Seismic Retrofitting of Deficient RC Structures with Internal Steel Frames by Using Pseudo Dynamic Testing Procedure

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SUMMARY:
Two story-three bay reinforced concrete (RC) frames with and without an internal steel frame (ISF) were tested by using continuous pseudo dynamic (PsD) test method. The ISFs, implemented to the interior span of the RC frame, consisted of fully composite beams and columns constructed by connecting new steel members to the existing deficient RC beams and columns. Test results indicated that the deficient RC frame retrofitted with ISF was sufficient to resist lateral demand imposed by Düzce earthquake record. The yielding of the steel members provided a ductile behaviour compared to the reference RC frame. Based on the observed damage state and dynamic response of the test frames, performance states were discussed for levels of imposed ground motion.

Keywords: Seismic Retrofit, RC Frames, Internal Steel Frame, Pseudo Dynamic Test

1. INTRODUCTION

There are numbers of life threatening buildings in the high seismic zone areas in many countries. Recent earthquakes (Northridge 1994, Kobe 1994, Kocaeli 1999, Taiwan 2003, India 2001, L'aquila 2009, Van 2011) revealed this fact that such structures need to be strengthened before a catastrophic earthquake may cause severe damage or collapse. The main deficiencies of the existing RC building in many of the developing countries owe either to lack of knowledge about seismic risk or to malpractice and insufficient quality control during construction. The poor quality control results in low strength concrete (in the range of 8 to 15 MPa), insufficient spacing of transverse confining reinforcement in beams, columns and joints, and insufficient splice length at column critical regions that may result in excessive bond slip of plain longitudinal reinforcement. While these deficiencies may be addressed by member-level seismic upgrade techniques, this paper focuses on a structural-level upgrade method.

Some of the main structural retrofitting techniques are adding structural walls (Jara et al., 1989; Altın et al., 1992; Canbay et al., 2003), steel braces (Badoux and Jirsa 1990; Bush et al. 1991; Pincheira and Jirsa 1995; Masri and Goel 1996; Maheri and Sabebi 1997; Ghobarah and Abou-Elfath 2001; Maheri and Hadjipour 2003; Molina et al 2004; Ozcelik and Binici 2006; Ozcelik and Binici 2008; Ozcelik et al., 2012), bonding FRP diagonal braces on the infill walls (Binici et al. 2007; Erdem et al. 2006). Precast concrete shear walls that fit perfectly into the existing frame (for example Kazunori et al. (1999)), steel frames attached externally to the perimeter of the existing frame described by Tsunehisa et al. (1999) and steel frames attached within the frame without using any anchors by Takahiro and Yasuyoshi (2005) and Ozcelik et al. (2011) are some other alternative retrofitting techniques. Among the techniques presented above, the most common structural-level strengthening technique is to integrate new structural RC shear walls with anchors because such walls provide significant lateral stiffness, strength, and energy dissipation capacity (Jara et al., 1989; Altın et al., 1992; Canbay et al., 2003). However, addition of the structural wall requires interrupting building use for a substantial period of time and may conflict with architectural requirements. Moreover, the shear wall retrofitting is restricted to only perimeter frames due to minimizing the disturbance to occupants (Pincheira 1993). Hence, this causes the concentration of the lateral forces in the walls that may require heavy strengthening works of the foundation (Jara et al., 1989).

The choice of the retrofitting technique to be applied on deficient buildings depends on the economical and architectural concerns, foundation type of the structure, disturbance limits to owners and occupants, government permissions, and design codes. Hence, the structural engineers need many options for seismic retrofit as one approach may not fit the needs of all cases. From this perspective, installing a new ductile steel frame seems to be attractive due to a number of reasons. From an architectural point of view, internal steel frames, when installed in the interior bays may better accommodate openings. In addition, as ISF may change the behaviour of a nonductile RC frame into an RC-steel composite ductile frame, they may successfully sustain large deformation demands with less damage. Furthermore, they may act as additional gravity load resistance elements to act as fuses against gravity collapse. Hence, the authors believe that internal steel frames (ISF) installed at strategically correct locations (for example in the weak and soft first story bays as symmetrically as possible so that undesirable torsional effects are minimized) may prove to be extremely efficient in seismic retrofits. The literature review reveals that there is lack of experimental data on seismic retrofit of multi bay multi storey RC frames with easy to install Internal Steel Frames (ISFs) under realistic simulated earthquake demands. The steel frames are installed within bays of the deficient RC frames, and are thus referred to in this paper as internal steel frames (ISFs). In order to investigate the performance of ISF retrofitted RC frames, an experimental research program was conducted to examine the use of steel frames to strengthen seismically deficient RC frames at the structural level.

