economical solution for the buildings under question. The infills would also increase the lateral stiffness of such damaged structures. Analyses made on some typical structures revealed that the infilled frames would act as structural walls and almost the total lateral force would be resisted by these walls. Consequently it was concluded that the frame members need not to be repaired or strengthened.

In Turkey, hundreds of damaged low and medium rise buildings were rehabilitated by introducing in-situ reinforced concrete infills during the last decade. Extensive experimental research has been carried out at the Middle East Technical University (METU) to understand the seismic behavior of reinforced concrete infilled frames. The work reported in this paper is a part of this research project.

The main objective of this experimental study was to investigate the difference in behavior between infilled frames compared with damaged and undamaged frames. It was also intended to compare the behavior of infilled frames where the infill is introduced to properly designed and detailed frames with those where the frames had the common deficiencies encountered in practice.

2. Research significance

Rehabilitation of damaged reinforced concrete buildings by filling some selected bays of the frames with reinforced concrete infills has proved to be one of the most feasible techniques in the seismic rehabilitation of existing buildings. This method is preferred when there are too many members to be rehabilitated and/or when the lateral stiffness of the structure is not adequate.

In the past, tests were made to understand the behavior of reinforced concrete infilled frames in METU (Ersoy and Uzsoy 1971), (Altin *et al.* 1992). In most of these tests, the frame was designed and detailed according the current seismic codes, (TSC 1975). Moreover, in most of the tests the infills were introduced to undamaged frames. In the past ten years, however, rehabilitation by introducing reinforced concrete infills involved damaged frames which were not designed and detailed properly. These frames had the deficiencies mentioned in the previous section.

The experimental research reported here was carried out jointly with the Middle East Technical University (METU) and concentrates on the behavior of infilled frames in which the infill had the common deficiencies encountered in practice (Turk 1998), (Canbay 2003), (Sonuvar *et al.* 2004). With the exception of one specimen, these frames were also damaged prior to the application of the infill. The deficiencies in the test frames were; poor concrete quality, inadequate lateral stiffness, lack of confinement at the ends of beams and columns and inadequate lap length of column longitudinal bars. The beam and column ties had 90 degree hooks at the ends and were not anchored into the core concrete (Turk 1998). The data obtained from the tests reported in this paper are believed to be very useful in developing design rules and details for rehabilitating the damaged framed structures having the deficiencies mentioned. Only a limited amount of data is available in the literature.

3. Test program

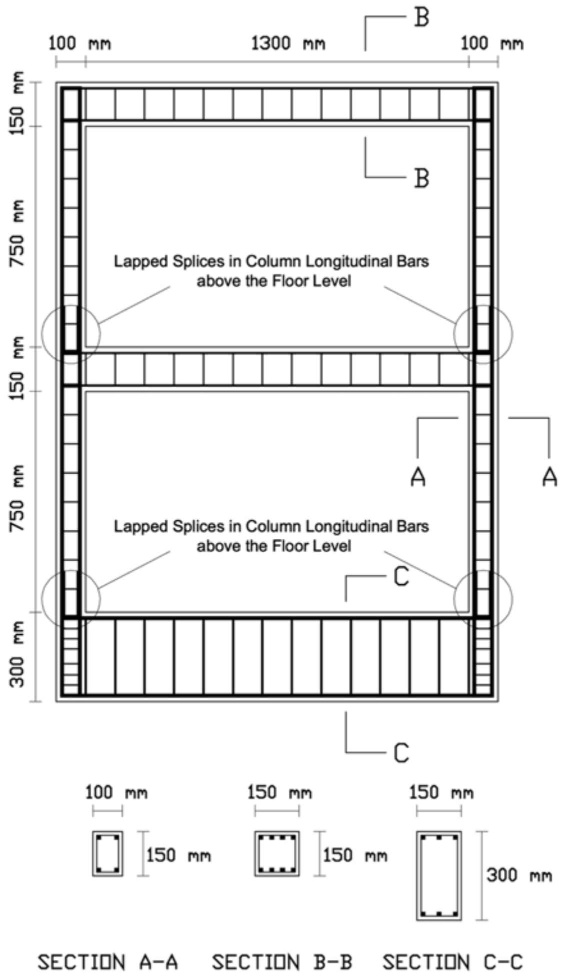
3.1 Test variables

The main variables investigated in this research can be summarized as follows:

1. The effect of the damage level of bare frames on the infilled frame behavior,
2. The effect of deficiencies/weaknesses in the bare frame members on the overall behavior of the infilled frame,
3. The relative behavior of infilled frames having damaged frames as compared to the ones having undamaged frames.

3.2 Tests specimens

Test specimens were one-third scale, one-bay, two story frames similar to the ones previously tested at METU (Altin *et al.* 1992). All frames except one were tested under reversed-cyclic lateral

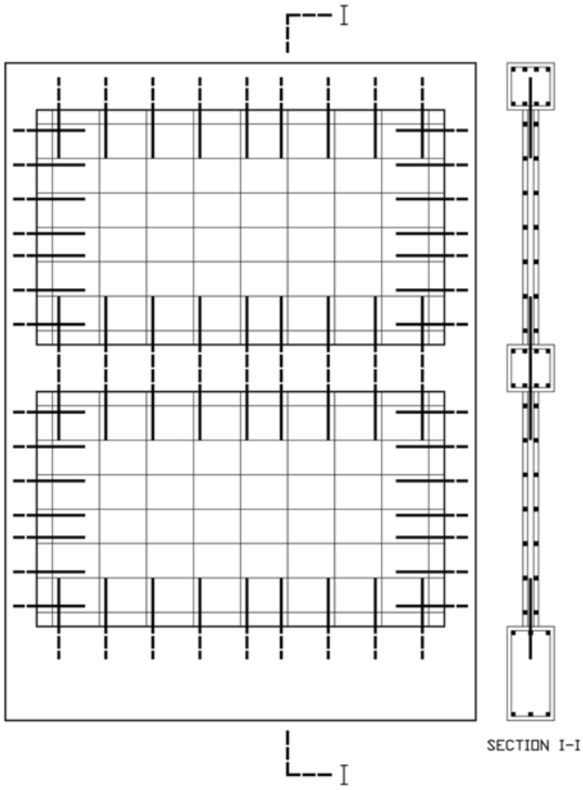


Notes:

