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 1088

COMPARISONS OF DIFFERENT RETROFIT TECHNIQUES WITH PSEUDO DYNAMIC TESTING

G. Ozcebe [1] B. Binici[2] O. Kurc[3] E. Kurt [4] and E.Canbay[2]

ABSTRACT

Seismic performance of two different retrofit methods was investigated by conducting pseudo dynamic testing of two story three bay specimens. The test frames have infill walls in the central bay and represents the seismic deficiencies including use of plain reinforcing bars, low strength concrete and insufficient confining steel. Two non-invasive and occupant friendly retrofit schemes, namely use of FRPs and precast concrete panels integrated on the infill walls were employed. Specimens were subjected to three different scale levels (0.5, 1.0, and 1.4) of Duzce ground motion. Control specimen experienced severe damage at 100% scale level and reached collapse stage due to the loss of integrity of the infill wall accompanied by heavy damage on the boundary columns of the infill wall. Experimental results confirm the lateral strength gain with 'some' ductility enhancement upon retrofitting. In this way, the test structure was able to survive the 1.4 scaled Duzce ground motion without heavy damage. Test results demonstrate the success of the two previously developed retrofit methods for simulated earthquake loads as long as sufficient drift control is ensured.

Introduction

The traditional approach of adding shear walls is the most commonly chosen alternative for current seismic retrofit applications in Turkey. However, the construction work involved for this retrofit scheme is extremely demanding. Furthermore, it results in lengthened retrofit time and necessitates relocating the occupants. In order to overcome these shortcomings, alternative retrofit schemes by utilizing the presence of substantial amount of infill walls can be quite efficient. In order to rely on these infill walls for collapse prevention during an earthquake, they need to be intervened. Two rehabilitation methods namely use of fiber reinforced polymer (FRP) and precast panels integrated on the infill walls were developed at Middle East Technical University (Baran 2005, Özcebe et. al. 2005, Binici et. al. 2007). In this way, it was aimed to provide a wider range of retrofit alternatives at the service of practicing engineers for seismic hazard mitigation studies. Later these retrofit methods were included as possible retrofitting techniques in the Turkish Earthquake Code (TEC 2007), perhaps in a slightly premature manner due to the fact that design

[1]Professor, Dept. of Civil Engineering, Middle East Technical University, 06531, Ankara, Turkey
[2]Associate Professor, Dept. of Civil Engineering, Middle East Technical University, 06531, Ankara, Turkey
[3]Assistant Professor, Dept. of Civil Engineering, Middle East Technical University, 06531, Ankara, Turkey
[4]Grad. Research Assistant, Dept. of Civil Engineering, Middle East Technical University, 06531, Ankara, Turkey

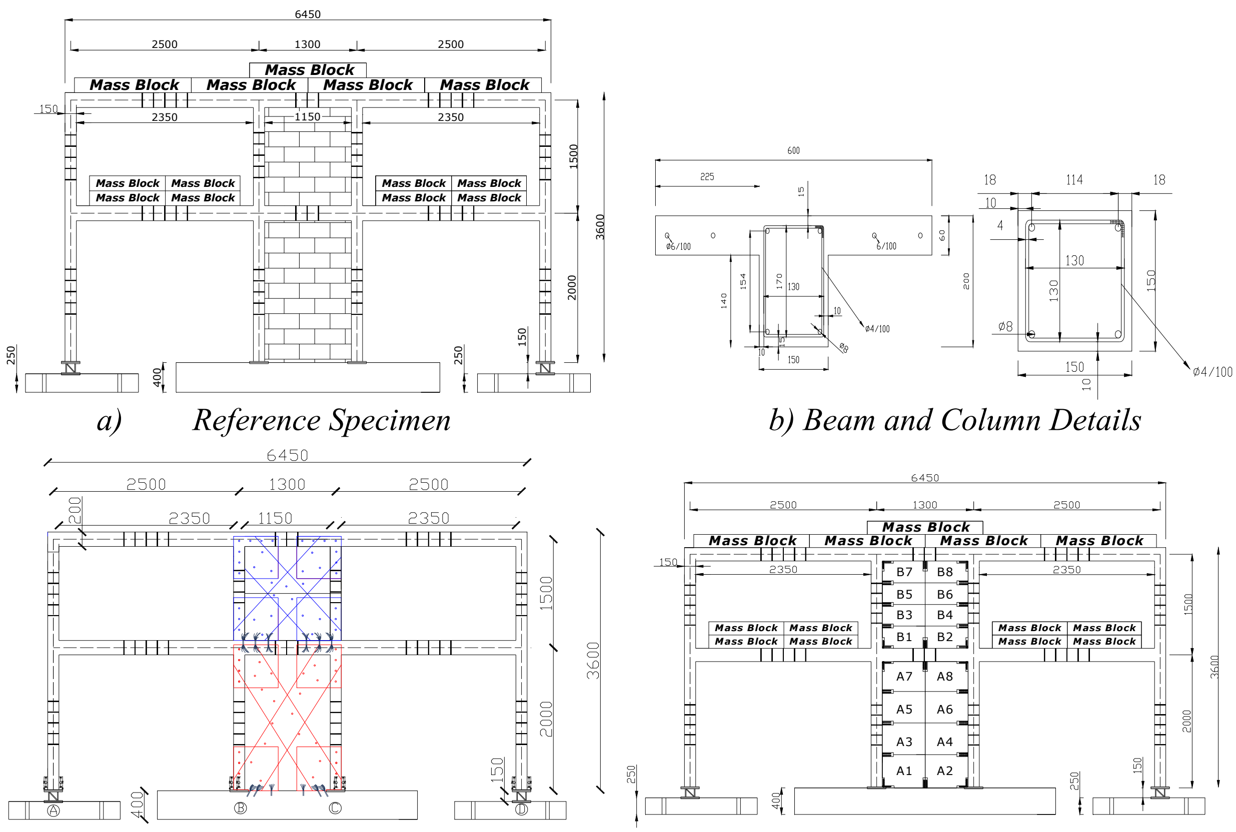
proposals were based solely on quasi-static cyclic testing. In this study we aim to explore the "true" seismic performance of these two retrofit methods by pseudo dynamic testing and critically examine their possible use.

