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A CASE STUDY OF STRENGTHENING OF DEFICIENT RC BUILDING WITH INTERNAL STEEL FRAME

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Abstract. *This study examines the strengthening of existing deficient reinforced concrete (RC) building by using internal steel frame (ISF). Test results indicated that ISF increased the lateral strength, ductility and energy dissipation capacity of the deficient RC building significantly. The test results were compared with simulations to observe performance levels. A case study building was analyzed to demonstrate the performance of an ISF retrofitted deficient RC building by using an analytical model calibrated with test results. The analyzed building was located at Marmara region in Turkey where the region most susceptible to severe earthquakes. The performance based evaluation with respect to Turkish Earthquake Code indicated that this building should be strengthened under Duzce ground motion demand. After strengthening by using ISF, the building was within the life safety performance level which is the performance level that needs to be satisfied for residential buildings. The modeling strategy and construction details of the ISF are also presented in this study.*

1 INTRODUCTION

Poor performance of reinforced concrete (RC) buildings was demonstrated dramatically in recent earthquakes (Northridge 1994, Kobe 1994, Kocaeli 1999, Taiwan 2003, India 2001) due to insufficient lateral load resisting system. Common deficiencies of RC buildings 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. To reduce the effect of these deficiencies on existing structures, seismic retrofitting techniques should be developed. Furthermore, application of the seismic retrofitting techniques should be encouraged to be used by the authority and people live in high seismic regions.

There are many strengthened techniques namely adding structural walls [1-4], steel braces [5-13], FRP diagonal braces integrated in the infill walls [14-16], precast-shear walls that fit perfectly into the existing frame [17], steel frames attached externally to the perimeter of the existing frame [17], and steel frames attached within the frame without using anchors [18]. Although the most commonly used strengthened technique is adding structural wall, this technique requires interrupting building use for a substantial period of time and may conflict with architectural requirements.

The literature review indicated that there is urgent need to develop a rapid, safe and practical strengthening technique. The internal steel frames (ISFs) which are one of the candidate retrofitting techniques are installed within bays of the deficient RC frames. The ISF is intended to easily accommodate wall openings for architectural requirements. In this study, firstly an experimental test frame with and without ISF was examined and then the analytical study was conducted to calibrate test frame with ISF. By using analytical results, the performance level of the frame with ISF was performed with respect to Turkish Earthquake Code (TEC 2007) [19]. Based on calibration of the test frame with ISF, an existing five story residential building was strengthened with ISF and the performance of such building was evaluated by using procedure suggested by TEC 2007 [19]. Duzce earthquake record was utilized in the nonlinear time history analysis to determine the performance points of the building before and after retrofitting. This study also explains the installation procedure of the ISF to the RC frame.

2 TEST FRAME AND ISF INSTALLATION PRODECURE

Two, one bay-by-one story, portal RC frames were examined to determine cyclic performance of the ISF. First test specimen SP1 was reference frame tested without any retrofitting while second specimen SP2 was tested after implementing ISF in the RC frame. Although comprehensive information about the test frames is available in elsewhere [20], a summary was introduced in this study. As shown in Figure 1, the center-to-center span length was 1400 mm and the column height was 1000 mm. The dimensions of the columns were 100 mm × 150 mm with four 8-mm diameter longitudinal reinforcement plain bars resulting in about 1.33 % longitudinal reinforcement ratio. 4 mm diameter plain bars were used for stirrups. TEC 2007 requires stirrups to be anchor using 135 degree hooks however 90 degree hooks were used for all columns and beam to simulate the detailing deficiency of the Turkish construction practice before the establishment of the modern seismic codes. To simulate the insufficient confinement details of the columns, the stirrup spacing of the columns was equal to the smaller dimension of the column section (100 mm). The 100 mm × 150 mm beam was cast with a 450-mm wide, 55-mm thick slab. A 70-mm transverse reinforcement spacing was used for the beams. The RC beam-column joint had only one column stirrup extending into the joint. The yield strength of the 4-mm and 8-mm diameter reinforcement bars was determined as 270 and 330 MPa, respectively. The target 28-day cylinder compression strength was 8 MPa to simulate the existing deficient structures with low concrete strength determined in field investigations [21-23].

A constant gravity load of 62 kN was applied by placing steel blocks on the RC frame (see Figure 1(c)). This gravity resulted in the axial load ratio (i.e. ratio of gravity load to axial load carrying capacity) for the RC column was roughly 20% for all specimens. Cyclic lateral loading was introduced by controlling the drift ratio (DR) as in Figure 1 (d).