2. EXPERIMENTAL PROGRAM

Figure 1 shows the test frame which was ½ scaled versions of a typical frame in the prototype RC frame building. Both the reference frame with infill walls (Kurt, 2010) and the ISF strengthened frame (Ozcelik, 2011) had the same RC frame details. Both distributed live (250 kg/m^2) and dead (300 kg/m^2) loads on slabs were considered in the design of prototype building. These loads produced about 13 and 23% column axial load ratio (ratio between applied load and column axial load capacity) at the first story exterior and interior columns, respectively. Fig. 1 shows the reinforced concrete member details of the test specimens. The RC frames consisted of 150 mm × 150 mm columns with four 8-mm diameter plain longitudinal reinforcement resulting in about 1.0% longitudinal reinforcement ratio (Fig. 1). Although the modern seismic resistance design codes (ACI 318 and Turkish Earthquake Code 2007 (TEC-2007)) requires stirrups to be anchored using 135 degree hooks, 90 degree hooks were used for all columns and beams to simulate the detailing deficiency of the old construction practice. The stirrup spacing of the columns was 100 mm in the plastic hinge regions to simulate insufficient confining steel reinforcement details. Moreover, there was not any stirrup at the beam-column joint and the strong column-weak beam requirement was violated for the test specimens.

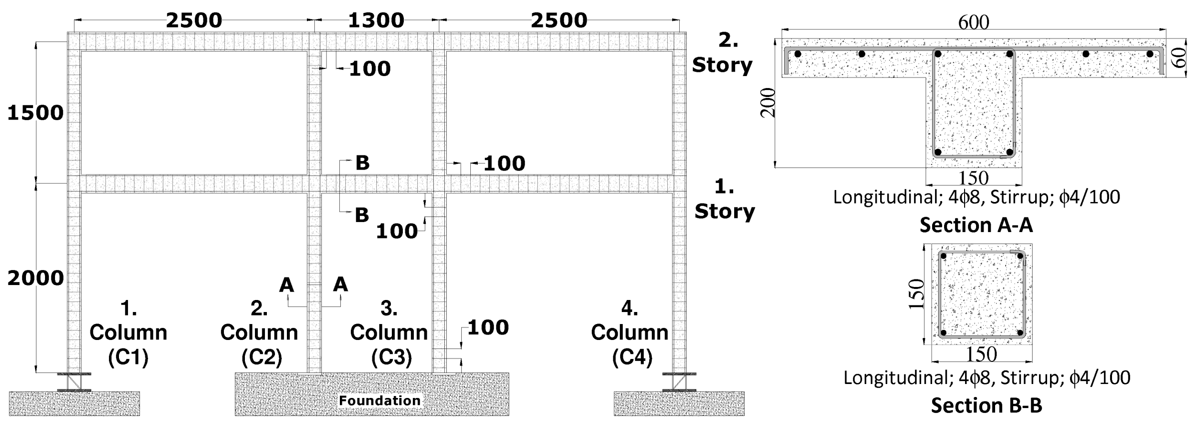


Figure 1. Test structure and test frame

The target compressive strength of the concrete and the maximum size of the aggregate was about 7.5 MPa and 12 mm, respectively. This low strength concrete with such maximum aggregate size was commonly observed in the existing deficient structures of the Turkish RC building stock as reported by the field investigations (Taşdemir et al. 1999; Dogangun 2004; Mazılıgüney et al. 2008). The in situ concrete strength of the each specimen is indicated in Table 1. The mechanical properties of the longitudinal and transverse reinforcement were 330 MPa and 290 MPa, respectively (Ozcelik 2011). The IPE200 wide flange section and steel plates were used for the ISF application. The yield strength of the IPE200 and steel plates were 310 and 330 MPa, respectively (Ozcelik 2011; ASTM 2004).

