SEISMIC RETROFITTING OF REINFORCED CONCRETE COLUMNS USING CARBON FIBER REINFORCED POLYMER (CFRP)

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ABSTRACT

Reinforced concrete columns with non-seismic detailing are often vulnerable to the effects of a major earthquake. In this study, the structural behaviour of undamaged and moderately damaged columns, which were retrofitted with carbon fiber reinforced polymers (CFRP) is investigated. The experimental program consists of four specimens having inadequate tie spacing, 90 degree hooks and plain reinforcing bars that were tested under lateral cyclic displacement excursions under a constant axial load of approximately 27% of the axial load carrying capacity. One reference specimen without any strengthening and one strengthened specimen with strengthening but without any pre-damage were tested. In addition, two similar specimens tested to moderate damage level of 2% drift ratio were repaired and retested. In one of the pre-damaged columns the repairing process was performed in the presence of constant axial load. The main parameters investigated in this study were the presence of axial load on the column during repairing, effect of pre-damage on ultimate displacements and the effect of CFRP wrapping on strengthened and repaired columns. The ultimate drift ratio for retrofitted columns increased to 6% which was only 3.5% for the reference column. Severe stiffness degradation was observed in both of the repaired columns due to the moderate level of pre-damage. For the repaired columns, the presence of the axial load during repairing did not significantly affect the ultimate drift ratio significantly however the enhancement in strength was not observed as compared to its companion specimen.

KEYWORDS

RC columns, axial load effect, CFRP, pre-damage, repair, strengthening.

INTRODUCTION

Poor performance of reinforced concrete columns during recent earthquakes in Turkey (Kocaeli 1999, Düzce 1999, Bingöl 2003) demonstrated the major deficiencies in building columns owing to low concrete strength, inadequate transverse reinforcement and poor structural detailing. Hence, convenient strengthening and repair strategies should be utilized in the light of new and suitable retrofitting materials. The traditional method of repairing or strengthening reinforced concrete columns by means of concrete or steel jacketing involves difficult implementation procedures especially in building columns. Consequently, new retrofitting methods for columns using CFRP should be investigated for repairing and strengthening processes that were found to be effective as concrete and steel jacketing in improving seismic response as stated by Seible et. al. (1997), Sheikh and Yau (2000), Iacobucci et. al. (2003), and Haroun et. al. (2005). By impeding premature bar buckling and providing confinement, CFRP retrofitted columns showed improved seismic resistance. The use of plain bars as main reinforcement was a common practice encountered in Turkey for columns built in structures in 1960’s and 1970’s which constitutes almost 60% of the current vulnerable building stock. Therefore there is a need to understand the behaviour of RC columns with and without any retrofit. So, the performance of FRP retrofitted columns require further examination due to seismic vulnerabilities commonly observed in Turkey include plain bars for longitudinal and transverse reinforcement, 90 degree hooks for stirrups and low concrete strength. Accordingly, a new experimental program investigating 1) the effect of CFRP wrapping on strengthened and repaired columns and 2) the effect of presence of axial load during repairing process was conducted in order to examine the seismic response of typical building columns.
EXPERIMENTAL PROGRAM

Test Specimens

The dimensions of the test specimens were $350 \times 350 \times 2000$ mm that were connected to a $1350 \times 500 \times 400$ mm stub. The details of the specimens are shown in Figure 1. The longitudinal reinforcement consisted of eight 22 mm diameter plain bars and the transverse reinforcement consisted of 10 mm diameter plain bars with a spacing of 200 mm with 90 degree hooks simulating typical deficient construction practice in Turkey. Table 1 presents the details of test specimens.

Concrete and Steel

All the specimens were cast with 3 batches of concrete having maximum 15 mm gravel size with a target compressive strength of 20 MPa. Concrete compressive strength values at the test day of the specimens are presented Table 1. The plain longitudinal bars and transverse reinforcement had 287 MPa yield strength ($f_y$) and 420 MPa ultimate strength ($f_u$) as obtained from tension test results.