Test Specimens

Reference test structure is a two story-one bay ½ scaled model of a typical frame of a low rise reinforced concrete building constructed in 1970s in Turkey. The central bay of the test frame has hollow clay brick infill (110mm × 130 mm × 130 mm with 65% void ratio and having a uniaxial compressive strength of 14 MPa) walls in both stories finished with mortar plaster having a uniaxial compressive strength of 12 MPa. Details of the test frame and experimental setup is presented in Fig. 1. Both for beams and columns, 4 - 8 mm diameter plain bars were used as longitudinal reinforcement resulting in about 1% reinforcement ratio. Plain bars had yield strength of 330MPa and an ultimate strength of 365 MPa. Plain bars with a diameter of 4 mm and yield strength of 290MPa were used as ties in all members with a spacing of 100mm and 90 degree hooks (Fig. 1). These member details along with low strength concrete uniaxial compressive strength (7MPa) to mimic the non-seismic details commonly encountered in Turkey.

The two retrofitted structures are identical to the reference frame with the only difference being the strengthening applied in the central bay on infill walls. Both strengthened specimens were designed to achieve a base shear capacity of about 1.3 times the capacity of the reference frame. For such a strength enhancement, the ductility and performance state of the retrofitted structures are aimed to be examined experimentally.

FRP retrofitting is employed by placing two CFRP (carbon fiber reinforced polymer) strips cut to 450 mm along the diagonal of the infill walls on both surfaces (Fig. 1). A custom made CFRP sheet having a uniaxial strength of 3,400 MPa with a modulus of elasticity of 230,000 MPa and a thickness of 0.16 mm was used in the design of the FRP diagonals. The integrity of the FRP and the infill wall-plaster composite was provided by using FRP dowels passing through the walls and fanned out on the surfaces. The CFRP diagonal braces are integrated to the boundary frame by CFRP anchors embedded in holes drilled through the foundation, columns and slabs. This retrofit method targets limiting the inter-story deformations and increasing the base shear capacity of the existing weak frame with the FRP tensile strength contribution. In addition, FRP helps in keeping the infill walls intact, hence delaying out of plane collapse possibility. For the second retrofitted test structure, eight precast concrete panels (CPs) with a thickness of 25 mm was bonded on the infill walls from one side (Fig. 1). Uniaxial compressive strength of the CPs was 40MPa on the test day. Panels had about 0.3% longitudinal mesh reinforcement in both directions for handling purposes. CP's were produced with shear keys with steel reinforcement (10 mm diameter) that were connected to the columns, beam and the foundation. Shear keys were filled with high strength mortar (uniaxial compressive strength of 50MPa) to provide force transfer between the panels and the boundary elements. The main advantage of using such a system (over in situ cast concrete shear wall) is the convenience of transporting lighter concrete elements and achieving the connections fairly easily without any disturbance to the occupants.