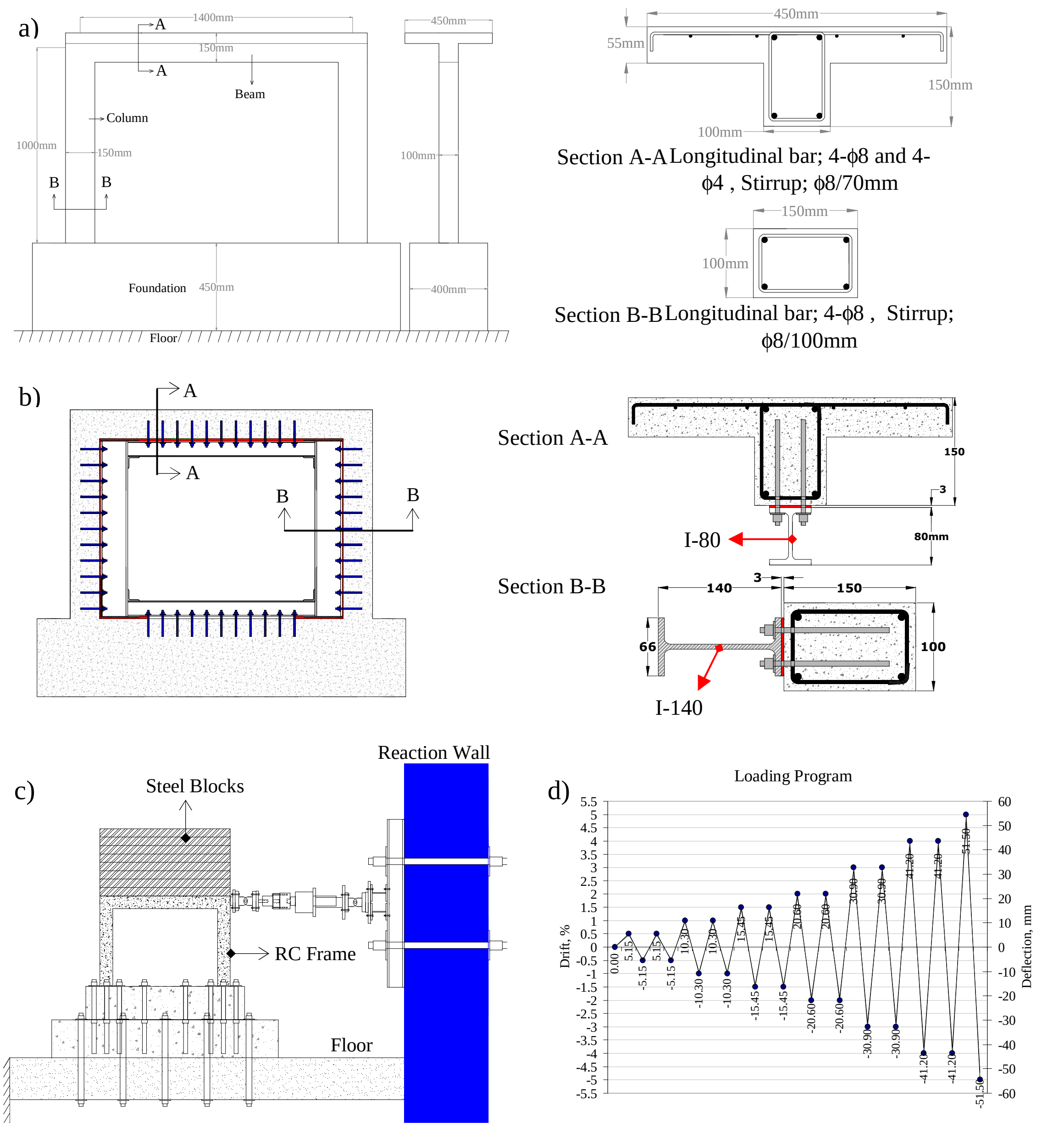


Figure 1: a) Analyzed building floor view, b) Analyzed 4-story frame, c) Beam and column section.

The ISF was composed of rigidly connected beams and columns. The ISFs were implemented after constant gravity load was applied on the RC frame to simulate actual retrofit conditions. The application procedure was as flows; firstly, anchor holes were drilled into the column face and bottom side of the RC beams. Than, these wholes were cleaned up by air blowing, brushing and air blowing (Figure 1 (b)). Next, epoxy primer was injected into these holes and the anchor rods were inserted and left for curing. In the second stage, a thin layer (about 3 mm) repair putty was applied on the RC member on all surfaces that contact the ISF (Figure 1). Before the epoxy cured, the anchor rods were tightened to fasten the individual steel members to the RC frame. Finally, the steel beams were welded to the steel column. The diameter of the anchor rods and holes were 6 mm and 8 mm, respectively. The anchors were embedded 120 mm into the RC beam and columns.

2.1 TEST RESULTS

Figure 2 shows the hysteretic response obtained from both specimens [20]. Specimens SP1 and SP2 developed a lateral stiffness (the peak positive and negative loading points during the first cycle (± 0.5% DR)) of 2.48 and 12.68 kN/mm, respectively. The lateral strength of the specimens SP1 and SP2 were 13.7 and 118.9 kN.

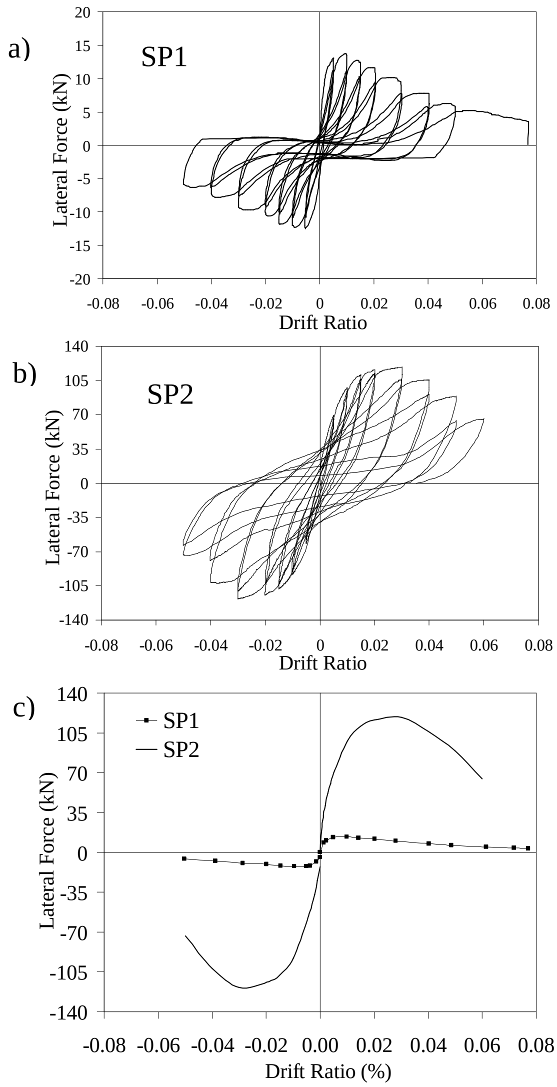


Figure 2: a) Cyclic response of the reference frame, b) Cyclic response of the strengthened frame, c) Envelope response of the frames.

