Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 880

SEISMIC UPGRADING OF DEFICIENT REINFORCED CONCRETE FRAMES WITH INTERNAL STEEL FRAME

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ABSTRACT

Performance of post installed internal steel frames (ISFs) to upgrade the existing deficient reinforced concrete (RC) frames was investigated experimentally in this study. Three strengthened and one as-built reference portal frame specimens were tested under constant gravity load and reverse cyclic lateral displacement excursions. ISFs connected to RC frames with and without anchors were tested. Test results indicated that placing an interior steel frame without any anchors as a new lateral load resisting system can increase the lateral stiffness and strength significantly. The strength increase was found to be limited by the horizontal joint shear strength of the RC frames. On the contrary, higher strength enhancements were observed upon using anchor connections between RC and steel frames. It was found that if the lateral strength needed is less than the horizontal joint shear strength, it is more practical and economical to use anchorless internal steel frames. However, if the lateral strength demand on the steel frame exceeds the horizontal joint shear capacity of the frames, it is preferable to use ISF with internal anchor connectors. It was also observed that ISFs also perform as means of gravity collapse prevention systems under the conditions of severe strength degradation and second order effects at high drift deformation demands. Finally the available drift capacity of strengthened frames in relation with the drift limits given in seismic rehabilitation design guidelines is discussed.

Introduction

Existing deficient reinforced concrete (RC) structures suffered greatly from recent earthquakes in Turkey, Taiwan and Pakistan because of insufficient lateral strength and deformation capacity. The common deficiencies that are mainly due to gravity load design in 1970s resulted in poor seismic performance of these RC structures. Some of the important deficiencies are poor detailing of transverse reinforcement in beams, columns and joints, excessive bond slip of longitudinal reinforcement due to the use of plain bars, discontinuity of the longitudinal reinforcement in beams and columns, and use of low strength concrete

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(approximately between 8 to 15 MPa uniaxial compressive strength). In addition to these deficiencies, irregularities in plan and elevation such as formation of soft and weak first storey are some commonly observed features of the residential building stock. There are a number of seismic retrofitting techniques, some applied globally to the overall structure and others applied locally at the individual member level to reduce the seismic vulnerability of RC structures. For the former techniques, to improve the lateral resistance of existing RC structures, a new lateral load carrying system is introduced by using structural walls (Canbay et al. 2003), steel braces (Özcelik and Binici 2008) or FRP diagonal braces integrated on the infill walls (Binici et al. 2007). Hence, installing an ISF within bays of the deficient RC frames can be categorized within global level strengthening techniques. According to the author's knowledge there is only limited amount of research especially on the use of post installed anchorless structural systems that is only available in the Japanese literature (Takahiro and Yasuyoshi 2001). In order to investigate the feasibility of using such post installed lateral load resisting systems to improve the seismic performance of the deficient RC structures, an experimental program was conducted. The use of ISF is worth evaluating for seismic strengthening due to advantages such as little disturbance on the functioning of the building and its occupants during retrofit and its ability to accommodate openings for architectural purposes and being effective in gravity collapse prevention. In this study ISF retrofitting technique with or without anchor connection with RC members was developed. The performance and limitation of the ISF with and without anchor connection was critically examined.

Specimen details and test setup

Four one bay-one story portal frame specimens having planar dimensions of 1400 mm x 1000 mm as shown in Fig. 1 were constructed and tested for this experimental study. The dimension of the frame was scaled by a factor of 1/3 from the prototype structure previously studied by (Ozcelik and Binici 2006). All of the RC frame specimens had similar dimensions and reinforcement ratios to allow a uniform basis of comparison. The column dimensions were 100 mm x 150 mm with four 8 mm diameter plain bars (Fig. 1). The mechanical properties of the longitudinal reinforcement obtained from uniaxial test were: 330 MPa yield strength, 465 MPa ultimate strength and 30 % elongation. The beams were cast with an effective slab width of 450 mm and a slab thickness of 55 mm with the objective of including the behavior of slab and placing the dead weight conveniently as in Fig. 1. 90 degree stirrups having 4 mm diameter plain bar were used for both columns and beams. Stirrup spacing of the columns were taken equal to the smaller dimension of the column section size (100 mm) to simulate insufficient confining details. The beam-column joint had only one column stirrup extending into the joint, hence it was insufficiently confined. The mechanical properties of the transverse reinforcement obtained from uniaxial test were: 270 MPa yield strength, 374 MPa ultimate strength and 23 % elongation. The 28-day target uniaxial compressive strength of concrete was 8 MPa for all specimens with a maximum 7 mm aggregate size. In place concrete strength at the test day for each specimen are presented in Table 1.