a) Reference Specimen b) Beam and Column Details

b) FRP Retrofitted Specimen c) CP Retrofitted Specimen

Figure 1. Test Specimens

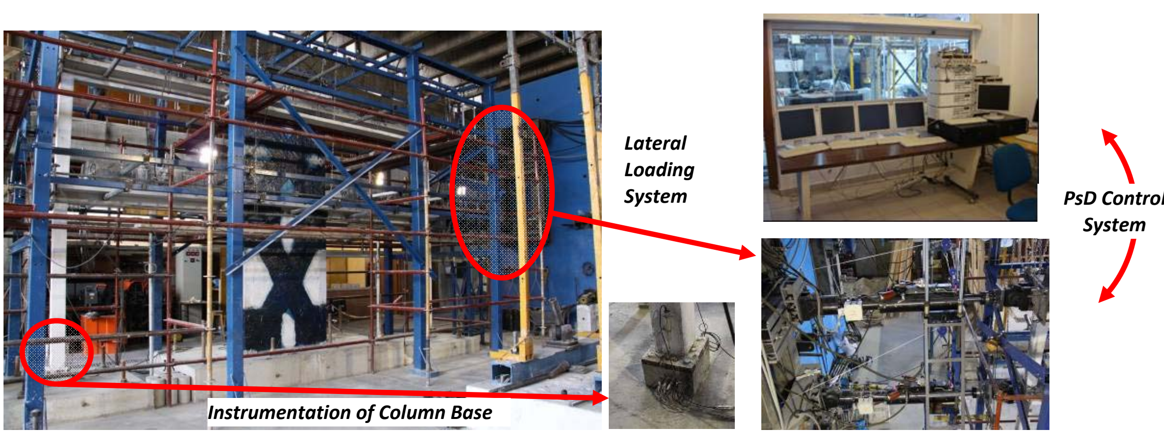
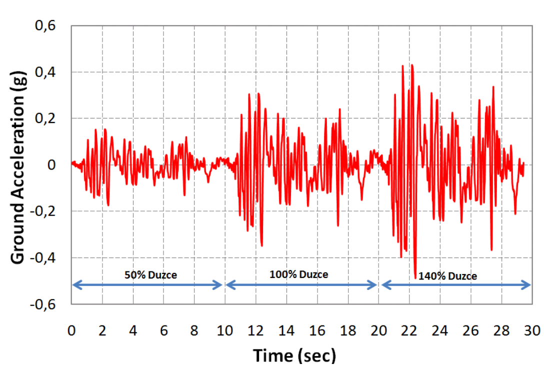
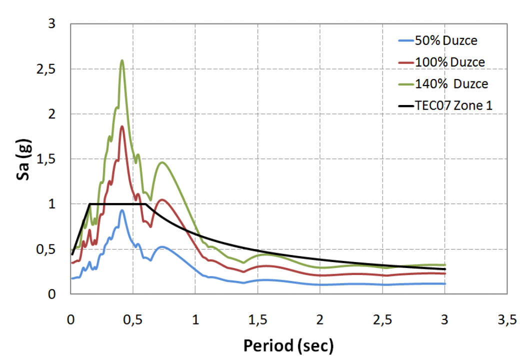


Figure 2. Experimental Setup

Experimental Program

Gravity load was simulated by using steel block weights (Figs. 1 and 2). Steel block locations were adjusted to obtain a gravity load ratio (i.e. axial load divided by the axial load carrying capacity) of 0.13 and 0.24 for exterior and interior columns, respectively. In addition to the story shear force and displacements, exterior column base internal forces and curvatures were monitored. A special 2-D load cell was manufactured, calibrated (Canbay et. al. 2004) and placed between the frame and foundation (Fig. 2). In this way, it was possible to obtain local member forces (Axial Force, Moment and Shear Force) in addition to local (curvatures) and global (base shear and inter-story drift ratio) deformation demands. PSD experiments were conducted using the continuous pseudo dynamic testing method (Molina and Verzeletti 1999). An explicit time integration scheme was employed for PsD tests. Integration process was executed continuously to eliminate possible relaxation errors. NS component of the 7.1 magnitude Duzce ground motion was employed for the tests. Tests were conducted about 1000 times slower compared to the real time motion, which was applied in three different increments (i.e. 50%, 100% and 140%) by scaling the acceleration record. Such a scaling was employed to investigate the response at three different hazard levels: a) 50% Duzce: Spectral acceleration (S_a) value for 50% Duzce is approximately similar to the base shear capacity ratio (base shear capacity divided by structure weight) of the bare frame at the structure's fundamental period. Hence, it is expected that structure will remain near or below yielding considering the presence of infill walls. It can be stated that this level should produce immediate occupancy compatible damage levels. b) 100% Duzce: Use of the actual Duzce ground motion recorded in 1999 Adapazarı earthquake can represent the hazard level realistically for less frequent events. c) 140% Duzce: This hazard level will correspond to a severe and rare earthquake and has approximately similar S_a value with the Turkish Earthquake Design Spectrum for Zone 1 on firm soil conditions at the pre-test estimated fundamental period of the structure. Acceleration- time series of the motion and acceleration spectrum of the motion is shown in Fig. 3. The original ground motion is compressed in time by a factor of 1/√2 to incorporate scale effects according to similitude law (Fig 3).