Table 1. Test specimen details.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$f'_c$ (MPa)</th>
<th>$f_c$ (MPa)</th>
<th>Longitudinal Steel Ratio (%)</th>
<th>Transverse Steel Ratio (%)</th>
<th>Axial Load Level, N/N0 (%)</th>
<th>Column Type**</th>
<th>CFRP Ply No</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-H-0-00</td>
<td>20.0</td>
<td>19.5</td>
<td>8 x 22 mm</td>
<td>D=10 mm at</td>
<td>24.8</td>
<td>Ref., NL</td>
<td>0</td>
</tr>
<tr>
<td>S-H-1-00</td>
<td>19.5</td>
<td>287</td>
<td>Plain bars</td>
<td>200</td>
<td>27.0</td>
<td>S - NL</td>
<td>1</td>
</tr>
<tr>
<td>S-HD-1-00</td>
<td>19.0</td>
<td>287</td>
<td>Plain bars</td>
<td>200</td>
<td>27.0</td>
<td>R - NL</td>
<td>1</td>
</tr>
<tr>
<td>S-HD-1-27</td>
<td>20.0</td>
<td>287</td>
<td>Plain bars</td>
<td>200</td>
<td>27.0</td>
<td>R - UL</td>
<td>1</td>
</tr>
</tbody>
</table>

* $N_0 = 0.85 f'_c A_g + A_s f_y$

** Ref., S and R indicate reference, strengthened and repaired columns, respectively.

NL and UL indicate CFRP application was made under no load and under load, respectively.

CFRP Application

Carbon fiber wrapping system was used to retrofit and repair the test specimens. CFRP plies having 0.165 mm thickness with an elasticity modulus of 230000 MPa and ultimate strain of 0.015 were used in retrofitting process. At the first phase of the application, the corners of each column were rounded to a radius of 30 mm. After rounding off the corners, a thin layer of undercoat was applied on the plastic hinge region of the column (within 650 mm from column base) using a brush for the strengthened column. For the repaired columns this process was done after the removal and cleaning of spalled cover concrete (Figure 2.a, 2.b). Then, the column was repaired using epoxy based mortar having a corner radius of 30 mm. This process was followed by the undercoat application again. (Figure 2.c). Subsequently, the epoxy based mortar was applied and FRP sheet was wrapped around the test region of the column (650 mm) starting from 15 mm above the column-stub interface (Figures 2.d). To achieve a successful interlock between column and the CFRP sheet, homemade CFRP anchorages were placed along the test region of the columns. In order to apply these carbon fiber anchor dowels, 12 mm diameter holes with a depth of 80 mm were drilled between two longitudinal bars at heights of 50, 250, 450, 600 mm. The CFRP anchor dowels were formed from 120 x 120 mm square carbon fiber strips. These strips were rolled in the fiber direction and tied with a string. After the careful folding process, 120 mm long dowels were obtained and its 80 mm length was inserted into the holes which were drilled and cleaned previously. (Figure 2.8a,b,c,d,e)
The anchorages were placed to prevent debonding of the overlap section of the CFRP sheet during the test. The strengthened and repaired specimens were tested precisely one week after the CFRP application (Figure 2.f).

Instrumentation and Testing

All columns were instrumented by Linear Variable Displacement Transducers (LVDTs) to measure horizontal deflections and dial gages for rotations at the critical regions. There were seven LVDTs located at 350, 1000, 1750 and 2000 mm from the column – stub interface measuring deflections along the height of the column. In addition four dial gage couples were placed at a height of 350 mm that were used to record rotations relative to column - stub interface and 50 mm above it. The difference between the dial gage readings that were measured from the critical section and the surface of the column stub was accepted as the concentrated bar slip rotations due to the slip of longitudinal reinforcement in the footing. Besides, two dial gages were used in order to measure any possible movement of the column stub and strong floor. Eight strain gages were placed on longitudinal reinforcement at 50 and 350 mm heights from the surface of the column stub. The load cells were used to record the changes in lateral and axial load stages at different drift levels. There were 8 strain gages used in order to record the strain in longitudinal rebars placed 50 mm and 350 mm above the base. Columns were guided with four rollers between the guide rails to assure bending in the plane of loading. The test setup and the details of instrumentation are shown in Figure 1.

All the column tests were conducted under constant axial load and cyclic lateral displacement excursions. The application of axial load was started just before the application of lateral loading. For the column that was repaired under axial load, the axial load level was maintained about one week during repair and curing of CFRPs to ensure proper curing of epoxy resin. Then the column was subjected to lateral loading excursions. The drift levels were incremented by 0.5% until the drift level of 3% having three cycles for each drift level. The following cycles beyond 3% drift level were incremented by 1% having two cycles for each level of drift.