1. Beam longitudinal bars: 8-8 mm bars (all specimens).
2. Column longitudinal bars: 8-10 mm bars (B1-C-H-00), 4-14 mm bars (B2-C-H-00), 4-8 mm bars (others).
3. Foundation beam longitudinal bars: 6-16 mm bars (all specimens).
4. Beam ties: 4 mm @ 40 mm at beam end zones, 4 mm @ 80 mm elsewhere (B1-C-H-00; B2-C-H-00).
5. Beam ties: 4 mm @ 100 mm throughout the entire span (B3-N-L-40, B4-N-L-15, B5-N-L-40).
6. Column ties: 6 mm @ 40 mm at column end zones, 6 mm @ 80 mm elsewhere (B1-C-H-00; B2-C-H-00).
7. Column ties: 6 mm @ 100 mm throughout the entire length (B3-N-L-40, B4-N-L-15, B5-N-L-40).
8. Foundation beam ties: 8 mm @ 100 mm
9. Column lap splice lengths: 15 bar diameters, i.e. 120 mm (B4-N-L-15),
   40 bar diameters, i.e. 320 mm (B3-N-L-40 and B5-N-L-40).

Fig. 1 Dimensions and reinforcement details of specimens

loading until considerable damage occurred. RC infills were introduced to the damaged specimens and these infilled frames were then tested under reversed-cyclic lateral loading until failure. No repair or strengthening was applied to the damaged frame members and joints. Four bare and five infilled frames were tested.



Notes:

1. Web reinforcement: 2-6 mm bars at 150 mm spacing (on both faces)
2. Dowel bar spacing for beams and columns: 8 mm bars at 120 mm spacing
3. Foundation beam dowel bars: 8 mm bars at 120 mm (R1-C-H-00, R2-C-H-00, R3-N-L-40)
   12 mm bars at 120 mm (R4-N-L-15, S5-N-L-40)
4. Embedment length of column dowel bars: 80 mm
5. Embedment length of beam dowel bars: 120 mm – 150 mm

Fig. 2 Details of infill reinforcement and connecting dowels of specimens: R1-C-H-00, R2-C-H-00, R3-N-L-40, R4-N-L-15 and S5-N-L-40

The alphanumeric characters used in the naming of the test specimens (e.g., B1-C-H-00) have the following significance. The first letter designates the specimen type. Here, the letter "B" stands for a bare frame test that constitutes the template for the repair and/or strengthening intervention. The letters "R" and "S", on the other hand, stand for the type of intervention applied [R for repair and S for strengthening]. B-specimens were tested under reversed-cyclic lateral loading until considerable damage occurred in the frame members. The R-specimens were manufactured by introducing RC infills to the damaged B-specimens. In the case of S-specimen the RC infill was introduced into the undamaged bare frame. The first numeral in specimen title is the sequence [or pair] id-number, thus specimens B1-C-H-00 and R1-C-H-00 are the members of the same pair, and they were therefore companion specimens. The letter following the first dash sign indicates the code compliance in the design. Those specimens having "C" in this position were designed according to the current Turkish

Table 1 Properties of the test specimens

Specimen Id.	Specimen type	Column long. reinf. ratio (ρ)	f_c' frame (MPa)	f_c' Infill panel (MPa)	f_y Column long. bars (MPa)	f_y Beam long. bars (MPa)	f_y Infill reinf. (MPa)	Axial load on each column (kN)
B1-C-H-00	Bare Frame	8 – 10 mm (0.04)	22.4	-	520	478	-	71.5
R1-C-H-00	Infilled Frame	8 – 10 mm (0.04)	-	22.9	520	478	396	71.5
B2-C-H-00	Bare Frame	4 – 14 mm (0.04)	23.1	-	473	478	-	71.5
R2-C-H-00	Infilled Frame	4 – 14 mm (0.04)	-	21	473	478	396	71.5
B3-N-L-40	Bare Frame	4 – 8 mm (0.013)	11.7	-	478	478	-	28.5
R3-N-L-40	Infilled Frame	4 – 8 mm (0.013)	-	25.9	478	478	376	28.5
B4-N-L-15	Bare Frame	4 – 8 mm (0.013)	10.7	-	478	478	-	28.5
R4-N-L-15	Infilled Frame	4 – 8 mm (0.013)	-	23.3	478	478	376	28.5
B5-N-L-40	Bare Frame	4 – 8 mm (0.013)	12.2	-	478	317	-	-
S5-N-L-40	Infilled Frame	4 – 8 mm (0.013)	-	19.5	478	317	376	49

[a]Dimensions of all test specimens are given in Figs. 1 and 2.
[b]In all specimens, beam longitudinal reinforcement were 8–8 mm ($\rho = 0.01$).
[c]In all specimens, longitudinal reinforcement of foundation beam were 6–16 mm bars with $f_y = 510$ MPa.
[d]The axial load level on all columns was about $N/N_o = 0.12$ (except for S5-N-L-40 where $N/N_o = 0.20$) N_o = Uniaxial load capacity of the column. ($N_o = 0.85 f_c A_c + f_y A_{st}$)