a) Acceleration-Time History

b) Spectrum of Scaled Ground Motions

Figure 3. Duzce Ground Motion

Experimental Results

The inter-story drift ratios (story drift deformation divided by story height) in time for the three different ground motion scales are shown in Fig. 4 for the three specimens. Base shear-top displacement responses of the specimens are shown in Fig. 5. Comparison of the backbone response of base shear versus average drift ratio (top displacement divided by structure height) is also presented in Fig. 5. The pictures of observed damage are presented in Fig. 6. Table 1 presents a summary of test results including both local and global demands observed for the different ground motion levels.

All specimens experienced minor damage for the 50% ground motion level. For the reference specimen, formation of interface cracks between the infill wall and the boundary frame elements was the most important observed damage. The maximum drift ratio achieved for the reference specimen was about 0.7% for this ground motion level whereas this value vas only about 0.1% for the retrofitted specimens. Base shear roof displacement plots show that ultimate base shear strength of the test frame was reached for the 50% ground motion. On the other hand base shear demand was only about ½ of the base shear capacity for the retrofitted specimens.

All specimens exhibited inelastic deformations for the 100% Duzce ground motion. The damage for the reference frame was significant diagonal cracking of the infill wall and initiation of cover spalling in column base plastic hinge regions. On the other hand, FRP retrofitted specimen exhibited lift off from the foundation due to the extension/slip of FRP dowels. CP retrofitted specimen had interface cracks along with some inclined cracks extending from the neighboring columns (Fig. 6c). Specimens were able to maintain the lateral load capacity at 100% scale. The maximum interstory drift ratio demand decreased from about 1.7% to about 0.8% and 0.5% for the FRP and CP retrofitted specimens, respectively. Maximum displacement ductility values calculated by dividing the maximum experienced displacement by the yield displacement obtained from the envelope curves are presented in Table 1. It can be observed that upon retrofitting, the ductility demand was reduced by about 30% at 100% scale level. Maximum strength enhancement for both retrofitted specimen was observed to be around 25%.

Final ground motion scale level (140%) brought the reference frame to a near collapse state. Plaster on the infill wall completely detached and the infill wall was vulnerable to out of plane collapse. Interior columns of the frame was severely damaged with visible longitudinal bar buckling (Fig. 6). A soft story mechanism was formed following the failure of the infill wall. This resulted in a 4.2% drift ratio of the first story, whereas the second story drift ratio was only about 1.3%. For the same ground motion FRP retrofitted specimen exhibited limited damage, the most important event being the pull-out of FRP dowels at the base. The infill wall remained intact and was not susceptible to out of plane failure. CP retrofitted specimen on the other hand experienced severe cracking at wall-column interfaces. The inclined cracks on the wall surfaces propagated from the columns to the wall base (Fig. 6). Both retrofitted specimens sustained maximum inter-story drift ratios of less than or equal to about 2%. The displacement ductility demand for the FRP retrofitted specimen was similar to that of the reference specimen, while the CP retrofitted specimen exhibited about 30% less displacement ductility demand. With the help of a drift control mechanism in the retrofitted specimens, column plastic rotation demands reduced by a factor of about 4; therefore limiting the damage on axial force resisting elements.