Plastic hinges were first observed at the bases for the reference frame (specimen SP1). A plastic mechanism was formed at a drift ratio DR slightly higher than ± 2%. Upon further lateral displacements, pinching behavior and severe stiffness degradation was observed. Specimen SP2 was designed to develop composite action in the beam and two columns. Cracks in the concrete widened during each loading excursion that produced tension in the concrete portion of the composite section. The specimen SP2 failed due to fracture of the welded beam-to-column connection that initiated at ± 3 % DR. These test results indicated that the ISF retrofitting increased the lateral strength, stiffness and energy dissipation capacity of the deficient RC frame significantly.

2.2 ANALYTICAL STUDY OF THE TEST FRAME

The test frame, specimen SP2, was analyzed by using nonlinear static pushover procedure in order to determine plastic rotations at the ends of the beam and columns. Firstly, moment rotation relations were derived from sections indicated in Figure 1(b). The composite column and beam members had two moment curvature relation whether bending directions is positive or negative. When the composite section was under positive bending, the steel member (I-80 for beam and I140 for column) was under tension but the concrete was under compression. At the negative bending direction, this case was vise verse. Hence, two different moment curvature relations were developed for both composite column and beam. The moment curvature relation was converted into moment rotation relation. The plastic hinge length was assumed as half of the section height (TEC 2007).

Mander confined concrete model was used for columns and beam [24]. Longitudinal bar buckling was modeled by employing the backbone curve of Dhakal and Maekawa [25]. P-Δ effect was incorporated into the analytical model. SAP2000 [26] was used for the nonlinear static analysis. The analytical model was designed by using exact section dimensions and material properties (Figure 3).

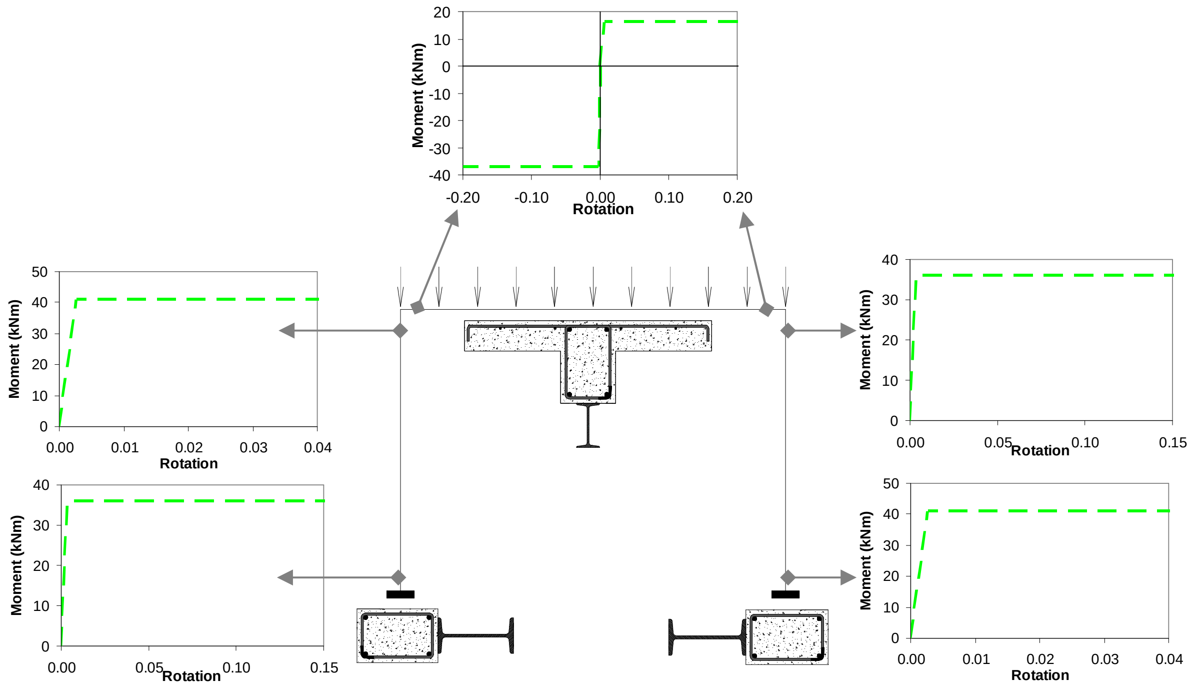


Figure 3: Analytical model of the specimen SP2.

In the TEC-2007, the moment rotation relation can be designed as elastic perfectly plastic behavior. Hence, after yielding, no hardening was used to model the moment rotation behavior (Figure 3). The performance of the member in the procedure suggested in TEC 2007 depends on the reinforcement bar and concrete strain (Figure 4). The following equations were defined for three performance levels in TEC-2007; immediate occupancy (IO), life safety (LS) and collapse prevention (CP) performance levels.