Table 2 Properties of the test specimens

Specimen Id.	Frame damage level	Code compliance	Length of column lap splices (l_o)	Spacing of connecting dowels at the foundation level
B1-C-H-00	Light	YES	Continuous	-
R1-C-H-00	-	YES	Continuous	8 mm bars @ 120 mm
B2-C-H-00	Heavy	YES	Continuous	-
R2-C-H-00	-	YES	Continuous	8 mm bars @ 120 mm
B3-N-L-40	Heavy	NO	40 d_b	-
R3-N-L-40	-	NO	40 d_b	8 mm bars @ 120 mm
B4-N-L-15	Heavy	NO	15 d_b	-
R4-N-L-15	-	NO	15 d_b	12 mm bars @ 120 mm
B5-N-L-40	No Damage	NO	40 d_b	-
S5-N-L-40	-	NO	40 d_b	12 mm bars @ 120 mm

[a]In all specimens axial loads were applied to the columns.
[b]Frames B1-C-H-00 and B2-C-H-00 were designed and detailed in accordance to the Turkish Seismic Design Code (TSC 75).
[c]In the frames (with the exception of B1-C-H-00 and B2-C-H-00) there were no ties at beam-column joints and tie hooks were 90^o.

Seismic Code (TSC 98) whereas those with “N” did not comply with the code requirements. The third letter in the specimen title is used for longitudinal steel percentage designation. The letter “H” here indicates high longitudinal steel percentage [i.e., 4%] while the letter “L” is for low longitudinal percentage [1.3%]. In specimen titles the last two digits are used to show the lap splice length in terms of bar diameter. Thus the specimen B3-N-L-40 had a lap length of 40 bar diameter. In the design of four specimens no lap splicing was used. The longitudinal reinforcement of these members was continuous. The last two digits in the title of these specimens were “00”.

The first two frames (B1-C-H-00 and B2-C-H-00) were duplicates of the specimens tested by Altin *et al.* (1992). They were designed and detailed in accordance with the 1975 Turkish Seismic Code (TSC 1975). In these two frames there were no lapped splices in the column longitudinal bars. The remaining three frames had some of the common deficiencies observed in existing residential buildings in Turkey. These included low concrete strength (11-12 MPa), inadequate confinement at the end of beams and columns, 90 degree hooks at the end of ties and inadequate lap splice length in column longitudinal bars made above the floor level (15 to 40 bar diameters). The ratio of column longitudinal reinforcement in specimens B1-C-H-00 and B2-C-H-00 was 0.04. In the rest of the test frames this ratio was 0.013.

The infills were introduced to the damaged frames except in Specimen S5-N-L-40. In this specimen the infill was introduced to the undamaged frame B5-N-L-40 in order to study the effect of frame damage on the behavior of the infilled frame. The details and properties of frame B5-N-L-40 were identical with those of B3-N-L-40 bare frame. Dimensions and reinforcement of the test frames are shown in Fig. 1.

The thickness of the infill was 50 mm in all infilled frames. The infills had orthogonal reinforcement on both faces (6 mm bars spaced at 150 mm). The infills were connected to the frame members by dowels placed in epoxy grouted holes in the frame members. The dowels and the infill reinforcement are shown in Fig. 2. It is important to mention here that in the previous METU tests made by Altin *et al.* (1992), frames with panel-frame connectors displayed superior behavior with higher strength and improved ductility compared to those with no connectors. This is why the connection type developed by Altin *et al.* (1992) is used to establish the frame-RC infill connection in the present study. Properties of test specimens (frames and infilled frames) are given in Tables 1 and 2.

3.3 Test setup, instrumentation and test procedure

All bare frames and infilled frames were tested under reversed cyclic lateral loading, simulating seismic action. One-third scale, one-bay, two-story twin specimens were cast and tested with a common foundation beam to benefit from the advantage of the symmetrical loading setup. Lateral load was applied to the specimens at the second story level. During the tests, a constant axial load was applied to the columns. The magnitude of the axial load for each test is given in Table 1. The test set-up consisted of a loading frame, loading equipment, instrumentation and the data acquisition system as shown in Fig. 3. Lateral loading was applied inside a closed loading frame and the twin test specimens were placed between the reaction beams in a horizontal position, resting on rollers. Lateral load was applied from the common foundation beam of the twin specimens. Axial load was applied to steel cross-beams by two prestressing cables stretching between the mid-spans of the second story beams. All tests were load-controlled and the loading program was established prior to the each test. Each half cycle started with zero loads and after reaching the pre-determined lateral