Table 1. Concrete Strength (MPa)

Ground Motion	Reference	ISF	
		1.story	2.story
50%	7.4	7.5	7.3
100%	7.4	7.5	7.3
140%	7.4	7.5	7.3

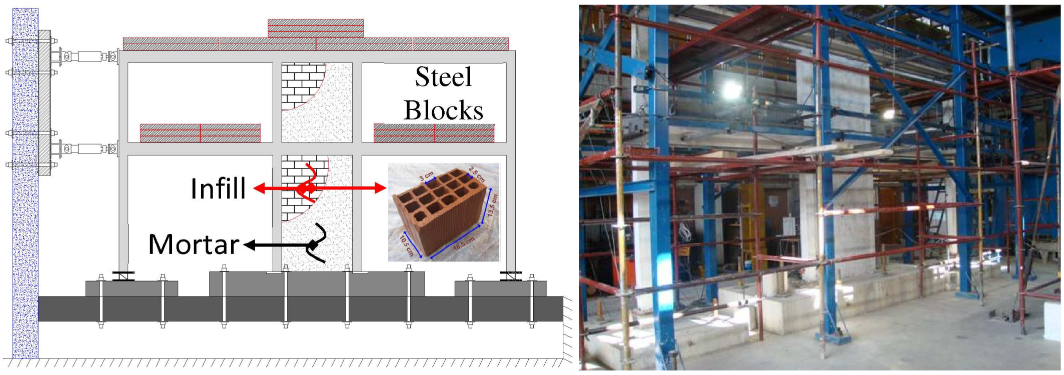
The reference frame with hollow clay brick infill wall at the interior span is shown in Fig. 2a. The brick member size was 110x130x130mm and its uniaxial compressive strength was 14 MPa. The mortar applied on the brick was 12 MPa. It is important to note that the infill brick wall was constructed after the steel block applied on the RC frame to represent the existing structures. Although the limited data about reference frame is given in this paper, its details were studied in elsewhere (Kurt 2010; Kurt et al., 2010).

The test setup of the ISF is shown in Fig. 2b. The ISF consisted of fully composite beams and columns constructed by connecting new steel members to the existing deficient RC beams and columns. The wide flange section (IPE200) and steel plates (7 mm thick plate) were used to construct the composite columns and beams, respectively. These steel members were connected to the RC frame by using anchors. The steel columns at the base were welded to the base plate post-installed to the foundation. The connection details of the composite beams and columns are available in elsewhere (Salmon et al., 2009; Ozcelik, 2011).