IO performance level;

$$(\varepsilon_{cu})_{IO} = 0.0035$$

$$(\varepsilon_s)_{IO} = 0.010$$

LS performance level;

$$(\varepsilon_{cg})_{LS} = 0.0035 + 0.01(\rho_s/\rho_{sm}) \leq 0.0135$$

$$(\varepsilon_s)_{IO} = 0.040$$

CP performance level;

$$(\varepsilon_{cg})_{CP} = 0.004 + 0.014(\rho_s/\rho_{sm}) \leq 0.018$$

$$(\varepsilon_s)_{CP} = 0.060$$

Where,
εcu; concrete strain at the top fiber
εcg; concrete strain at the top fiber of the confined concrete
εs; Reinforcement bar strain
ρs; available volumetric ratio of the stirrup of the member
ρsm; volumetric ratio of the stirrup of the member calculated by utilizing the TEC-2007

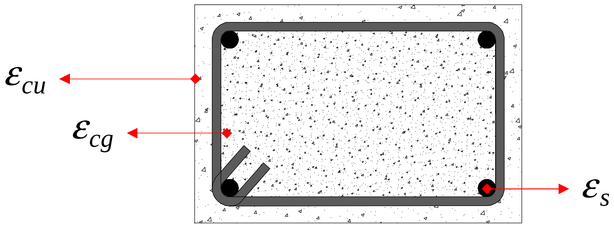
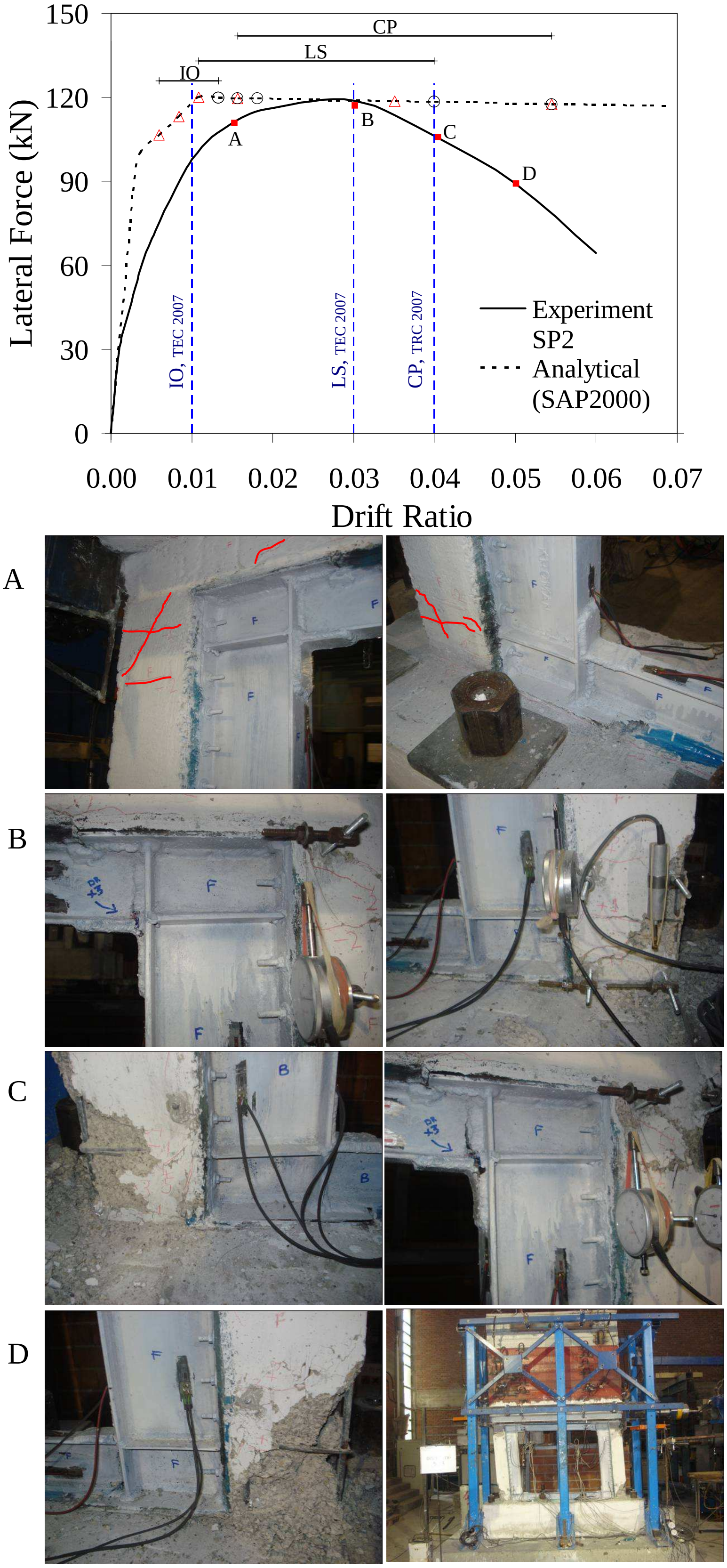


Figure 4: Strain at the cross section of the RC member.

Figure 5 indicates the static pushover curve of the test frame (specimen SP2). This figure also indicated the performance of the columns and beam in term of drift ratio. IO, LS and CP were determined by the method suggested in TEC 2007. Each performance levels indicated in Figure 5 exceeded the suggested strain limits at the bottom and top of the columns. Furthermore, at the end of each performance levels, the damage observed during the test are also seen in Figure 5. The interstory drift ratio (IDR) limits suggested in TEC 2007 are marked on this figure at 1, 3 and 4% DR. The average DR of the each three performance levels are about 1.0, 2.5 and 3.5 %, respectively.