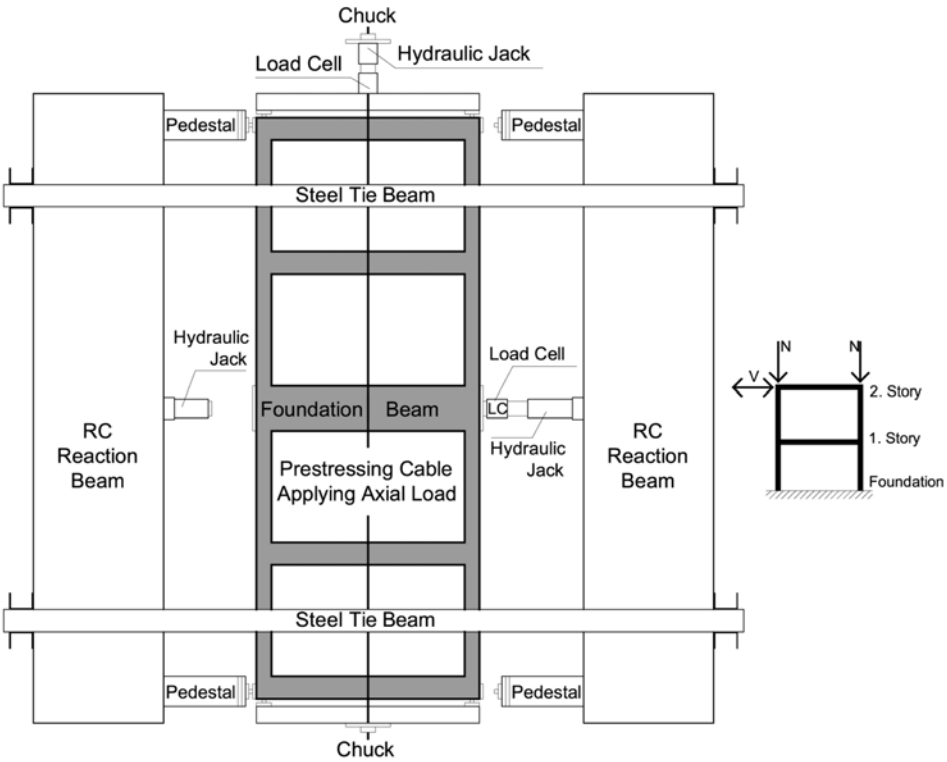


Fig. 3 Test setup

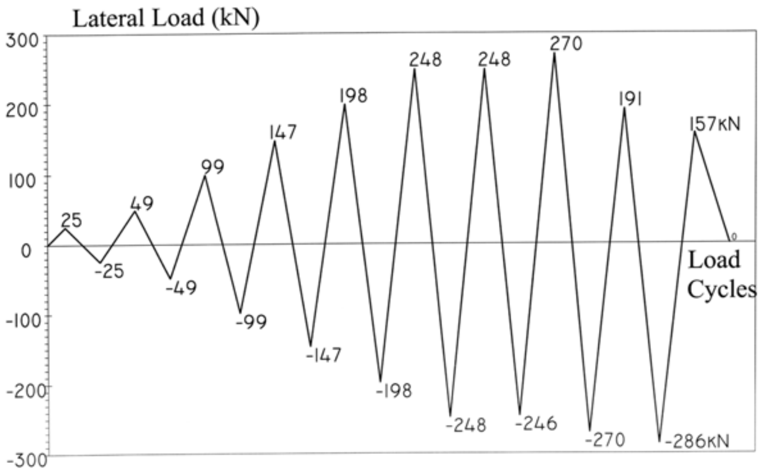


Fig. 4 Typical load history curve for an infilled specimen (R1-C-H-00)

load level, the specimen was unloaded. The direction of loading was reversed and the second half cycle was completed with similar loading and unloading stages. During the tests, necessary adjustments were made in the loading program depending on the behavior observed. The behavior of the specimen was monitored by observing the real-time load versus displacement curves drawn on the screen of the data acquisition system. A typical load history diagram for the infilled specimen is given in Fig. 4. The reversed cyclic loading program applied did not simulate any specific earthquake. For all bare and infilled specimens, the load level at first and second full cycles was kept low in order to remain in the elastic range. In the successive cycles, the load level was increased up to the yield level. Yield level was determined experimentally. During the tests, the lateral force versus lateral displacement relationship was monitored closely. The yield load and the corresponding displacement were determined when the lateral force-lateral displacement curve started to show nonlinearity under increasing lateral load during cycles. It is important to note that it was generally intended to apply the same load program to each specimen, especially in cycles before strength degradation initiates.

4. Observed behavior and discussion of test results

Lateral load versus first story drift ratio curves for the infilled frames are given in Fig. 5 to Fig. 9. In each figure, the results obtained from bare frame tests are also plotted.