TEST OBSERVATIONS

The lateral force – column tip deflection ($P - \Delta$) were measured using four LVDTs that were located at 2000 mm from the base of the column. The $P - \Delta$ responses of columns and important events during the test such as column – stub cracking, CFRP debonding and CFRP rupture is presented in Figure 3.

A similar failure mode was observed at the base of all columns due to column plastic hinging. Flexural cracks were observed in the first lateral displacement excursions and those cracks opened further and joined at both faces as the displacement excursions increased. Considering the fact that all the columns were flexural dominant having an aspect ratio of about 5.7, the observed failure mode for all columns was initiated with cracking of the column – stub interface and progressed with further opening of this crack simultaneously with concrete crushing or CFRP debonding. The progression of concrete crushing and/or CFRP debonding resulted in buckling of longitudinal bars and sudden explosion of CFRP sheet. Plastic hinging started with the crushing of the corners at the base of the column and the gradual advance in the length of the hinging region reached up to a length of approximately the depth of the section, $h$. All the CFRP retrofitted columns experienced a sudden rupture of the CFRP sheet and consequently the observation of the plastic hinge lengths eventuated after that. However, for the reference specimen the formation of the hinging length was observed during the loading cycles. Test results are summarized in Table 2 and the envelope curves of $P - \Delta$ and $M - \Theta$ responses for the columns are shown in Figure 4. The strains in longitudinal rebars above 50 mm of the base are presented in Figure 5. It can be also observed that at the end of damage cycles the longitudinal rebars are very close to yielding.
Reference Column: S-H-0-00

In the reference specimen, S-H-0-00, the column-stub interface cracked at 0.5% drift level and as the column performed inelastic displacement cycles, the slipping of the reinforcement increased as a result of the widening of the interface crack. Furthermore, evenly distributed flexural cracks were observed at both faces of the column at heights of 50 to 1000 mm. First cover crushing, initiated from the longitudinal cracks at the base of the column at a height of approximately h, was observed at 2% drift level. Lateral load carrying capacity degraded below 80% of the ultimate was followed by the longitudinal bar buckling at a drift level of about 4%.

Retrofitted Columns: S-H-1-00, S-HD-1-00, S-HD-1-27

All the retrofitted columns were wrapped with 1 layer of CFRP within the expected plastic hinge region of 650 mm starting from 15 mm above the column base. In all columns, the column-stub interface cracking was observed at 0.5% drift level. In the strengthened column, S-H-1-00, the flexural cracks were developed at heights of 650 to 1000 mm from the base of the column at the initial cycles of displacement excursions. These cracks were closer to each other than those observed in the reference specimen. In the following cycles, the flexural cracks opened further and closed in the opposite cycles of deformations. As a result of the horizontal CFRP cracks at 100 mm from the base, CFRP at the other face of the column started to debond at about 2.5% drift level. The confinement effect of CFRP helped in maintaining the lateral capacity constant and preventing longitudinal bar buckling. The interface crack (i.e., crack observed at column base stub interface) widened which resulted in increase of the longitudinal reinforcement slip at the column-stub interface and the lateral load reached the lateral load carrying capacity at about 3% drift. The first rupture at the CFRP sheet occurred at 5% drift and at 6% drift level the lateral load dropped 80% of the capacity by buckling of the longitudinal reinforcement. The repaired columns, S-HD-1-00 and S-HD-1-27 exhibited similar deformation behaviour. In the first phase of the tests, both of the repaired columns were subjected to a moderate damage level of 2% drift. Evenly distributed flexural cracks were formed at heights of 50 to 1000 mm from the column base. The only difference was the presence of axial load during the repairing process, which was about 27% of the capacity in the column S-HD-1-27. The flexural cracks that formed in the pre-damage state opened further and no new flexural cracks outside the strengthened region occurred in the retesting stage. The drift levels at which CFRP debonding, CFRP rupture and 20% drop in lateral capacity took place were identical for the three specimens (2%, 5% and 6% drift respectively). The lateral load capacity of the repaired column S-HD-1-27 did not change as compared to the capacity observed at the damage stage whereas its companion column S-HD-1-00 gained lateral capacity enhancement of about 15%. For the repaired columns, the strength of the columns decreased compared to the strengthened column as a result of the initial damage. Furthermore, reductions in energy dissipation capacities were observed when compared to the strengthened specimen.