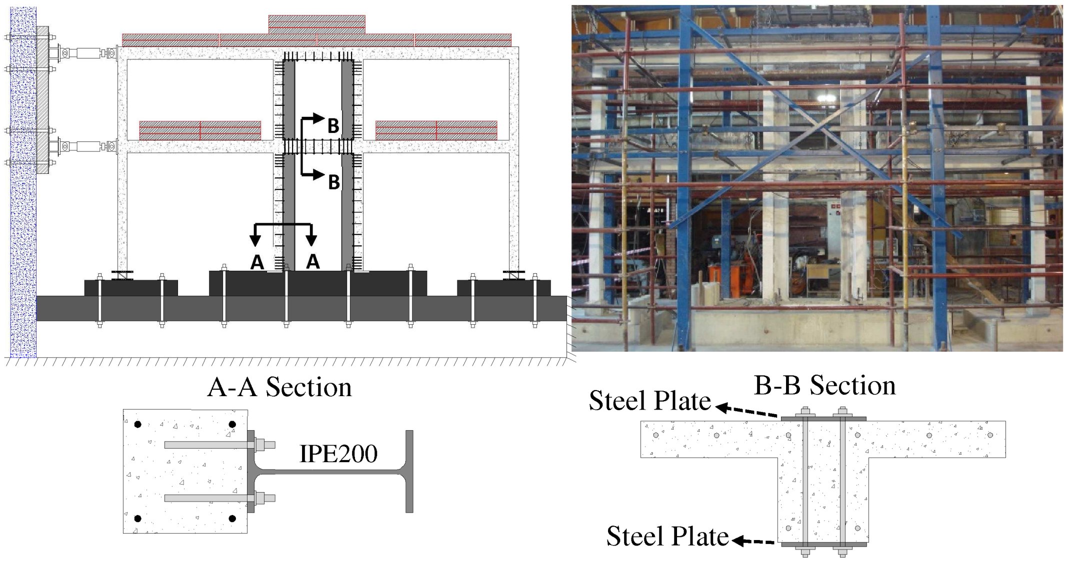
3. INSTRUMENTATION AND LOADING

Figure 3 shows the instrumentations and loading systems. Two computer controlled actuators were used to impose lateral displacements to the RC frame (Fig. 3a). The 500-kN-load cells were placed between the actuators and the RC frame to measure the lateral force at each story level. Two Linear Variable Differential Transducers (LVDTs) were used to measure the floor displacement at first and second floors. Two LVDTs placed 150 mm apart from each other within the potential plastic hinge zone for each column base were used to measure the column curvatures (Fig. 3c). Two in house fabricated transducers (Fig. 3d) were placed under the exterior columns (column C1 and C4, see Fig. 1) to acquire exterior column base moment, shear and axial force. The details about the transducer production and calibration are available elsewhere (Canbay et al., 2004). The axial load on the columns was applied by using steel blocks (Fig. 3d). The locations of the steel blocks serving as gravity loads are shown in Fig. 3b. The infill walls and the ISFs were placed in the interior span of the RC frame hence steel blocks were not placed in the interior span of the first story. A steel frame (Fig. 3a) was constructed around the RC frame to act as a safety in holding the steel blocks.

The continuous pseudo dynamic testing method proposed by Molina et al. (1999) was utilized for the PsD experiments. A 2x2 lumped mass matrix was used for numerical integration. The masses of the first and second story were 5000 and 7000 kg, respectively. Instead of a synthetic ground motion an actual ground motion was found to simulate the hazard level that could be expected for the prototype building. Hence, the north-south component of 7.1 moment magnitude 1999 Duzce ground motion was used. The peak ground acceleration (PGS) of Duzce was scaled at three different levels from low to high seismic intensity. 50%, 100% and 140% PGA scaling was used unless a component and structural failure was observed. Fig. 4 displays acceleration time series of the motion and pseudo acceleration spectrum of the motion, respectively. The original ground motions were compressed in time by a factor of $1/\sqrt{2}$ to include scale effects with respect to similitude law (Bertero at al., 1984; Elkhoraibi and Mosallam, 2007).