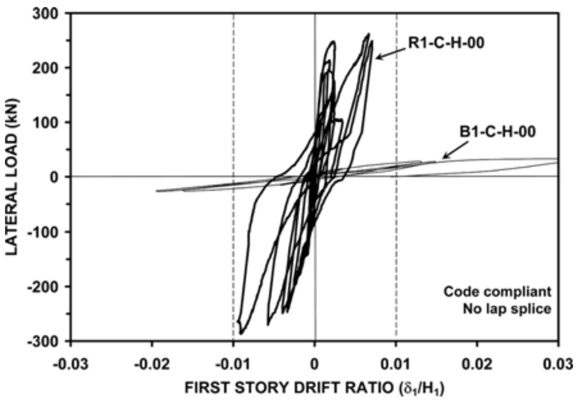


Fig. 5 Lateral load-first story drift ratio of specimens B1-C-H-00 and R1-C-H-00

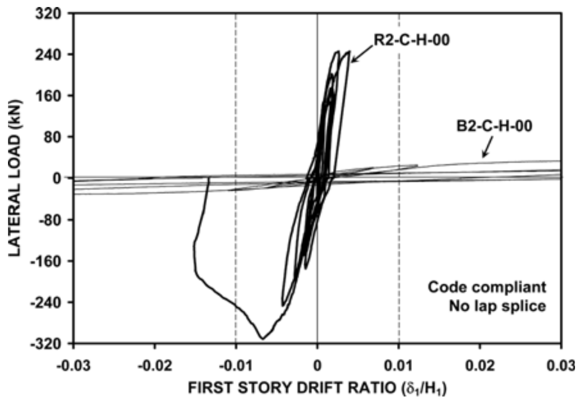


Fig. 6 Lateral load-first story drift ratio of specimens B2-C-H-00 and R2-C-H-00

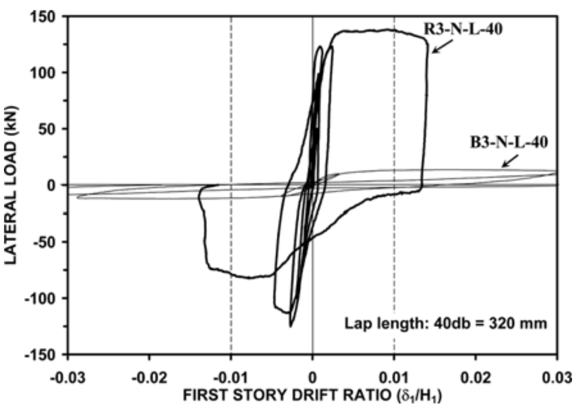


Fig. 7 Lateral load-first story drift ratio of specimens B3-N-L-40 and R3-N-L-40

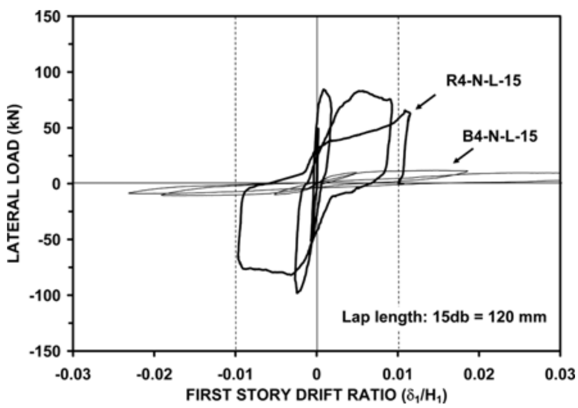


Fig. 8 Lateral load-first story drift ratio of specimens B4-N-L-15 and R4-N-L-15

The drift ratio was obtained by dividing the displacement of the first story with respect to the foundation by the story height. On each figure, the interstory drift ratio of one percent is also marked (Sozen 1987). As can be seen from these figures, significant increase in both strength and lateral stiffness are observed as compared to bare frames. The drift limit of 1 percent was exceeded after attaining the maximum strength in all cases.

In all bare frame tests first cracking was observed at the bottom of the first story columns. These were typical flexural cracks. At later stages, the column longitudinal reinforcement yielded at the level of these initial cracks, leading to a sway mechanism. In specimen B1-C-H-00 the test was

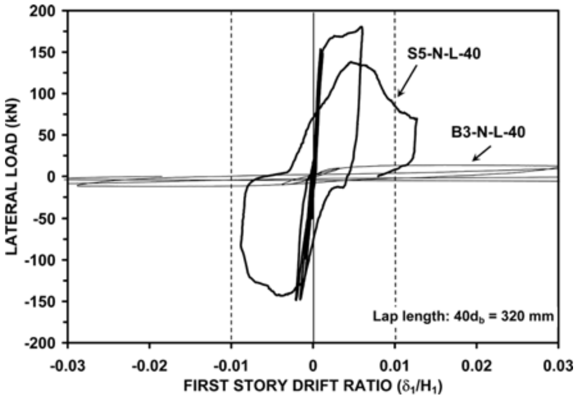


Fig. 9 Lateral load-first story drift ratio of specimens B3-N-L-40 and S5-N-L-40

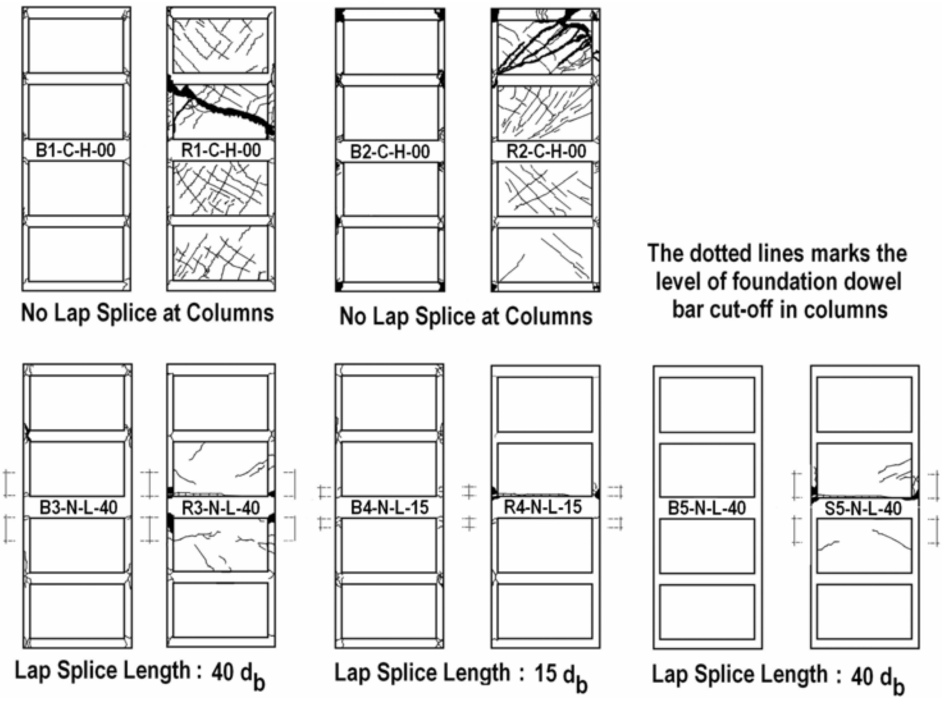


Fig. 10 Crack patterns of the bare and infilled frame specimens

terminated when the lateral drift was about 1.5 times the yield displacement. No cracks were observed in the first and second story beams during the test. Crack patterns for the bare and infilled frames are shown in Fig. 10.

The first crack observed on the infilled specimens was a flexural crack on the tension face of the first story boundary column at the base. This was the reopening of the existing crack which had formed during the frame test. In the first two infilled specimens (R1-C-H-00 and R2-C-H-00), which had frames designed and detailed in accordance with the 1975 Turkish Seismic Code (TSC 1975), failure was initiated by yielding of the infill reinforcement along a horizontal crack at the foundation level. The final failure took place along a diagonal crack in the first story infill. Prior to failure, numerous diagonal cracks were observed on the infills (Fig. 10). No significant differences were observed in the behavior and strength of these two infilled frames, although the frame of R1-C-H-00 had light damage and that of R2-C-H-00 had heavy damage. It was concluded that the level of frame damage was not a major issue if bare frames are designed and constructed in accordance with seismic code. In these two specimens column longitudinal bars were continuous from the foundation to the top of the frame, i.e., the column longitudinal reinforcement was not lapped.