Testing procedure

The constant gravity load of 62 kN was applied with steel blocks which were equal in planar size to the slab width (Fig 1). Table 1 presents the axial load ratio (i.e. ratio of gravity load

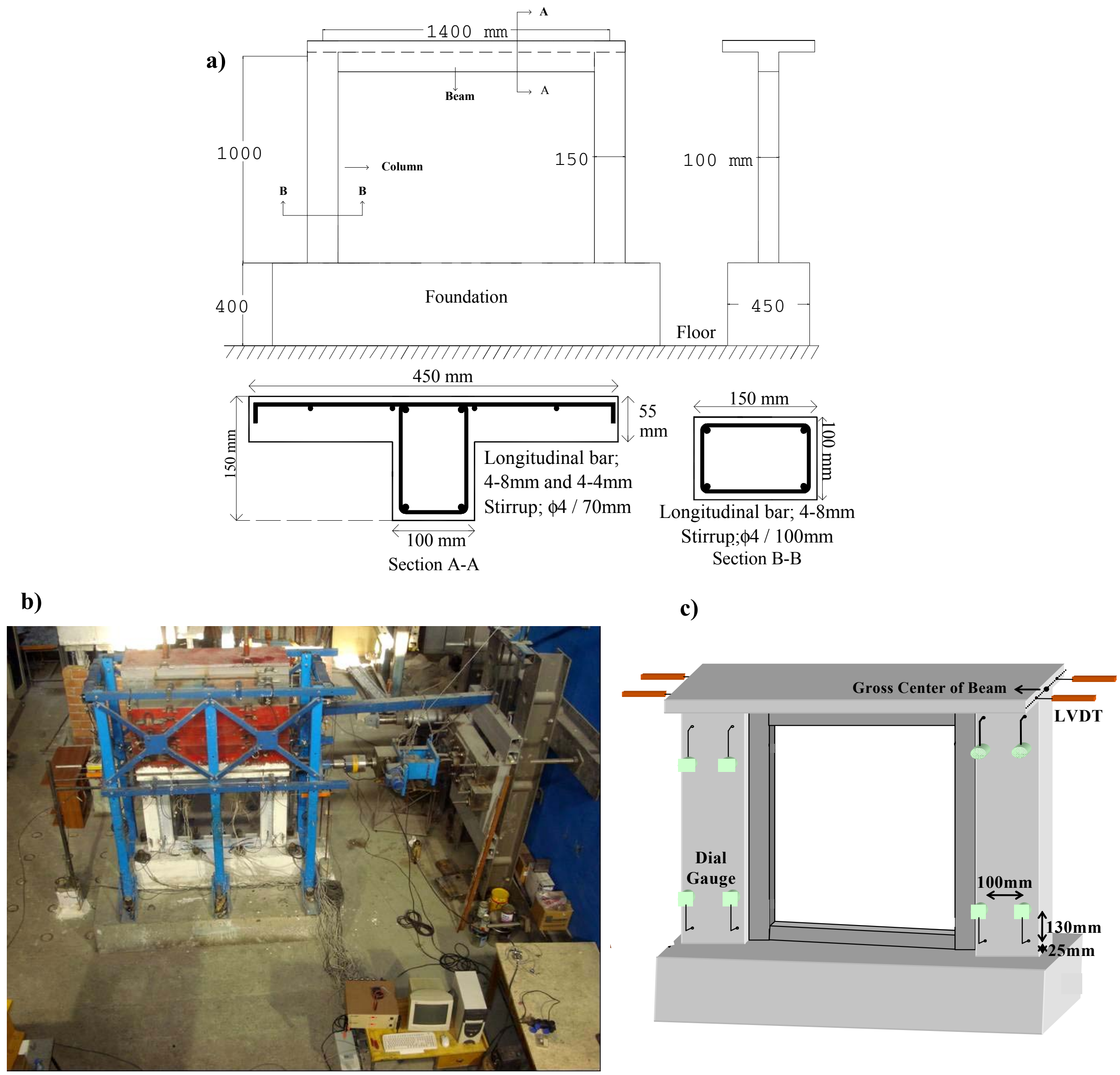

Figure 1. Specimen and test setup; (a) Dimensions of specimen; (b) Test setup; (c) Instrumentation scheme