DISCUSSIONS

Effect of CFRP Wrapping

Strengthening or repairing square columns with 1 ply of CFRP sheet significantly improved the ductility and energy dissipation capacity of the columns. However, the strength and energy dissipation capacity of the repaired columns, S-HD-1-00 and S-HD-1-27, were lower than the strengthened column, S-H-1-00, which had no prior damage. This can be attributed to cracking and stiffness degradation that occurred in the pre-damage state, i.e. no effort was needed for the reopening of the interface crack at the base and other flexural cracks outside plastic hinge region. Wrapping 1 layer of CFRP sheet increased the ultimate drift ratio of the retrofitted columns, S-H-1-00, S-HD-1-00 and S-HD-1-27 by a factor of 1.5 compared to the reference column S-H-0-00.

Effect of Repairing under Axial Load

The effect of axial load during repairing was evaluated by comparing the columns S-HD-1-00 and S-HD-1-27. It was observed that both of the columns performed similar inelastic responses achieving similar ultimate drift deformations. The presence of axial load during the repairing process did not affect the ultimate drift ratios (~6%). The drift levels at which the CFRP debonding and rupture took place were also similar (2% and 5%, respectively) for the specimens repaired in the presence and absence of axial load. The only effect of axial load during repairing was the gain in lateral capacity of about 15% as compared to the reference column. This difference can be attributed to the high compressive strength of the epoxy repair mortar (about 70 MPa) compared to the concrete compressive strength (about 20 MPa) of the original column. The high variation in compressive strength (3.5 times the concrete) forced the repaired column S-HD-1-00 to increase its lateral capacity after the application of the axial load. However, for the repaired column under axial load the section,
lateral load carrying capacity was kept constant (i.e. axial load was always carried by the damaged column, not by the epoxy mortar) as the section behaviour was not significantly affected by the high compressive strength of the epoxy mortar.

Table 2. Test Results

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Interface Cracking</th>
<th>CFRP</th>
<th>20% Capacity Drop</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>u, Drift (%)</td>
<td>u, Drift (%)</td>
<td>u, Drift (%)</td>
</tr>
<tr>
<td>S-H-0-00</td>
<td>2.0 (crushing)</td>
<td>4.0</td>
<td></td>
</tr>
<tr>
<td>S-H-1-00</td>
<td>0.5</td>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>S-HD-1-00</td>
<td>2.0</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td>S-HD-1-27</td>
<td>2.0</td>
<td>6.0</td>
<td></td>
</tr>
</tbody>
</table>

Figure 3. Lateral load vs. Deflection responses of columns. (Dotted lines indicate damage cycles for repaired columns)

Figure 4. Envelope responses for: a. Lateral load vs. Deflection b. Moment vs. Curvature
SUMMARY AND CONCLUSIONS

Results from an experimental study, in which four columns were tested under constant axial load and cyclic lateral displacement excursions that simulated seismic forces, are presented in this study. Each column had dimensions of $350 \times 350 \times 2000$ mm cast vertically together with a stub of $400 \times 500 \times 1350$ mm that represented a typical near full scale building column. Three specimens representing the strengthened and repaired columns were retrofitted with 1 layer of CFRP sheet. Other column was termed as reference column as it represented a deficient building column that had plain longitudinal bars, insufficient transverse reinforcement and relatively low concrete compressive strength. The effect of CFRP confinement and presence of axial load during repairing were studied. The following conclusions can be drawn from this study:

1. 1 layer of CFRP sheet used to confine the plastic hinge regions of the columns significantly improved the seismic behaviour of the deficient columns by improving the ductility and the energy dissipation capacity of the columns.

2. The energy dissipation capacities of the repaired columns were lower than the strengthened column due to the initial cracks that formed during the application of the moderate level of previous damage.

3. The presence of axial load level of 27% of the capacity during repairing had almost no influence on the ultimate drift and ductility of the columns. The repaired columns behaved in a similar manner with the strengthened column. On the contrary, the lateral load carrying capacity increased by 15% of the reference column in the absence of the axial load during repairing. This fact was due to the higher compressive strength of the epoxy mortar compared to the existing compressive strength of concrete that influenced the section capacity in the absence of axial load. When the axial load was present there was no gain in the lateral capacity of the column. In this case, the section capacity was not affected since the axial load was not perceived by the epoxy mortar repair section of the column.

REFERENCES