It is worth mentioning here that the damage on the frame is specified as light if only hairline cracks develop at beam and column end zones with no indication of further distress in the reinforcement. Heavy damage, on the other hand, is the level of damage where crushing of the cover concrete was observed. In these specimens the longitudinal reinforcement generally yielded, however, there were no signs of core crushing or buckling of longitudinal steel.

The failure of the remaining three infilled frames was premature due to the existence of lapped splices in the column longitudinal bars above the floor levels. As would be recalled, the frames of these infilled frames had the deficiencies mentioned previously. Although the failure was premature, the strength and lateral stiffness of these infilled frames were still significantly higher than those of the bare frames.

The lateral stiffness is an important parameter in order to evaluate the seismic performance of a structure. Lateral stiffnesses of the bare and the infilled frames are given in Table 3. In this table both initial and prior to failure stiffness values are presented. Initial stiffness values were defined as the initial slope of the lateral load-displacement curve at the first forward cycle. Prior to failure stiffness was defined by the slope of the line drawn from the origin to the forward peak point of the last load cycle. In the last column of this table, ratios of the initial stiffnesses of infilled frames to those of bare frames are given. As can be seen from this column, even in specimen R4-N-L-15, where the lap splice length was very short (15 bar diameters), the stiffness ratio was nearly 20. This ratio was highest in S5-N-L-40, which had an undamaged frame.

Frames B3-N-L-40 and B4-N-L-15 were constructed with the deficiencies mentioned earlier. The only difference between these two specimens was the length of the lapped splices in the column longitudinal bars. The lap lengths in B3-N-L-40 and B4-N-L-15 (and in the corresponding infilled frames R3-N-L-40 and R4-N-L-15) were 40 and 15 bar diameters respectively. Both B3-N-L-40 and B4-N-L-15 had "heavy damage" prior to the introduction of the infill. Although the lap lengths in these two specimens were different, their stiffness ratios were almost the same.

All infilled frames with lapped splices in the column longitudinal bars failed by yielding of longitudinal reinforcement along a horizontal crack at the foundation level. At this stage slip occurred in the lapped splice regions of the columns. In Fig. 10, the separation of the infill from the foundation and damage of the test specimen in the vicinity of the foundation beam are shown. This type of failure was designated as "flexure + dowel slip".

Table 3 Stiffness of the test specimens

Specimen Id.	Experimental values				
	Stiffness		Stiffness degradation	Normalized initial stiffness[a]	Ratio of initial stiffnesses (Infill frame/bare frame)
	Initial (kN/mm)	Prior to failure (kN/mm)	(%)	(kN/mm)	
B1-C-H-00	4.3	0.8	81	4.0	24.3
R1-C-H-00	104.0	19.6	81	97.2	
B2-C-H-00	4.9	1.1	78	4.5	20.4
R2-C-H-00	94.1	26.1	72	91.8	
B3-N-L-40	3.5	1.1	68	4.5	19.5
R3-N-L-40	100.0	14.0	86	87.8	
B4-N-L-15	3.3	0.9	71	4.5	19.7
R4-N-L-15	96.1	11.7	87	89.0	
B5-N-L-40	-	-	-	-	28.6
S5-N-L-40	127.5	68.6	46	129.1	

[a]Normalized initial stiffness is defined as the initial stiffness multiplied by $\sqrt{20/f_c'}$, where f_c' is the compressive strength of the infill wall concrete in MPa.

Table 4 Summary of the lateral strength of the infilled specimens

Specimen Id.	f_c' (Infill) (MPa)	Tensile reinforcement ratio for I-Section (%)	Panel reinforcement ratio (%)	Measured maximum base shear (kN)	Measured maximum base moment (kN)	Measured nominal shear stress (MPa)
R1-C-H-00	22.9	1.8	0.75	286.4	500.0	$1.00\sqrt{f_c'}$
R2-C-H-00	21.0	1.8	0.75	310.9	539.6	$1.13\sqrt{f_c'}$
R3-N-L-40	25.9	0.9	0.75	137.3	240.5	$0.45\sqrt{f_c'}$
R4-N-L-15	23.3	0.9	0.75	98.1	169.5	$0.33\sqrt{f_c'}$
S5-N-L-40	19.5	0.9	0.75	180.4	312.9	$0.68\sqrt{f_c'}$

[a]Values on the third column include the longitudinal reinforcement of the boundary columns and the vertical reinforcement of the infill.
[b]Measured nominal shear stress is defined by $\tau = V/(t \times d)$, where d_= 0.8 l_w, l_w = 1500 mm and t = 50 mm. Here f_c' is in MPa.

It should be noted that in this test program, the lapped splices were severely punished since the lateral load resulted in axial tension in one of the boundary columns. In the infilled specimens having lapped splices in the column longitudinal bars no cracks were observed on the second story infills (Fig. 10). No slip was observed in the dowels that connected the infill to the frame members. Slip occurred only in the foundation dowels.

The lateral strength of each infilled frame is given in Table 4. Maximum base shear and maximum moment carried by the infilled specimens are given in columns 6 and 7 of this table

Table 5 The comparison of lateral strength of the test specimens

Specimen Id.	Type of specimen	Maximum lateral strength V_{max} (kN)	Corrected V_{max}[a] (kN)	Strength increase due to infill $V_{max\text{-}inf}/V_{max\text{-}bare}$
B1-C-H-00	Bare Frame	33.3	-	8.6
R1-C-H-00	Infilled Frame	286.4	267.6	
B2-C-H-00	Bare Frame	34.1	-	9.1
R2-C-H-00	Infilled Frame	310.9	303.4	
B3-N-L-40	Bare Frame	13.5	-	10.1
R3-N-L-40	Infilled Frame	137.3	120.6	
B4-N-L-15	Bare Frame	11.8	-	8.3
R4-N-L-15	Infilled Frame	98.1	90.9	
B5-N-L-40 [b]	Bare Frame	13.5	-	13.3
S5-N-L-40	Infilled Frame	180.4	182.74	

[a]Corrected $V_{max} = \sqrt{20/f_c'} \times V_{max}$
[b]Undamaged bare frame identical with B3-N-L-40

respectively. In the last column of Table 4, calculated nominal shear stresses are tabulated. Shear stresses were calculated considering a rectangular cross section. The length of the rectangle was $0.8l_w$ and the width was t = 50 mm. Here l_w was the distance between the exterior faces of the boundary columns (l_w = 1500 mm).