Table 1. Experimental program

Specimen	Steel Frame Member Sizes (mm)		Steel Frame Member Cross Section Area (mm^2)		Strengthened Method	Concrete Compressive Strength (Mpa)	Axial Load Ratio
	Column	Beam	Column	Beam			
SP1	-	-	-	-	-	8.1	0.18
SP2	80x80x4 (27.8*)	70x70x3 (17*)	1216	804	without anchor	7.5	0.19
SP3	I-140 (81.9*)	I-120 (54.7*)	1820	1420	without anchor	10.6	0.16
SP4	I-140 (81.9*)	I-80 (19.5*)	1820	761	with anchor	7.4	0.19

* Section modulus in cm^3

to total axial load carrying capacity of columns). An additional steel frame was constructed around the specimen to refrain the dead weight falling in the case of a sudden gravity collapse. Four displacement measurements at the story level were taken using LVDTs to record interstory drift deformations. Two sets of electronic dial gauges were placed at the top and bottom of each column to measure axial deformations, hence curvature changes. An electromotor driven displacement controlled screw jack with a maximum loading speed of 0.2 mm/sec was used to apply lateral displacement excursions. Cyclic loading was introduced by controlling the drift ratio for all specimens. Starting from 0.5% drift ratio, 0.5% drift ratio increments were imposed until2% drift ratio. Afterwards, drift ratios (at each drift ratio two cycles were performed) were incremented by 1% until 5% drift ratio was reached.

Strengthened Specimens with ISF

Three of the four specimens were strengthened with ISF after placing gravity load on the RC frame in order to simulate the actual retrofit conditions on site. The ISF was composed of steel beams and columns with welded connections. End plates at the top and bottom ends of the column and angles were used to construct a nearly rigid connection between the beam and column. There were two different methods used to install the ISFs. Fig. 2 shows the details of the installation methods, while Fig. 3 shows pictures of the ISFs installed in each of the RC frames. In the first method, no anchors were used between RC frame and ISF. A thin layer of repair putty was applied on RC column to obtain a smooth bonding surface. Afterwards, epoxy was wiped on putty and surface of steel columns and left for curing for three days. Finally, the steel beams were welded to the steel columns with details in Fig 2. In the second method, in addition to the epoxy bond used in method I, anchor rods (threaded rods) were used to achieve fully composite action between the RC frame and ISF. For the ISF with anchors, first, anchorage holes were drilled on the whole inner boundary of RC frame members (beam, column and foundation) and cleaned up by air blowing. Then epoxy primer was injected into these holes followed by the insertion of the anchorage rods. All the anchors were left for a curing time of three days. After curing of epoxy resin of anchors, a thin layer of repair putty was applied on the RC member on all surfaces that contact the ISF, as in method I to obtain a flat surface with enhanced shear-friction strength. Immediately after the application of repair putty, ISF beams and columns with predrilled holes at anchor locations were installed. The steel beams were welded to the steel column with details in Fig 2. A sufficient number of anchors were used to develop the plastic strength of the ISF based on beam hinging mechanism of the composite frame, which was estimated to be 8 times the lateral strength of the RC bare frame. The diameter of the anchorage rods and holes were 6 mm and 8 mm, respectively, whereas the depths of anchorages were selected as110 mm from the concrete surface. The mechanical properties of the anchorage rod obtained from uniaxial test were: 848 MPa yield strength, 1120 MPa ultimate strength and 11 % elongation capacity.

Test parameters

Four experiments were performed to investigate the behavior of ISF strengthened RC frames. The specimens were tested under roughly 20% column axial load ratio (i.e. axial load on a column divided by the axial load capacity of the column). First specimen (SP) was a reference bare frame without an ISF. There were mainly two parameters for the strengthened specimens in this study as shown in Table 1. First one was member type used for the ISF (square HSSs were