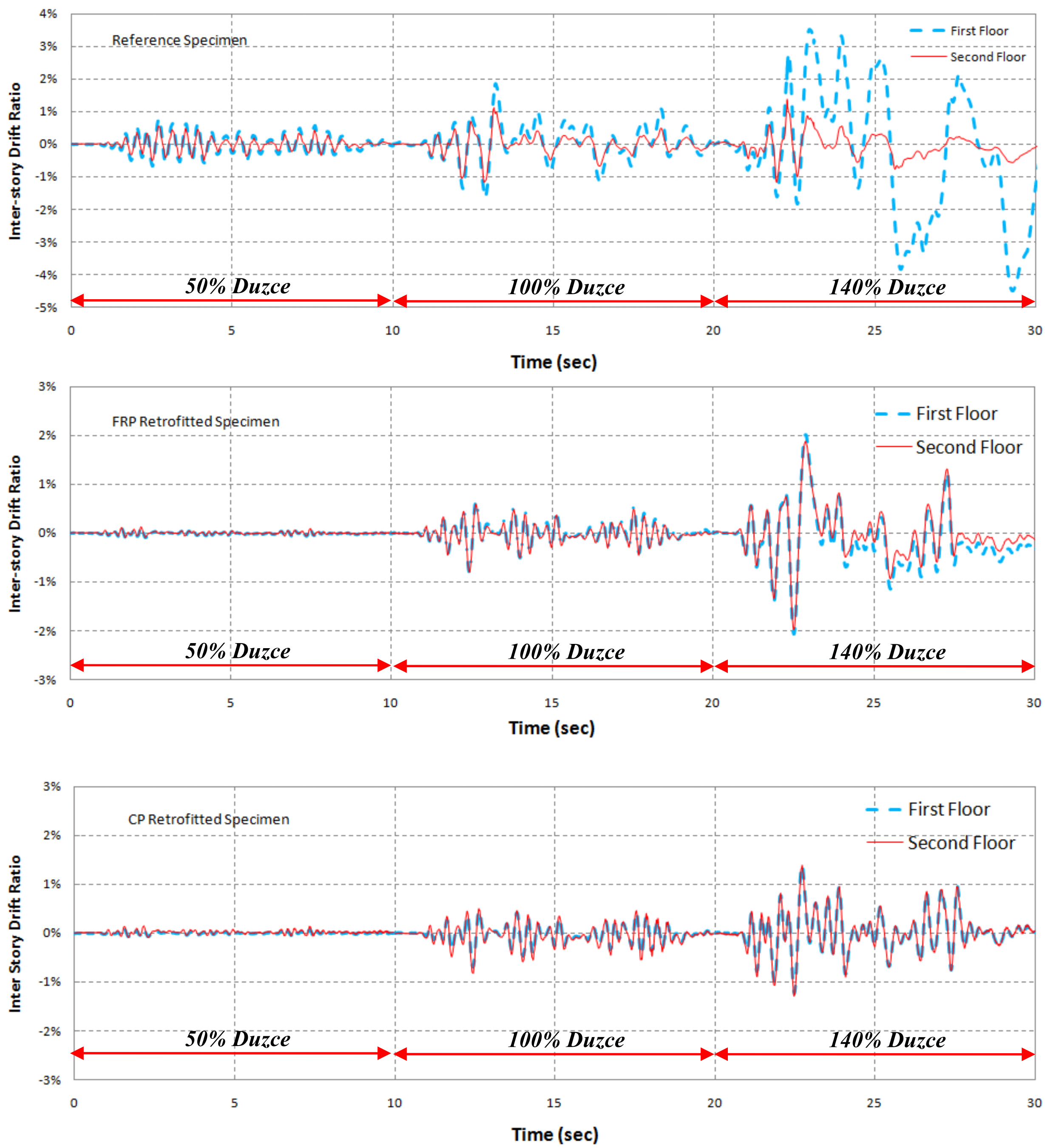
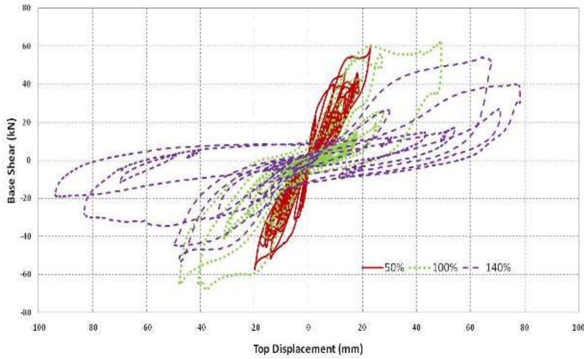
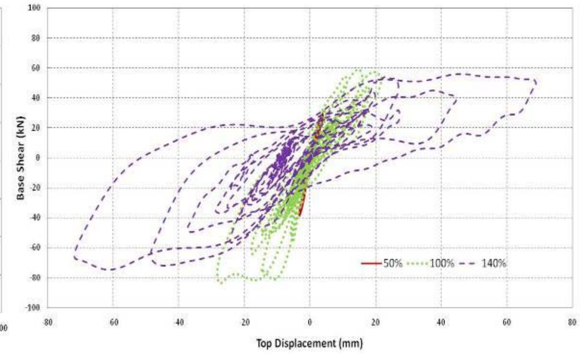