As can be seen from this table, nominal shear stresses reached in infilled frames with properly designed and detailed frames (R1-C-H-00 and R2-C-H-00) were approximately twice the stresses reached in the other specimens, which had the deficiencies mentioned previously. This difference was especially notable in the cases of specimens having inadequate splice lengths. It should be noted that in Specimens R1-C-H-00 and R2-C-H-00, the ratio of longitudinal reinforcement in the boundary columns was three times as much when compared to the others. The higher reinforcement ratio resulted in higher flexural capacity, and thus in higher shear stress. The lowest shear stress was obtained for R4-N-L-15 which had the shortest splice length (15 bar diameters). These comparisons clearly demonstrate the adverse effect of lapped splices on the behavior of infilled frames. It should also be noted that before the introduction of the infills to the damaged frames, no repair was made.

In the third column of Table 5, maximum base shears, recorded during the tests, are presented. The relative strengths of infilled frames with respect to companion bare frames are listed in column 4 of Table 5. As can be seen in this column, even the lowest ratio observed in the case of R4-N-L-15 was greater than eight. This clearly demonstrates how the strength increases significantly with the introduction of the infill even when the damaged frame has several deficiencies. In the fourth column of this table, normalized values of the maximum base shear, V_{max}, are given. The normalization was made by correcting V_{max}, to a concrete strength of 20 MPa.

Concrete compressive strength of the frame columns was not expected to affect the strength of the infilled frame significantly, since the failure mode was basically flexure. The quality of concrete in the frame members, however, has a definite influence on the resistance of connecting dowels and column lap splices, since the success of the anchorage of dowels and anchorage of longitudinal bars is closely related to the concrete quality of the frame (Orangun *et al.* 1977).

The strengths of R1-C-H-00 and R2-C-H-00 were much higher than those of the rest of the

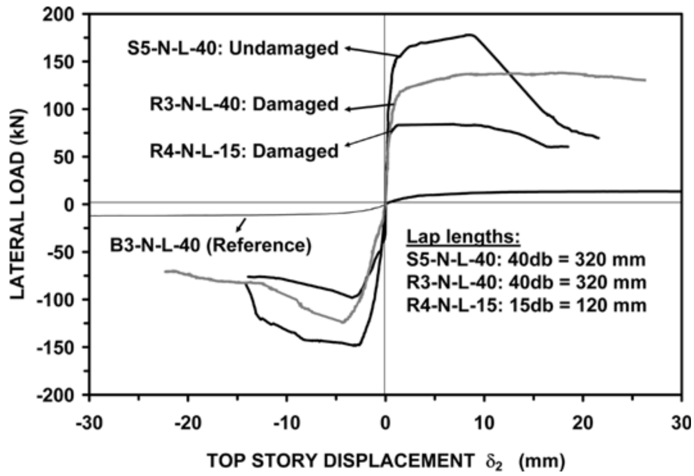


Fig. 11 Comparison of envelope curves of infilled frame specimens having lap splices in column longitudinal bars : Specimens B3-N-L-40, R3-N-L-40, R4-N-L-15, S5-N-L-40

infilled frames. The frames of these two specimens were properly detailed and the longitudinal bars of the columns were not lapped. Moreover, the column longitudinal reinforcement ratio in R1-C-H-00 and R2-C-H-00 was three times those of R3-N-L-40, R4-N-L-15 and S5-N-L-40. Since the primary failure of all infilled specimens was flexure, the column reinforcement ratio had a significant effect on strength.

In Fig. 11 envelopes of the hysteretic load-top story displacement curves of infilled frames R3-N-L-40, R4-N-L-15 and S5-N-L-40 and bare frame B3-N-L-40 are shown. It should be noted that B5-N-L-40 was identical with B3-N-L-40. In both R3-N-L-40 and S5-N-L-40, the splice length in column longitudinal bars was 40 bar diameters where R4-N-L-15 had a splice length of 15 bar diameters. In R3-N-L-40, the infill was introduced to a damaged frame (B3-N-L-40) whereas in S5-N-L-40 the infill was introduced to an undamaged frame (B5-N-L-40). Specimen S5-N-L-40 with

Table 6 Summary of the test results

Specimen Id.	Max. load in forward cycles (kN)	Max. load in backward cycles (kN)	2nd story interstory drift ratio at Max. load (%)	1st story interstory drift ratio at Max. load (%)	Failure mechanism
B1-C-H-00	33.3	27.2	2.63	3.06	Column Sway Mechanism (Flexure)
R1-C-H-00	263.8	286.4	1.00	0.89	Flexure + Diagonal Tension
B2-C-H-00	34.1	32.4	2.03	3.64	Column Sway Mechanism (Flexure)
R2-C-H-00	245.2	310.9	0.67	0.67	Flexure + Diagonal Tension
B3-N-L-40	13.5	12.2	1.34	1.61	Column Sway Mechanism (Flexure)
R3-N-L-40	137.3	125.1	0.87	0.68	Flexure + Dowel Slip
B4-N-L-15	11.8	10.8	1.02	1.48	Column Sway Mechanism (Flexure)
R4-N-L-15	84.3	98.1	0.15	0.24	Flexure + Dowel Slip
B5-N-L-40	-	-	-	-	-
S5-N-L-40	180.4	149.1	0.40	0.54	Flexure + Dowel Slip

an undamaged frame had a higher strength compared to R3-N-L-40 that had a damaged frame. The increased strength was not only due to the presence of undamaged frame but also due to the increased area of dowels at the foundation level. The initial stiffness of R3-N-L-40 with the damaged frame was 30 percent less than that of S5-N-L-40. Moreover, the stiffness degradation in R3-N-L-40 was also faster when compared to S5-N-L-40.