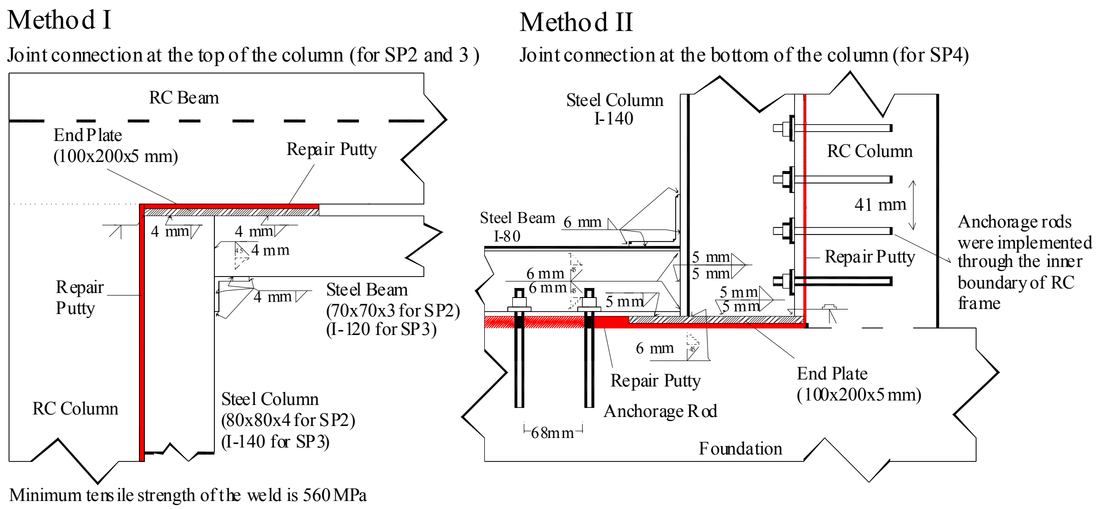

Figure 2. Connection details of test specimens

Figure 3. Pictures of test specimens during the construction

used in specimen SP2; I-sections were used in specimens SP3 and SP4). The second parameter was the ISF installation method (method I was used in SP2 and 3; method II was used in SP4).

Discussion on test results

Hysteretic response of the test specimens is presented in Fig. 4. This figure also shows important events such as plastic hinge formation in the RC columns (determined from curvature measurements), fracture initiation of ISF elements and joint failure of RC frame. Deformation levels corresponding to hinge formation from the measurements at the columns ends were marked on the cyclic responce, Fig 4. Pictures of test specimens during the test are presented in Fig. 5. Summary of test results are presented in Table 2.

The ultimate lateral strength was 13.7 kN for specimen SP1. Defining the lateral stiffness as the slope between the peak of positive and negative strength at first cycle (at ±0.5% drift ratio (DR)), the lateral stiffness of specimen SP1 was 2.5 kN/mm. All the possible column plastic hinges were observed at ±2.5 % DR. Upon further increase in loading amplitude for specimen SP1, pinching and severe stiffness degradation was observed. The ultimate lateral strength of specimen SP2 was about 4.0 times that of reference frame. The lateral stiffness of specimen SP2 was about 2.5 times that of the reference frame. For specimen SP2, the separation between RC and ISF columns were observed at ±2 % DR. The fracture initiation at the beam column connections of the ISF was observed at both ends of the beam along the fillet weld connecting the beam to the column at ±3% DR. Prior to the fracture in the ISF beams, while increasing