A; Crack iniation, B; Fracture initiation of the steel beam, C; Concrete spalling at the bottom of the column, D; Base hinge mechanism and lateral displacement at 5% DR.

Figure 5: Analytical result of the test frame (specimen SP2).

3 CASE STUDY

In this section, the performance based design of existing five story RC building located in the Istanbul is presented. The building is a reinforced concrete frame structure with rigid shear wall surrounding the basement (Figure 6). Within this study, a performance

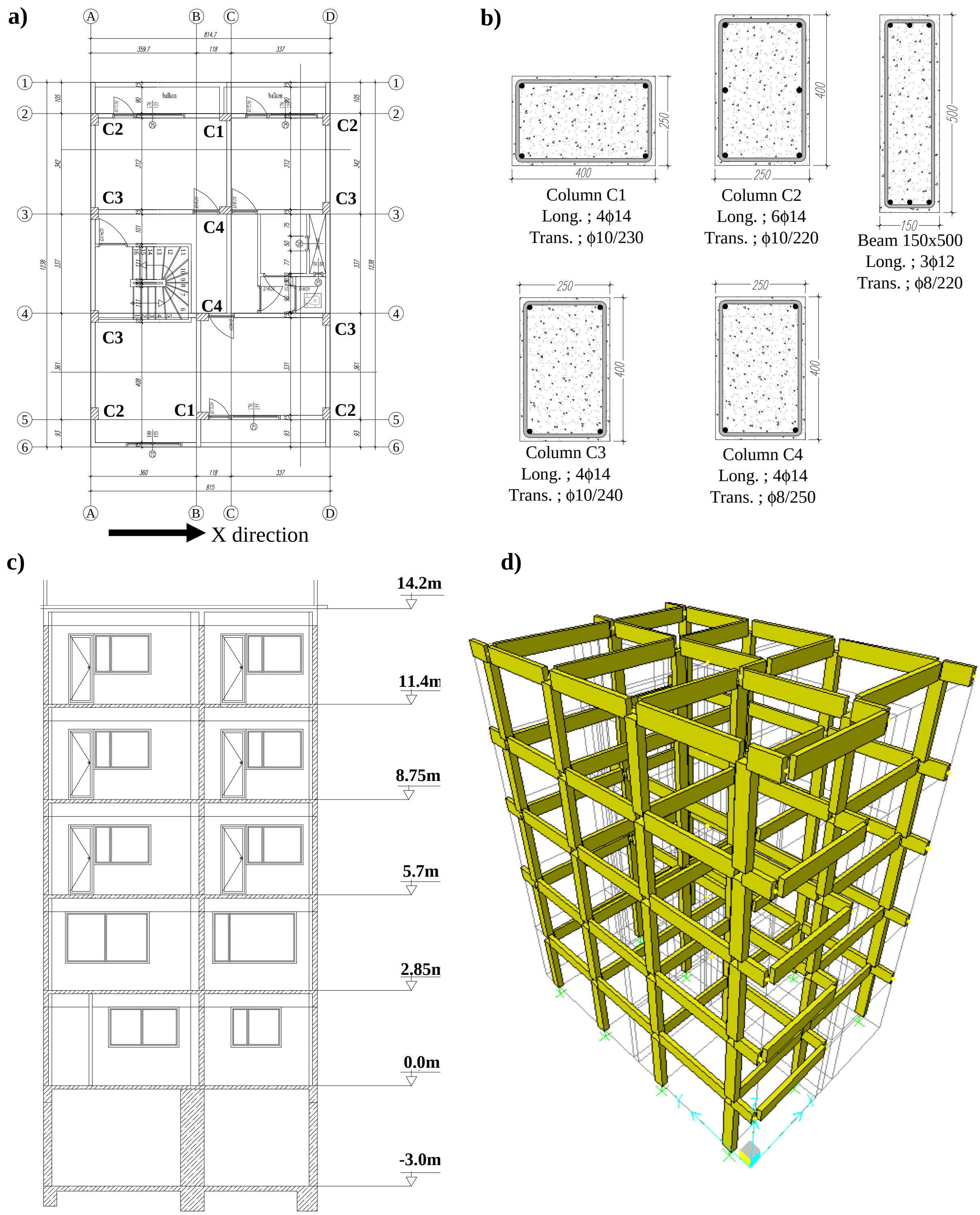


Figure 6: Five story building a) Plan view, b) column and beam dimensions, c) front view of the building, e) analytical model of the building (SAP2000).

evaluation method based on nonlinear pushover analysis is carried out using structural data. The strengthening of the building based on the methodology described previously from the test data is performed. Figure 6 indicates the plan view of the building. Uniaxial compressive strength and modulus of elasticity (calculated from [27]) are 8 MPa (close to test frame) and 13435 MPa, respectively. The yield strength of reinforcing steel was found as 220 MPa. The dimensions of the building in the x and y direction are 8.75 m and 12.23 m, respectively. The columns and beam dimensions are 250x400 and 150x500 mm, respectively. The orientation and also size of the beams and columns are shown in Figure 6. The stirrups spacing of the columns and beam are about 220 mm with a clear cover of 20 mm. It is important to mention that the stirrup spacing of columns and beams does not satisfy the current code TEC-2007 [19]. Furthermore, the in-situ concrete strength is lower than the code specified minimum. The steel grade of the longitudinal and transverse reinforcement is S220 whose yield strength is 220 MPa.