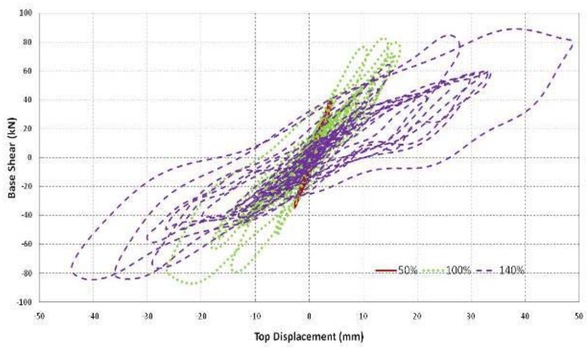
Figure 4. Drift Ratio Response of Test Specimens



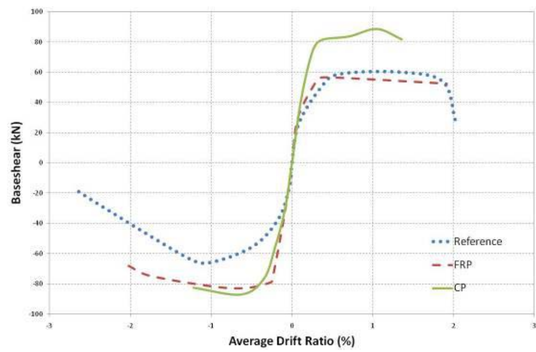
a) Reference Specimen



b) FRP Retrofitted Specimen



CP Retrofitted Specimen Response



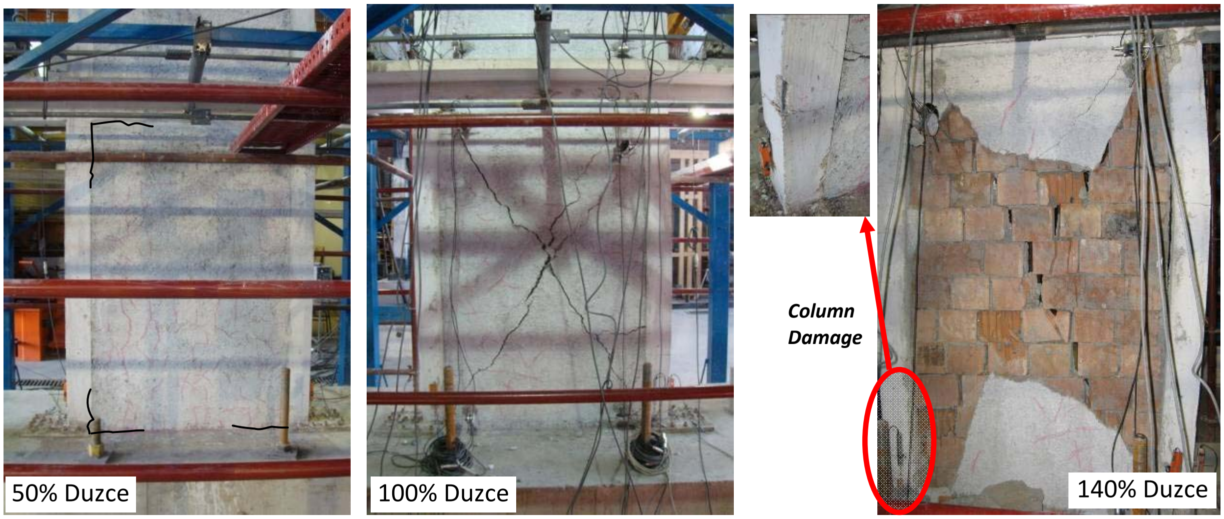
d) Comparisons of Envelope

Figure 5. Base Shear-Top Displacement Response

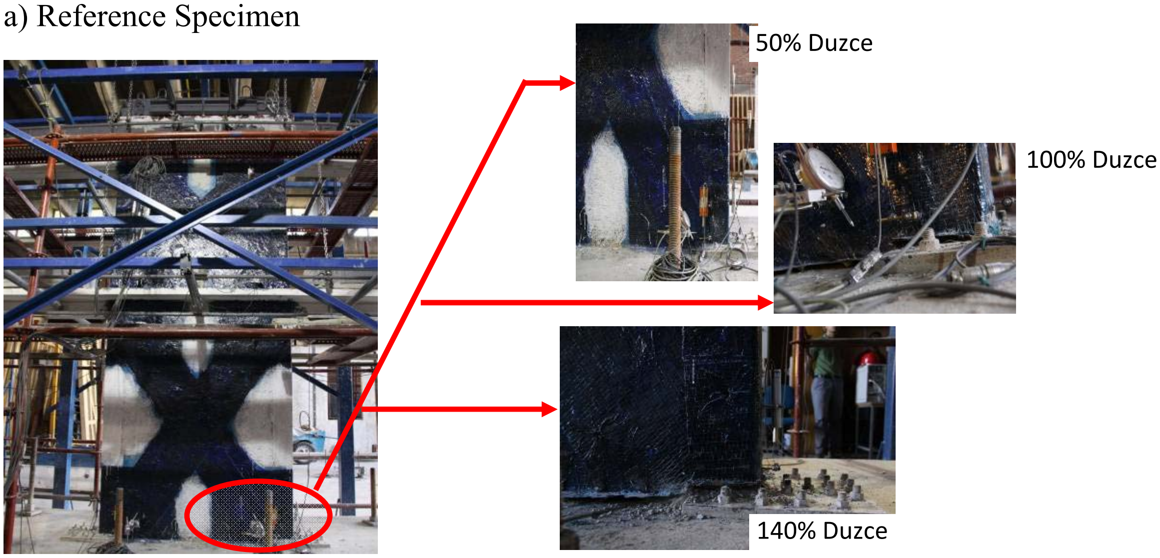
Table 1. Summary of Test Results

Specimen Name	Duzce Ground Motion Scale Level	Max Disp. Story 1 (mm)	Max Disp. Story 2 (mm)	Max. Interstory Drift % (Story 1)	Max. Interstory Drift % (Story 2)	Max. Base Shear (kN)	Max. Disp. Ductility