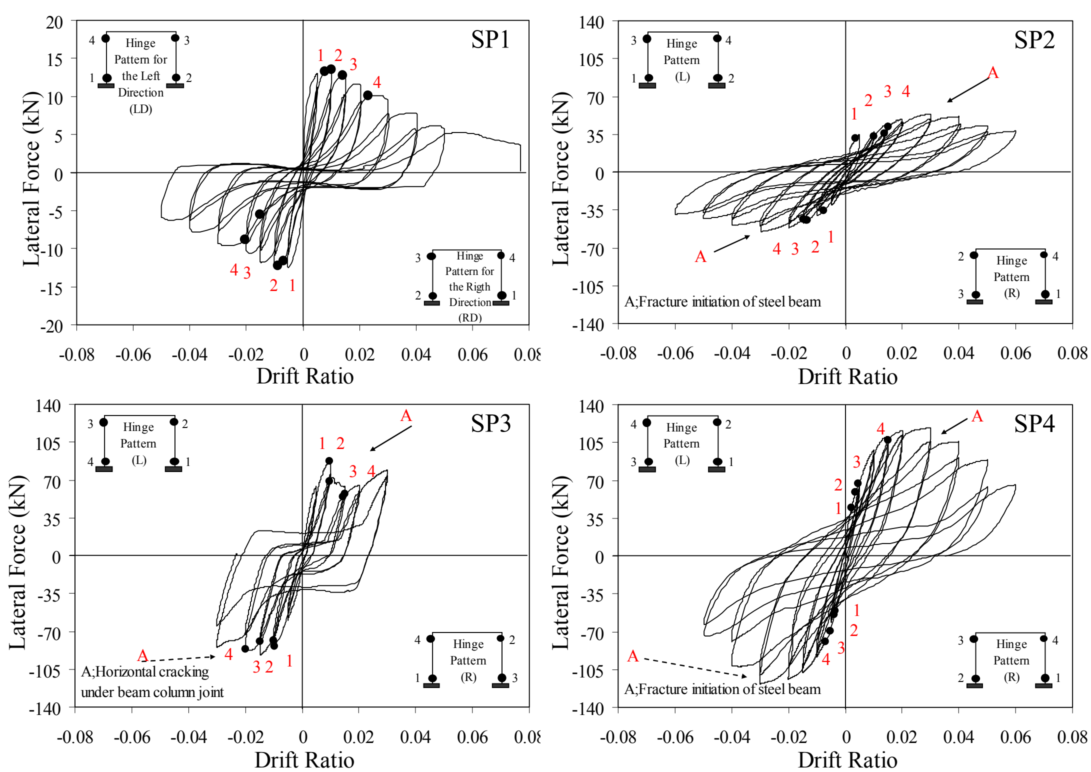

Figure 4. Cyclic response of specimens

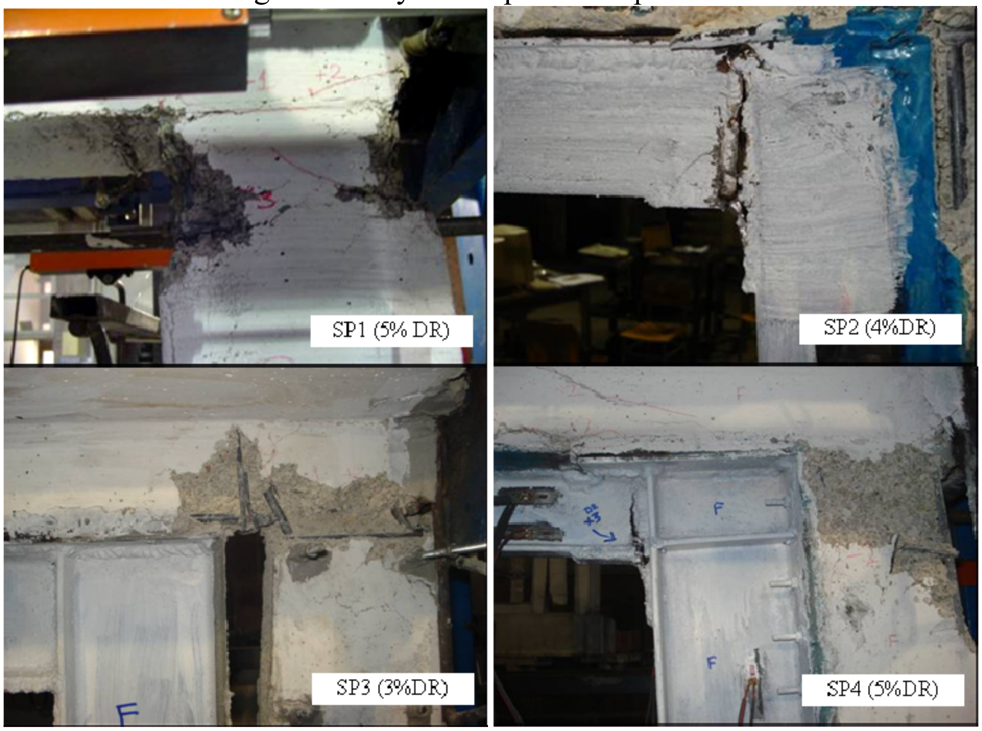

Figure 5. Pictures of the test specimens during the tests

Table 2. Test results

Specimen Name	Ultimate Lateral Load (kN)	Lateral Stiffness (kN/mm)	Displacement Ductility* Direction Rigth	Displacement Ductility* Direction Left	Dissipated Energy (kN.mm)	Drift Ratio (%) Capacity** Direction Rigth	Drift Ratio (%) Capacity** Direction Left	Drift Ratio (%) at Failure***	Failure Modes	Ratio of Ultimate Lateral Strength to Ultimate Lateral Strength of Bare Frame	Ratio of Stiffness to Stiffness of Bare Frame	Ratio of Energy Dissipation to Energy Dissipation of Bare Frame
SP1	13.7	2.5	2.6	2.8	2081.9	2.0	2.0	-	Column mechanism			
SP2	55.0	6.4	10.9	5.2	10184.4	4.0	4.0	4.0	ISF beam mechanism	4.0	2.6	4.9
SP3	91.8	11.9	3.1	3.1	11043.3	3.0	3.0	3.0	Beam-column joint failure	6.7	4.8	7.2
SP4	118.9	12.7	17.8	11.7	24553.5	4.0	4.0	4.0	Composite frame beam mechanism	8.6	5.1	11.8