3.1 STRENGHENED BUILDING WITH ISF

The strengthened technique was applied to enhance the lateral load resisting capacity of the direction only. Figure 7 indicates the strengthened bays of the building. There were two strengthened cases namely ISF 1 and ISF 2. The difference between them is; the ISF2 had additional strengthened bays, axis 3-3 and axis 4-4, at the first and second story. The steel members to build composite columns and beams are I-400 and 13mm-thick-steel plate (Figure 7). The yield strength of the both steel members was taken as 235 MPa.

3.2 PERFORMANCE OF THE BUILDING WITH AND WITOUT ISF

Nonlinear static pushover analyses were conducted in order to estimate displacement capacity of the building for the required evaluation techniques. The 3D computer model of the building was generated using SAP2000 [26] from the original drawings of the building (Figure 6). All the joints on each floor were constrained in order to model the diaphragm effect. Moment–rotation properties derived from sectional analyses with the plastic hinge length (taken equal to half the member depth in the direction of loading as suggested by TEC 2007) idealization were assigned to the beam and column ends (similar to given moment rotation as seen in Figure 3). Axial force-moment yield surfaces obtained from interaction diagrams were used for column plastic hinge regions. Load distributions proportion to story mass and first mode amplitude were used for pushover analysis for x direction. Prior to conducting the pushover analyses, gravity loads and 30% of the live load on the structure were applied. The displacement-controlled pushover analysis was then performed to obtain performance point of the building and plastic deformations (rotations) of the members. After performing the pushover analysis and obtaining the capacity curve, the performance points of the building in x direction was calculated using method namely single degree of freedom (SDOF) approach employing the Duzce ground motion (DGM) (Figure 7). Pushover curve was converted into the acceleration displacement response spectrum (ADRS), Figure 8, by using Equation 1:

$$S_a = \frac{V_b}{\alpha_1 W} \text{ and } S_d = \frac{\Delta r}{\Gamma_1 \varphi_{r,1}} \tag{1}$$

where W is the total weight of the MDOF structure, Vb is the base shear, Δr is the roof displacement of the MDOF structure, $\alpha 1$ is the modal mass coefficient for the first mode (first fundamental mode), and $\Gamma 1$ is the modal participation factor for the first fundamental mode.

ϕr,1 is the amplitude of the first fundamental mode at the roof, Sa is spectral acceleration, and Sd is the spectral displacement.