Demand	Max.Column Base Plastic Rotation Demand (Exterior Column 1)	Max.Column Base Plastic Rotation Demand (Exterior Column 4)
Ref.	50%	15	23	0.74	0.51	60	1.8	0.009	0.009
	100%	35	49	1.72	1.05	68	4	0.012	0.014
	140%	85.3	93.8	4.20	1.30	54	7.8	0.040	0.047
FRP	50%	2.2	4	0.1	0.1	39	0.4*	0.008	0.008
	100%	16.3	28.2	0.8	0.8	84	2.9	0.009	0.009
	140%	42	71.8	2.1	2.0	75	7.8	0.020	0.018
CP	50%	1.9	4.1	0.1	0.1	40	0.4*	0.008	0.008
	100%	14.3	26.4	0.4	0.5	87	2.8	0.008	0.009
	140%	28.2	48.8	1.4	1.4	61	5.3	0.012	0.011

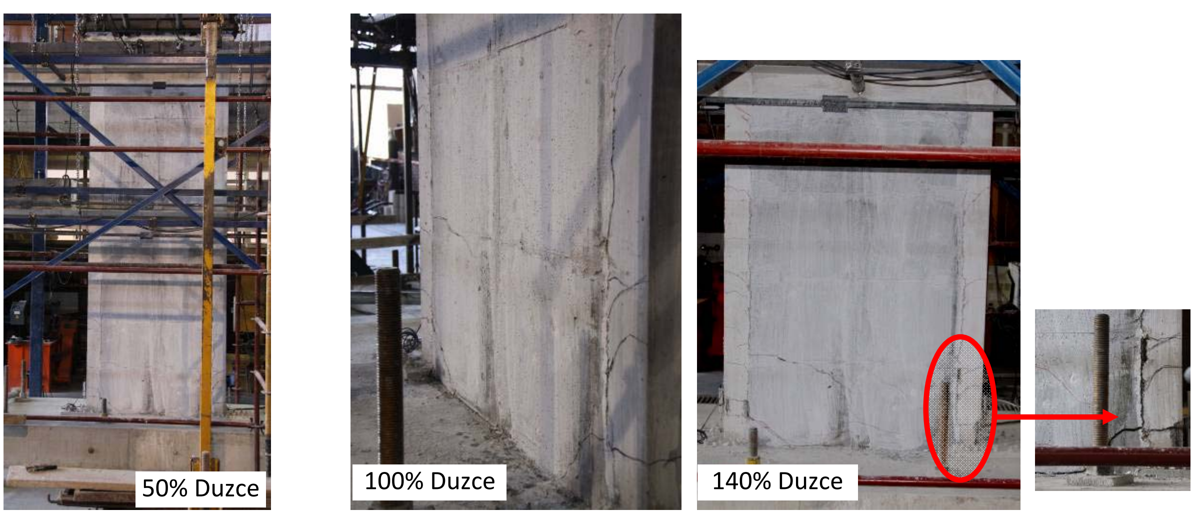
*: Estimated yield displacement is not reached.