* Ratio of the displacement at %15 lateral capacity drop to the displacement at the first yield curvature.
** Drift ratio at 15 % lateral capacity drop.
*** Drift Ratio at failure observed during the tests.

plastic hinge rotation was measured at the RC columns, limited damage was observed in the ISF.

After fracture of steel beams lateral strength decreased, Fig. 4. Steel columns had no visible damage during the test because of weak beam-strong column design procedure.

The ultimate lateral strength of specimen SP3 was about 6.7 times that of reference frame. The lateral stiffness of specimen SP3 at the first cycle was 11.90 kN/mm (about 4.8 times that of the reference frame). The serious limitation of the anchorless ISF was observed in specimen SP3. The excessive damage at the beam-column joint of the RC frame was observed soon after ±1 % DR. The separation between RC and steel columns was first observed at ±1.5 % DR. The horizontal shear resistance of the joint at the top level of ISF was observed to be mainly due to concrete contribution and dowel resistance of RC column longitudinal bars. After horizontal joint cracks widened and concrete contribution at the joint severely degraded with each altering cycle, longitudinal column bars at the joint started to resist against imposed lateral force demand under axial and shear deformation. As a result, the lateral strength of specimen SP3 decreased between ±1 and ±2 % DR due to the shear damage at beam-column connections. The strength enhancement observed after ±2 % DR can be attributed to the strain hardening effect of column longitudinal bars subjected to cyclic axial-shear loading, Fig. 4. No fracture initiation of members in ISF was observed during the test. The test results of specimen SP3 proves that the benefit of ISF may be limited if failure in the beam-to-column joint of the RC frame cannot be controlled.

The ultimate lateral strength of specimen SP4 was 118.9 kN (about 8.6 times that of reference frame). The lateral stiffness of specimen about 5 times that of the reference frame. The cyclic response of the specimen SP4 was controlled by the behavior of the composite sections of beam and columns. In case of double curvature in columns and beam, concrete part of the composite sections was subjected to tension or compression stress cycles. Therefore, when the concrete part of the composite section was subjected to tensile stresses, concrete contribution for the moment capacity of the composite section was lost and cracks widened at these locations. There was no anchorage failure in specimen SP4 during the test. No joint failure was observed and failure mode was fracture initiation of steel beam observed at ±3 % DR. SP4 sustained the highest lateral strength among the strengthened frames. Superior performance of this specimen can be attributed to the anchor connection provided between ISF and RC frame all around the inner boundary.

Fig. 6 compares the envelope of lateral strength versus deformation response obtained from all specimens. Fig. 7 indicates the cumulative energy dissipated at the completion of each DR up to the end of ±4% DR cycles, except for specimens SP3, in which beam-column joint failure occurred after completion ±3% DR cycles. Proposed upgrade schemes resulted in strength increases of 4 to about 9 times the strength of the reference RC frame. On the other hand, energy dissipation capacity increased by amount of 4.9 to 11.8 due to less pinching, stiffness and strength degradation. As a result, if the seismic energy induced by the ground motion is desired to be dissipated by the structure, the reference frame specimen SP1 had poor seismic energy absorption and dissipation capacity with a rapid strength degradation requiring upgrades. Upon upgrading, there was less pinching, stiffness degradation and larger energy dissipation capacity in frames strengthened with ISF. It can be observed that highest energy dissipation and strength enhancement was observed for specimen SP4 with fully composite section.

Based on the observations from the experiments, damage levels at the following drift ratios can be examined, Fig. 6; a) Minimum Damage is defined as the onset of steel beam yielding or first column hinging at the RC frame. In this performance limit, no significant damage except minor concrete cracking was observed.