a) Reference Frame



b) ISF retrofitted frame

Figure 2. Test setup

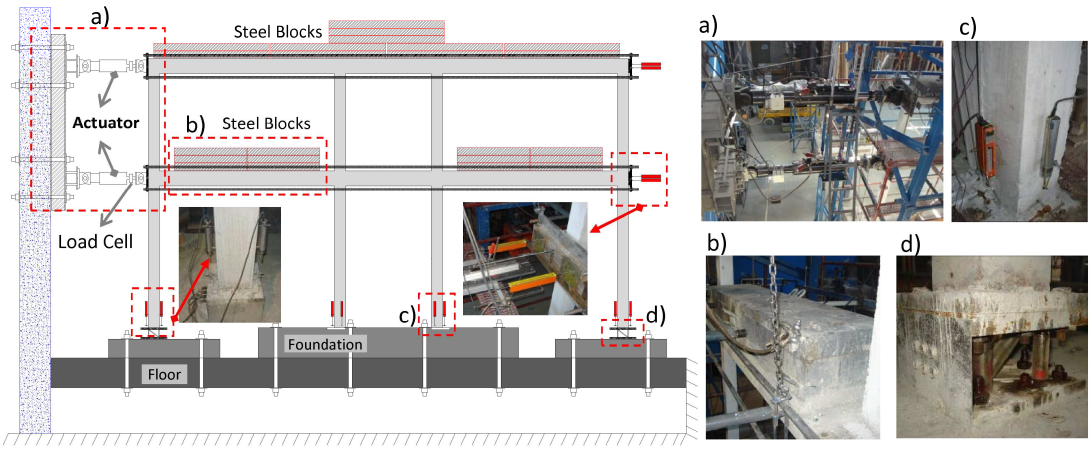


Figure 3. Instrumentations and loading

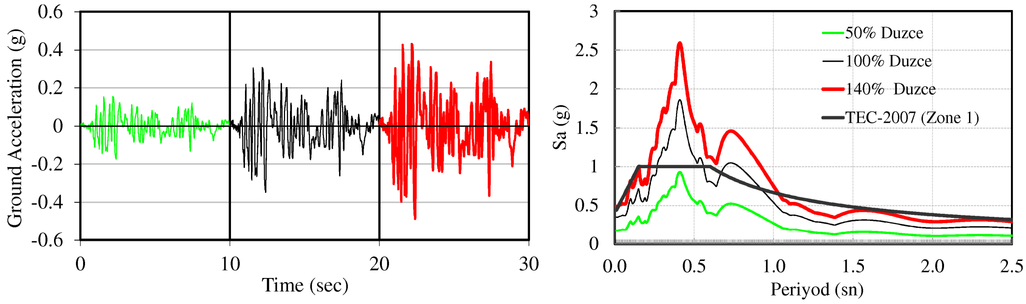
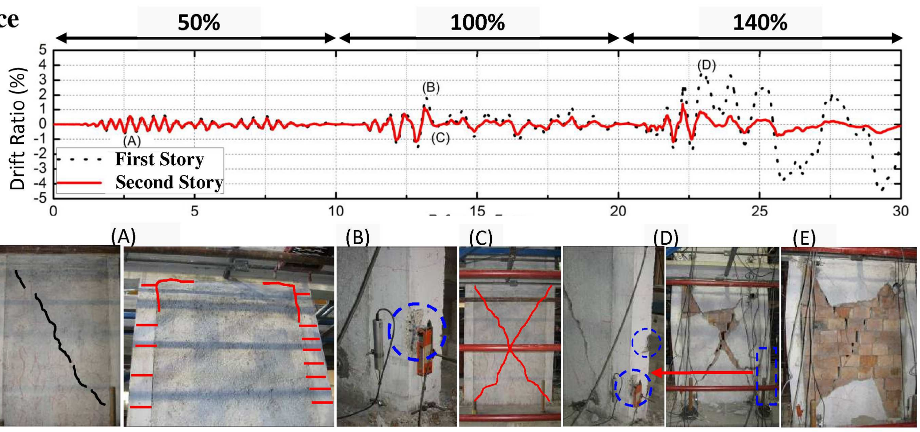


Figure 4. Ground acceleration and spectrum of the Duzce

4. TEST RESULTS

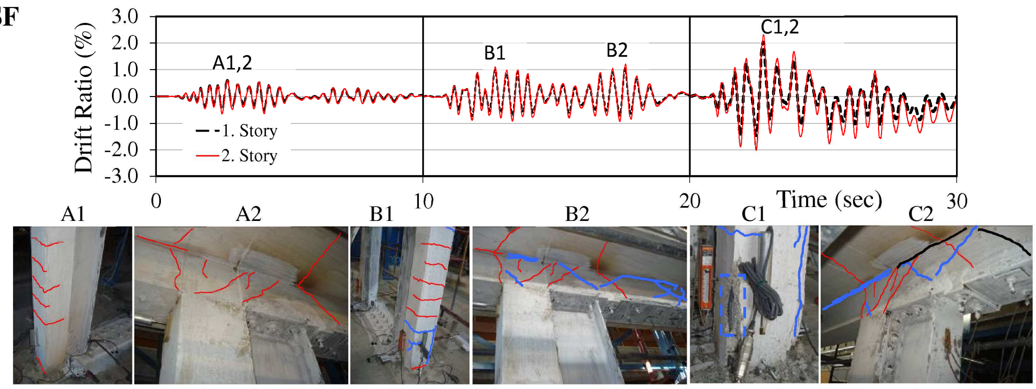
The lateral displacement demand in terms of inter-story drift ratio (IDR) vs. time of the PsD tests for the reference and ISF frame is given in Fig. 5. The IDR is defined as the ratio between relative story displacements to the story height. This figure also indicates the physical damage correlated with IDR of the test frames. Table 2 presents the base shear force and inter-story drift ratio (IDR) of the test frame. In addition, plastic rotation demands at the bottom of the first story columns are presented in Table 3. These plastic rotations were calculated from the measurements taken from the bottom of the columns. Fig. 6 presents the envelope response of the both frames. Fig. 7 shows the maximum inter story drift demands vs story height of the test frames. The performance limits namely immediate occupancy (IO), life safety (LS) and collapse prevention (CP) in terms of inter story drifts given in the TEC 2007 was also indicated in this figure.