a) Reference Specimen



b) FRP Retrofitted Specimen



c) CP Retrofitted Specimen

Figure 6. Specimen Damage

System Identification

Effective period values were obtained by employing the method of Molina amd Pegon (1999) from the measured displacements and restoring forces for a specified time interval. Equation of motion for the two degree of freedom system is solved for the secant stiffness and equivalent viscous damping values. Using the secant rigidity, fundamental period of the structure for the whole duration of the ground motion is calculated as shown in Fig. 7. It can be observed that initial period of the reference test frame (0.22 sec) elongated to about 0.5 seconds and 0.9 seconds at the end of 50% and 100% Duzce motions, respectively. Both retrofitted specimens exhibited a fundamental period of about 0.16 second, meaning they had approximately double the initial stiffness of the reference frame. Initial period of the retrofitted specimens elongated to about 0.18 seconds and 0.23 seconds at the end of 50% and 100% Duzce motions, respectively. This states that, retrofitted specimens had slower stiffness deteriorations and at the end of 100% Duzce motion their stiffness became similar to the stiffness of the reference frame at the undamaged state. In fact, this is an indicator of the successful drift control provided upon retrofitting. The fundamental period-time response for the 140% Duzce ground motion shows a lot of jagged pulses. This response, especially observed after the first 5 seconds of the motion, shows correlation with the reversed cyclic damage of the infill wall in terms of diagonal cracking (Fig. 6). Both retrofitted specimens exhibit a fundamental period of about 0.5 second on average for 140% Duzce ground motion. The damage control, visually observed in the retrofitted test specimens is reflected in Fig. 7 with the smoother and smaller period variations, especially for the CP retrofitted specimen.

Conclusions

The PsD test performance of FRP and concrete panel retrofitted test specimens along with a companion reference frame is presented for a ground motion demand at three scale levels. Observed damage levels in relation to local and global earthquake engineering demand parameters are discussed. The identification of the time series of restoring force quantities enabled uncovering the period change and its possible correlation with damage. Following important remarks requires emphasis:

- The three level scale Duzce ground motions (50%, 100% and 140%), resulted in approximately minimum damage, moderate damage, and severe damage states for the reference frame without any retrofit. Severe strength degradation and out of plane collapse susceptibility of the infill wall along with significant damage on the interior columns at the end of tests indicate the role of infill walls on deciding the final state of the structure.
- Both retrofitted schemes designed to produce comparable strength and stiffnesses behaved in an accepted manner. The CP application resulted in a reduced displacement ductility demand. FRP retrofitted specimen on the other hand was more beneficial in terms of keeping the wall intact and dissipating the energy through a rocking mechanism.
- FRP retrofitted specimen failed as a result of anchor slip whereas capacity of the CP retrofitted specimen was determined soon after significant diagonal cracking was observed at the lower third of the specimen. Both retrofitted specimens were observed to withstand a displacement ductility demand of at least 5 without any significant strength drop. Hence, it is believed they can safely be employed limiting the inter-story drift

demands below 2%.

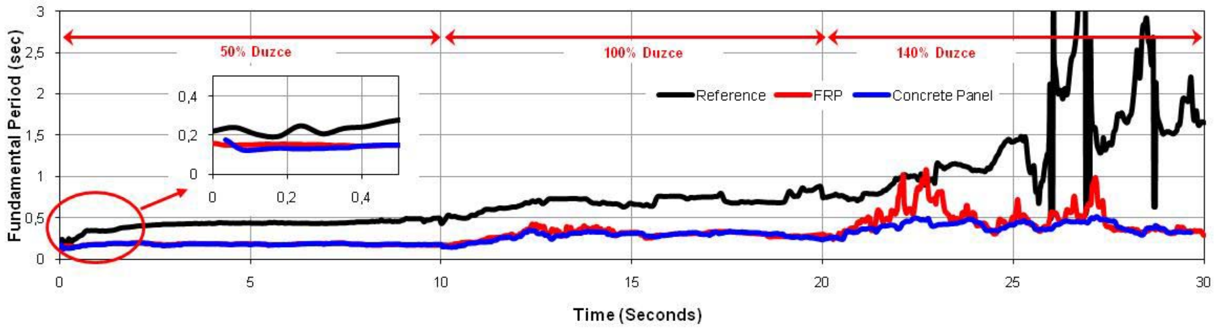


Figure 7. Specimen Damage

Acknowledgments

Funding provided by TUBITAK under grant number 106M451 is gratefully acknowledged.

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