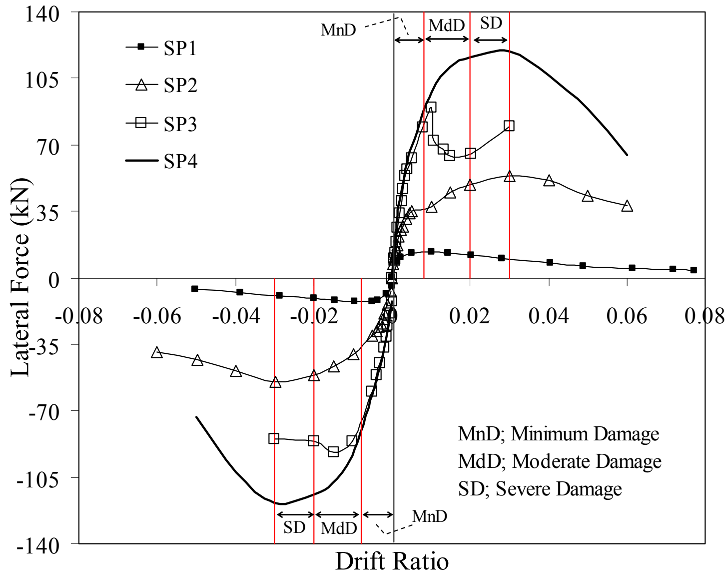

Figure 6. Envelope response of the test specimens

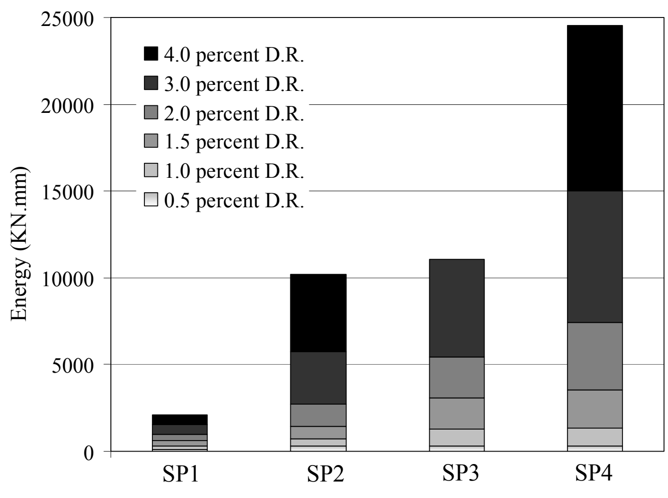

Figure 7. Energy dissipation capacity of the test specimens

0.8 % DR as minimum damage level can be accepted for strengthened frames with ISFs. b) Moderate Damage is defined at the occurrence of significant damage (i.e. plastic hinge formation in the column) on the RC frame. Significant yielding may occur at the steel beam. This performance limit was selected at 2 % DR. c) Severe Damage defined as the fracture initiation of ISF. 3 % DR is a safe limit for this damage level. The proposed values in this study are similar to the proposed drift ratio levels of Turkish Earthquake Code 2007 for similar performance levels of immediate occupancy, life safety, and collapse prevention.

Conclusions

The response of deficient RC frame before and after strengthening with ISF is examined experimentally in this study. The experiment results indicated that ISFs can effectively increase the stiffness, strength, and energy dissipation capacity of seismically deficient RC frame buildings. The strength increase is limited by the horizontal shear strength of the beam-column joints of the original RC frame. Hence, for target strength enhancements remaining below the joint shear capacity of the frame with post installed ISF, use of anchorless ISF can be acceptable. Alternatively, the ISF might be constructed to function compositely with the RC frame by using anchor connections. More costly and time consuming procedure has of course the benefit of obtaining higher strength enhancements than the previous alternative. Additional benefit of the ISF may be the collapse preventing nature of the ISF even after the RC frame lost its axial load carrying capacity. Hence, ISF can serve not only as a new lateral load resisting system but also may act as a backup axial load carrying mechanism. Observed damage levels (i.e. minimum damage, moderate damage and severe damage) are attempted to be correlated with the well known earthquake engineering demand parameter of drift ratio. Accordingly, minimum damage, moderate damage, and severe damage levels were identified to beat drift ratios of 0.8%, 2%, and 3%, respectively.

Acknowledgments

The research discussed in this paper was conducted at Middle East Technical University (METU)-Structural Mechanics Laboratory. Funding provided by TÜBİTAK (project no: 106M493) and ÖYP program (supported by Akdeniz University) are acknowledged.

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