a) Reference Frame



A: Initial cracks, B: Spalling of concrete, Bar buckling, C: Diagonal cracking, D: Bar buckling, E: Final stage

b) ISF



A1: cracks on the interior columns, A2: cracks on the joint, B1: cracks on the interior columns, B2: cracks on the joint, C1: Bar buckling, C2: cracks on the joint

Figure 5. Physical damage observed during the tests for the reference and ISF retrofitted frame

Table 2. Test results of the reference frame and ISF

Ground Motion	Base Shear Force (kN)		Maximum Inter-story Drift Ratio (%)			
			1. Story		2. Story	
	Reference	ISF	Reference	ISF	Reference	ISF
50% Duzce	60.4	67.6	0.7	0.6	0.6	0.7
100% Duzce	67.9	88.2	1.8	1.1	1.1	1.2
140% Duzce	54.5	116.6	4.5	2.0	1.4	2.3

Table 3. Column demands of the reference frame and ISF

Ground Motion	Column Plastic Rotation Demands							
	Column C1		Column C2		Column C3		Column C4	
	plastic rotation(θ_p) / curvature ductility (μ_ϕ)		plastic rotation(θ_p) / curvature ductility (μ_ϕ)		plastic rotation(θ_p) / curvature ductility (μ_ϕ)		plastic rotation(θ_p) / curvature ductility (μ_ϕ)	
	Reference	ISF	Reference	ISF	Reference	ISF	Reference	ISF
50% Düzce	0 / 0.3	0 / 0	0.003 / 1.9	0 / 0	0.001 / 1.5	0 / 0	0 / 0.6	0 / 0
100% Düzce	0.004 / 2.0	0 / 0	0.006 / 2.8	0 / 0	0.008 / 3.5	0 / 0	0.006 / 2.6	0 / 0
140% Düzce	0.038 / 9.4	0.001 / 1.4	0.055 / 16.9	0.001 / 1.2	0.025 / 8.3	0.004 / 2.2	0.036 / 11.5	0.003 / 2.0