When the envelope curves of infilled frames given in Fig. 11 are compared with that of the bare frame (B3-N-L-40), the significant improvement in both strength and stiffness due to infill application can be observed. On the same figure, the difference in behaviors of R3-N-L-40 and R4-N-L-15 can be seen. The splice length in column longitudinal bars in R3-N-L-40 and R4-N-L-15 was 40 and 15 bar diameters, respectively. Although the difference in stiffnesses was insignificant, the reduction in strength due to shorter splice length was approximately 30 percent. This reduction occurred in spite of the fact that the area of connecting dowels at the foundation level of R4-N-L-15 was twice that of R3-N-L-40.

Among the common deficiencies observed in existing buildings (listed in the previous section) lapped splices in column longitudinal bars with inadequate length made above the floor level seem to have the most adverse effect on the behavior and strength of infilled frames. A summary of the test results and the type of failure observed are given in Table 6.

5. Conclusions

Low and medium rise residential and office buildings in Turkey have experienced considerable damage during recent earthquakes which caused significant casualties and financial loss. Subsequently, many of damaged buildings had to be rehabilitated effectively and economically in a short period. In practice, several techniques have been utilized for the rehabilitation of damaged reinforced concrete buildings. One of the most effective method for improving the lateral strength and stiffness of such frame buildings is to introduce reinforced concrete infill walls into strategic bays in both directions. Experimental data is needed to estimate the strength and stiffness of infilled frames especially with non-ductile damaged frames. Extensive experimental research has been conducted at METU on one-bay, one-story, and one-bay, two-story infilled frames (Ersoy and Uzsoy 1971, Altin *et al.* 1992, Turk 1998, Canbay *et al.* 2003, Sonuvar *et al.* 2004). In the study reported herein, a one-bay, two-story reinforced concrete frame was used as the test specimen. The reinforced concrete infills were added to the frame, which was damaged under reversed-cyclic lateral loading. In the test specimen, the frame members included most of the common weaknesses encountered in residential buildings in Turkey as well as in other countries. The following conclusions can be drawn based on the experimental results reported in this paper. It is important to note that these conclusions are based on the test data obtained from the tests on 1/3 scale specimens. Further tests may be needed to generalize these conclusions to full scale structures.

1. The introduction of reinforced concrete infills increased the strength and lateral stiffness significantly. Even in the case of Specimen R4-N-L-15, in which the most inferior response was observed, a nineteen-fold stiffness increase and an eight-fold strength increase were achieved as compared to the companion bare frame.
2. In general, the damage level of the frame (light or heavy) did not seem to affect the behavior of the infilled frame significantly, provided that the infills were connected to the frame members properly.

3. Although the damaged frame members and joints were not repaired, the behavior of the infilled frames was quite satisfactory.
4. The presence of lapped splices in column longitudinal bars above the floor level had an adverse effect on the strength of the infilled frame. This adverse effect was more pronounced when the lap length was small. The strength of R4-N-L-15 with a splice length of 15 bar diameters was about 30 percent less as compared to R3-N-L-40 which had a splice length of 40 bar diameters.
5. The infilled frame with a damaged frame (R3-N-L-40) exhibited inferior behavior when compared to the one with an undamaged frame (S5-N-L-40). When these two specimens are compared, it is seen that the strength of R3-N-L-40 was 20 percent less compared to S5-N-L-40. The reduction in the initial stiffness due to this difference [damaged or undamaged frame] was about 30 percent. It should be noted that the area of dowels connecting the infill to the foundation in S5-N-L-40 was twice that of R3-N-L-40. Increasing the foundation dowels in S5-N-L-40 probably increased the capacity of this specimen.
6. The failure of all infilled frames was initiated by yielding of the vertical reinforcement of the infill and the longitudinal bars of the boundary columns. Therefore the ratio of longitudinal reinforcement in the boundary columns had a significant effect on the strength of the infilled frames.
7. Concrete strength of the frame to which the infill is applied did not influence the behavior of the infilled frame significantly. However low strength concrete requires a greater anchorage length for the dowels connecting the frame members to the infill.
8. To achieve the full lateral capacity of the infilled frames, it will be required to apply local strengthening to the boundary columns in bottom regions where the lap splice length of bars is inadequate (Valluvan *et al*. 1993, Sonuvar *et al*. 2004).
9. The tests reported in this paper indicated that the rehabilitation technique employed in this study may effectively be used in the seismic rehabilitation of reinforced concrete structures.

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Conversion Factors

1 mm = 0.0394 inch; 1 MPa = 0.1450 ksi, 1 kN = 0.2248 kip

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Notation

The following symbols are used in this paper:

d_b : Diameter of column longitudinal bars
f_c' : Compressive strength of concrete
f_y : Tensile yield strength of reinforcement
h : Story height, infill wall height
h_1 : First story height of the infilled frame
h_2 : Second story height of the infilled frame
l_o : Column lap splice length
l_w : width of infilled frame specimen (distance between the exterior faces of boundary columns)
N : Applied axial load on each column of the test specimens
N_o : Uniaxial load capacity of a cross-section
t : Thickness of the infill panel
V : Base shear
$V_{\max}$: Maximum base shear applied to test specimens
$V_{\text{max-inf}}$: Maximum base shear applied to infilled frame specimens
$V_{\text{max-bare}}$: Maximum base shear applied to bare frame specimens
δ : Lateral displacement
δ_1 : First story lateral displacement
δ_2 : Second story lateral displacement
ρ : Ratio of the column longitudinal reinforcement