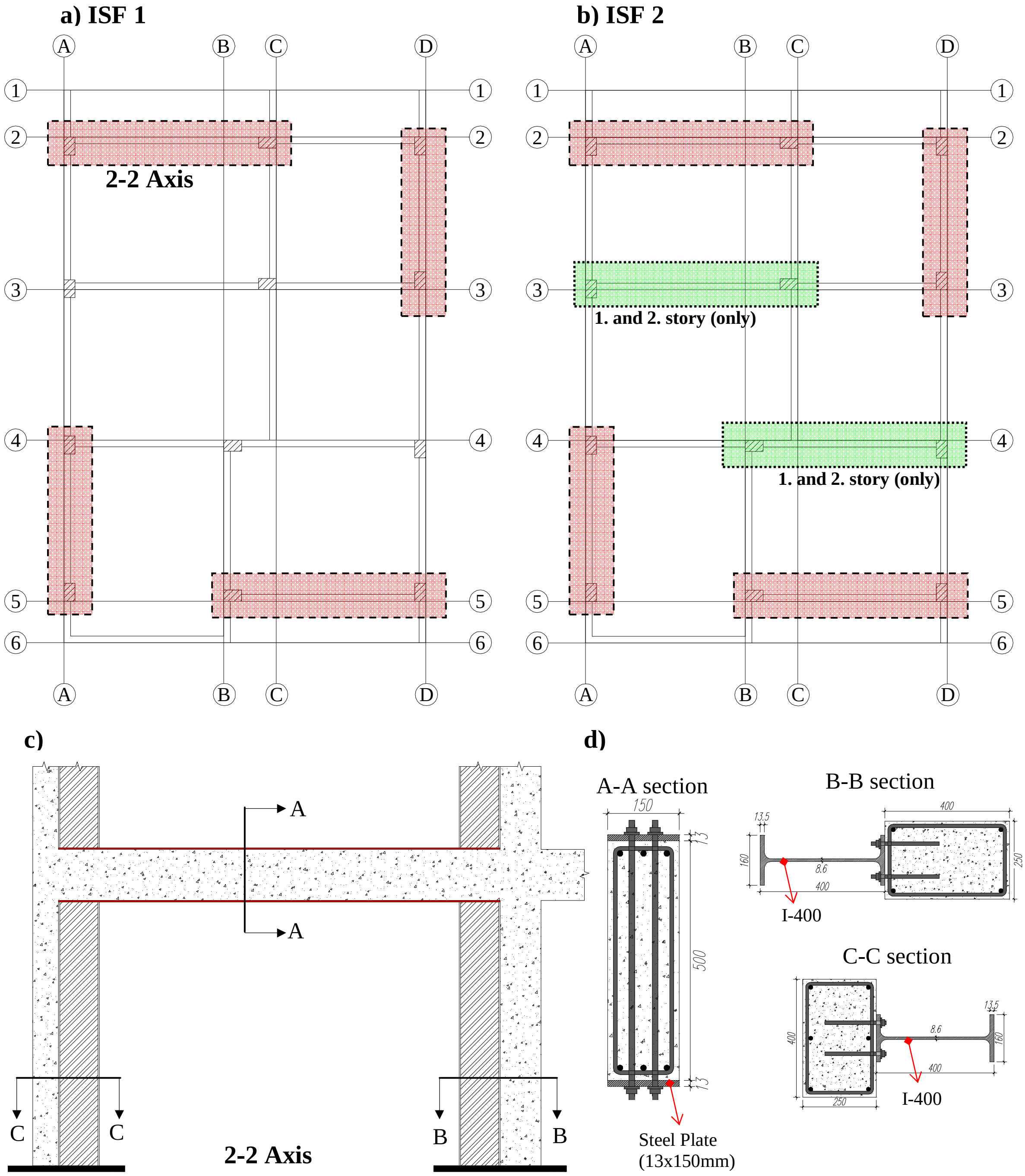


Figure 7: a) and b) Strengthened building with ISF1 and ISF2, c) frame view of the axis 2-2, d) Section of the composite members.

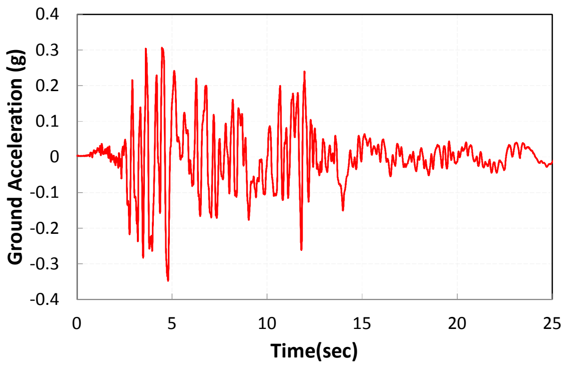


Figure 7: Duzce ground motion

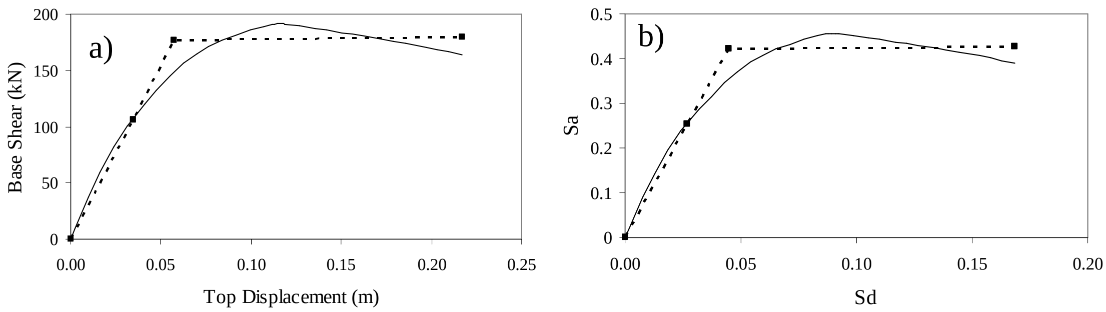


Figure 8: a) Pushover curve of the building, b) ADRS of the building.

For the SDOF approach, the linearization was performed based on the procedure given in FEMA 353 [29] (Figure 8). The mass of the building is taken as the mass corresponding to the governing x modes and 5% critical damping is assumed. Using the bilinear idealization with elastic unloading a SDOF analysis is conducted using the DGM to obtain the top displacement (performance point).

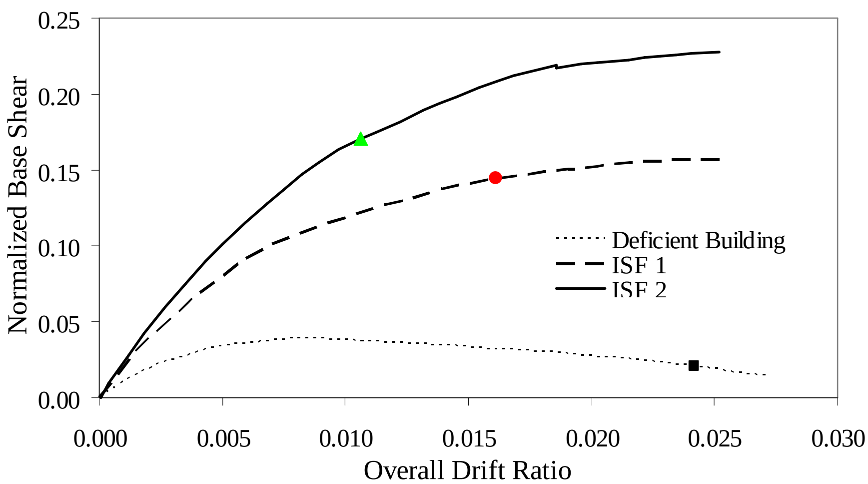


Figure 9: Pushover curve of the buildings, building with ISF1 and ISF2.

The performance points according to DGM are shown on the pushover curves in Figure 9. It can be observed that the building experiences an overall drift ratio of about 2.4 % in the x direction prior to retrofit.

A member by member evaluation is then performed to determine the damage level of the members. The number of columns and beams at different performance levels are presented in Table 1. This evaluation indicated that 100% and 36% of the first story columns and beams of the deficient building with any ISFs were at the total collapse (TC) performance level for x directions, respectively. This results indicates that this deficient building needs to be retrofitted.

Upon retrofitting, the number of the columns which were in the TC performance level decreased. Although 33% of the first story columns of the building implemented ISF 1 was within the TC performance level, this condition did not satisfy the performance level of the residential building suggested in TEC 2007. Finally, the desired performance level of the deficient building retrofitted with ISF2 was obtained by increasing numbers of retrofitted bays.