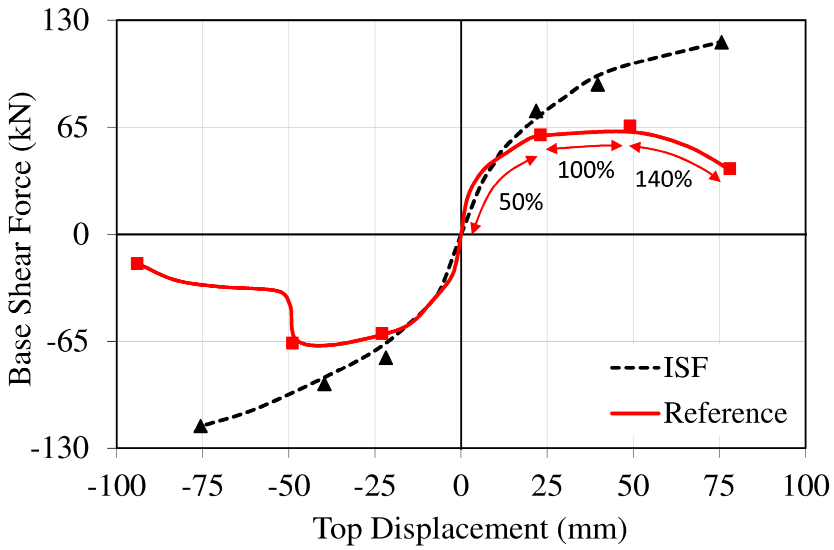


Figure 6. Envelope Response of the test frames

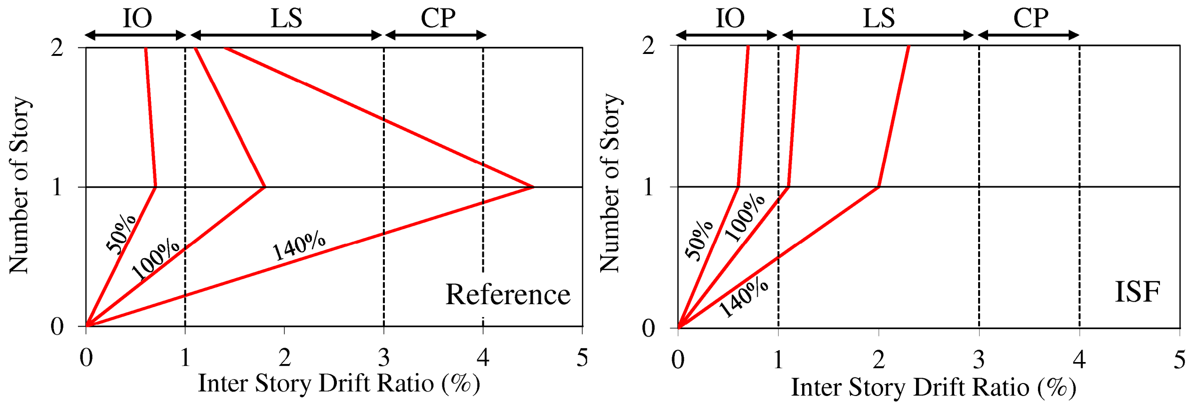


Figure 7. Maximum inter story drift demands vs number of story for the test frames

4.1. Test Results of the Reference Frame

The reference frame test results were presented in detail previously (Kurt, 2010; Kurt et al., 2011), hence a brief summary of results will be provided herein to serve as a basis of comparison with the frame strengthened with ISF. The maximum base shear forces of the reference frame as given in Table 2 were 60.4 kN for 50 % Duzce, 67.9 kN for 100 % Duzce, 54.5 kN for 140 % Duzce test. The maximum IDRs of the first and second story were 0.7 and 0.6% for 50 % Duzce, 1.8 and 1.1% for 100 % Duzce, 4.5 and 1.4% for 140 % Duzce test. During the 50% Duzce test, minor damages such as cracks at maximum moment regions of the first story columns, at the infill wall-frame boundaries and along diagonals occurred. The concrete crushing at the base of interior columns, longitudinal bar buckling and diagonal cracking along the diagonals of the first story infill wall were observed during the 100% Duzce test. At the 140% Duzce test, the maximum IDR of the first and second story was 4.5% and 1.8%, respectively. Hence, as seen in Fig. 7, a soft story mechanism occurred. The lateral strength of the reference frame dropped significantly due to damage of the infill wall.