Column	1. story			2. story			3. story			4. story			5. story		
Performance Levels	Deficient	ISF 1	ISF 2	Deficient	ISF 1	ISF 2	Deficient	ISF 1	ISF 2	Deficient	ISF 1	ISF 2	Deficient	ISF 1	ISF 2
IO	0	4	10	1	11	12	5	12	12	12	12	12	12	12	11
LS	0	3	2	4	0	0	6	0	0	0	0	0	0	0	1
CP	0	1	0	3	0	0	1	0	0	0	0	0	0	0	0
Total Collapse	12	4	0	4	1	0	0	0	0	0	0	0	0	0	0

Beam	1. story			2. story			3. story			4. story			5. story		
Performance Levels	Deficient	ISF 1	ISF 2	Deficient	ISF 1	ISF 2	Deficient	ISF 1	ISF 2	Deficient	ISF 1	ISF 2	Deficient	ISF 1	ISF 2
IO	12	16	18	16	19	21	17	19	18	23	19	19	24	22	24
LS	2	5	4	1	4	3	7	5	6	1	5	5	0	2	0
CP	0	1	0	5	1	0	0	0	0	0	0	0	0	0	0
Total Collapse	8	0	0	2	0	0	0	0	0	0	0	0	0	0	0

Table 1: Performance levels of the members.

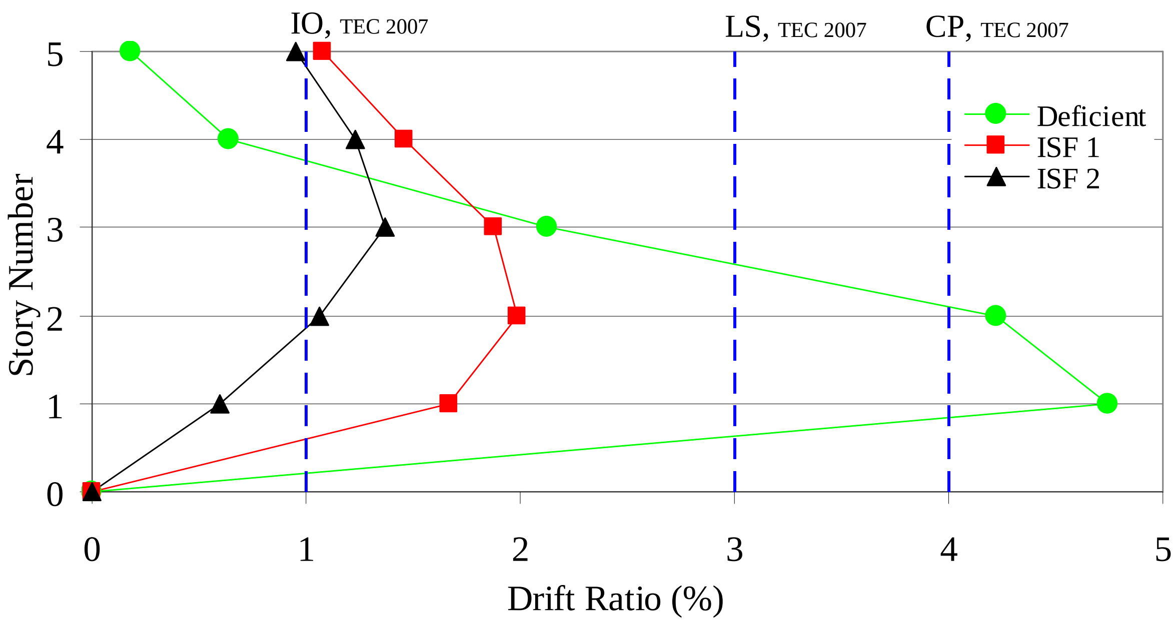


Figure 10: IDR of the building at the performance points.

The IDR profiles for x directions obtained from pushover analysis at performance points of DGM are shown in Figure 10. It can be observed that highest IDR, which was about 4.7 % in the x direction, occurred in the first story level of the building without any ISF retrofit. Upon retrofit the IDR reduced to about 1.4% for the x directions. It should be noted that the observed drift ratio is in good agreement with those limits proposed in the TEC 2007 (Figure 10). This result shows that the ISF retrofit scheme was successful in controlling drift deformations and reducing the demands in the columns. As a results, the ISF retrofit design presented above was found to be successful in LS performance level of the building by reducing the deformation demands on the RC columns and controlling the drift deformations. In addition, above results clearly indicates that a retrofit technique needs to increase lateral stiffness and strength aside from increasing global ductility capacity (if any member base retrofitting technique is not used) when ductility capacity of the existing columns and beams are insufficient.

4 CONCLUSION

The test results indicated that the ISF increased the lateral strength, stiffness and energy dissipation capacity of the deficient RC frames. The analytical study of the test frame simulated the behavior of the test frame successfully. The test frame load-deformation and damage observed during the test was correlated by the proposed performance levels suggested in TEC 2007. It was observed that the performance levels defined in TEC 2007 can be utilized conservatively to evaluate the frame retrofitted with ISF. Based on the data gained from the test and analytical results of the test frame, a case study consisted of a real existing residential building before and after ISF retrofit was evaluated by utilizing procedures suggested by the TEC 2007 under imposed DGM demand. It was observed that although seismically deficient five story RC building did not satisfy the performance levels of a residential building acceptance criterion with respect to TEC 20007, it was adequate after retrofitting with ISF. As a result, the ISF can be considered as a rapid, safe and practical retrofitting technique.

5 AKNOWLWEDGE

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