4.2. Test Results of the ISF Retrofitted Frame

The maximum base shear of the ISF was 67.6 kN for 50% Duzce, 88.2 kN for 100% Duzce, 116.6 kN for 140% Duzce test. The IDRs of the first and second story was 0.6 and 0.7% for 50% Duzce, 1.1 and 1.2% for 100% Duzce, 2.0 and 2.3% for 140% Duzce test. At 50% Duzce test, flexure cracks were observed throughout the column height. In addition, minor inclined cracks were observed at the beam-column joint. Cracks in the concrete of the composite columns developed during each lateral loading excursion that produced tension in the concrete side of the composite column sections. It was observed that there was no yielding measured at the bottom of the columns. The base shear capacity did not drop and a nearly elastic behaviour was observed during this test. At 100% Duzce test, further cracks occurred throughout the beam span. Although there was no yielding monitored from the strain measurements at the bottom of the first story columns (from the steel column), the first story composite beam (from the steel plate) had limited inelastic behaviour. In spite of the damage mentioned above, there was no drop in the base shear capacity of the ISF retrofitted frame. At 140% Duzce test, longitudinal bar buckling at the bottom of the column C3 was observed. It progressed between 2 and 3 seconds of the ground motion. In additional to yielding of steel plate at the bottom and top of the first story composite beam, steel column member (IPE200) had sustained deformations and plastic hinges occurred at the bottom of first story columns. Although such a severe damage occurred, the base shear capacity of ISF did not decrease (Table 2 and Fig. 6). There was no anchorage rod failure in any of the tests. The base plate under the IPE200 steel column anchored to the RC foundation had uplift during the PsD tests. This shows that the ISF application may need special attention for the foundation due to concentration of the lateral demand to the only one bay. Hence, likewise many shear wall applications (Aguilar et al., 1989); the foundation retrofitting may be required for the existing structures while retrofitting with ISF. Consequently, the ISF application may be seen as a complementary alternative to other retrofitting solutions within a hybrid retrofit approach (Foutch et al., 1989).

5. COMPARISON OF REFERENCE AND ISF RETROFITTED FRAME

Results summarized in Table 2 show that base shear capacity of the ISF retrofitted specimen was about 2 times that of the reference frame. The maximum IDR at the end of 140% Duzce test was about 4.5% and 2.3% for the reference and ISF retrofitted frame, respectively. The lateral stiffness defined as the ratio between yield strength and top displacement was 4 and 2.3 kN/mm for the reference and ISF retrofitted frames, respectively (Fig. 6). The lateral stiffness of the reference frame was 1.7 times that of ISF due to contribution of the infill wall. It can be observed that the reference frame exhibited some deformability followed by sudden loss of strength due to infill wall failure. On the other hand, the ISF retrofitted frame exhibited nearly elastic behaviour up to 140% Duzce test. Table 3 compares the plastic rotation and curvature ductility values measured in the experiments. It was observed that plastic rotation at column bases occurred on all scale levels for the reference frame. Column base plastic rotations, however, were apparent during 140% Duzce tests for the retrofitted frame. This is an indication of ISF relieving the demand on columns while effectively controlling lateral deformations. Except spalling of the concrete at the composite columns, there was no brittle failure that would result in strength degradation of the elements or the frame. Hence it can be concluded that the main premise of ISF retrofits is the strength and ductility increase and drift control. These benefits can certainly be useful in retrofitting low rise deficient RC frame structures. Furthermore, based on the global drift limits suggested by the TEC 2007, the reference frame was beyond the collapse prevention performance level and this show that the original frame need an effective retrofit to sustain a sufficient seismic performance under seismic attacks. In fact, this result was clearly indicated by the damage observed during the PsD test of the reference frame (Fig. 5). Upon retrofitting, the ISF frame was within the life safety performance level in terms of global drifts defined in the TEC 2007 (Fig. 7).

6. CONCLUSION

The seismic performance of a deficient two story-three bay RC frame was investigated with and without ISF retrofit. Following conclusions can be drawn based on the observed response of test specimens:

1. The RC test frame with low concrete strength, no joint reinforcement and insufficient confining steel reinforcement in columns with 90 degree hooks was successfully upgraded by using an ISF retrofitting technique.
2. ISF increased the lateral load and energy dissipation capacity and deformability significantly. Hence, it can be stated that these systems are possible candidates for retrofitting of deficient RC frame buildings.
3. The connection details for the composite members were successful in order to accommodate plastic deformation demands at the critical locations (at the beams and base of the columns) without significant damage.
4. The reference frame was beyond the collapse prevention performance level during the 140% Duzce motion. The performance level of the ISF frame, on the other hand, was within the life safety limit in terms of global drift limits given in TEC 2007.

AKCNOWLEDGEMENT

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 greatly appreciated.

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