RETROFIT OF SLENDER SQUARE REINFORCED CONCRETE COLUMNS WITH GLASS FIBER-REINFORCED POLYMER FOR SEISMIC RESISTANCE

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Abstract—This paper explains an experimental research program on the use of GFRP (Glass Fiber-Reinforced Polymer) for retrofitting small-scale slender R/C columns to enhance seismic performance. An important deficiency in many existing nonductile reinforced concrete frames is the inability of the columns to undergo significant deformations while maintaining their load-carrying capacity. As a result, relatively brittle modes of column failure, accompanied by soft storey structural failure mechanisms, are possible. Providing additional confinement to the columns allows them to behave in a more ductile manner.

In this study, six 1/2-scale columns were constructed and tested under cyclic loading to examine the effectiveness of the retrofitting technique for improving the seismic resistance of slender concrete columns. Three columns were tested after being retrofitted with GFRP wraps at the potential plastic hinge zone, while three others were tested in the “as-built” condition. One of the “as-built” specimens was constructed according to ACI (318-02) detailing and five others were built in accordance with ACI (pre.1971). In general, the common mode of failure for the “as-built” samples was a brittle failure due to bond deterioration of the lap-spliced longitudinal reinforcement. Test results suggest that GFRP wraps can significantly increase the flexural strength and ductility of slender rectangular reinforced concrete columns.

Keywords—Slender columns, GFRP wraps, retrofit, seismic performance, ductility

1. INTRODUCTION

The behaviour of columns in earthquakes is very important since column failures lead to additional structural failures and can result in total building collapse. A large number of residential and commercial buildings were built in seismic prone zones with soft stories at first-floor level. Recent earthquakes in Kobe (Japan-1995), Izmit (Turkey-1999), Chichi (Taiwan-1999), Gujarat (India-2001), Boumerdes (Algeria-2003) and Bam (Iran-2003) have emphasized that these columns can cause partial or even the total collapse of buildings.

Moreover, many existing multi-storey reinforced concrete frame buildings were designed for gravity loads and lateral forces that may be much smaller than prescribed by existing building codes. The lateral load resistance of these buildings is not adequate, even for moderate earthquakes, as a result of the nonductile reinforcing details and lack of sufficient reinforcement. In other words, older reinforced concrete columns can be especially vulnerable structural elements because of insufficient lap splice in the longitudinal reinforcements, lack of confinement in flexural hinge zones, and also having 90° degree hooks. Inadequate spacing and configuration of transverse reinforcement, resulting in inadequate details as well as a propensity for reinforcement buckling, often limit the flexural response of existing columns. Furthermore, if soft storey columns have such defective details, the strength and ductility will be reduced remarkably. In fact, past earthquakes have demonstrated that these columns are more vulnerable against major earthquakes where they are located along the perimeter of buildings and withstand low axial loads and large flexural moments.

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In Iran, slim R/C columns with insufficient lap splices, and/or too widely spaced ties with 90° hooks are extensively used as consequence of the poor design and/or poor construction practice; thus, a large stack of R/C buildings are extremely vulnerable to moderate and strong earthquakes. Seismic retrofit of such numerous vulnerable buildings is an important technical and societal issue that should be resolved. This emphasizes a need for developing retrofit techniques to enhance flexural strength and ductility of R/C columns.

Indeed, applying FRP sheets in order to repair and retrofit R/C buildings has become widespread over the last 15 years. Several researches have been conducted into the use of fiber reinforced polymer composite jackets for seismic strengthening and the repair of columns having deficiencies such as inadequate lap splice in the longitudinal reinforcement or a lack of shear strength [1-5].


As a result, based on an extensive literature survey, no research has been carried out on the retrofit of slender rectangular concrete columns with deficient lap splices by fiber-reinforced polymer. Also, few studies have been investigated on the seismic behavior of retrofitted rectangular columns according to ACI (pre. 1971) with GFRP wraps.

The work presented in this paper deals with the experimental studies on the cyclic behavior of strengthened slender square reinforced concrete columns by GFRP wraps at the plastic hinge region. A comparison of “as-built” and retrofitted specimens allows a direct evaluation of the effect of GFRP sheets on the strength and ductility of the slender R/C columns with some defective detailing.

In this study, all the specimens tested up to a point near the balance point in the interaction diagram of axial load and bending. Furthermore, all the columns could be classified as slender columns. The specimens represented the columns which are located at the perimeter of the R/C buildings. In these columns, flexural moment plays an important role in their behavior, and their performance is more critical when they have a short splice length.

2. EXPERIMENTAL PROGRAM

Six column specimens, square in shape and 1:2 scale, were designed. The specimens consisted of a cantilever column fixed in a strong foundation block. Five of the specimens were characterized by short splice length, widely spaced transverse reinforcement with a 90° hook, in accordance with ACI (pre.1971) detailing, and one of the specimens was constructed according to ACI (318-02) detailing. [9]

Three columns were retrofitted before being testing and the rest were tested in order to compare with the strengthened specimens. All the specimens were tested under a constant axial and cyclic lateral load to simulate seismic loading condition. The main variable parameter was the axial load level.

a) Specimens

The columns had 150-mm×150-mm cross sections with eight 8-mm diameter longitudinal bars. Five of the specimens had 4-mm ties, spaced at 150-mm, with 90° hooks. These specimens were according to ACI (pre.1971). Splice length in these specimens was considered 20 times the longitudinal bar diameter. One of the designed specimens was constructed according to ACI (318-02) detailing. 4-mm diameter ties were situated at 60-mm with 135° hooks, and were applied to reinforce this column transversely.

The specimens signified that the columns are placed in a multistorey building frame between the maximum moment and contraflexure point. The height of the columns was 1000-mm, but the lateral load was applied at the 800-mm level of the column. Specifications of the specimens are listed in Table 1.
Table 1. Specifications of the specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Axial load (kN)</th>
<th>Layers of GFRP</th>
<th>$A_{sh}$</th>
<th>$\rho_{sh}$ %</th>
<th>Splice (20 dB)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-S1</td>
<td>21.2</td>
<td>0</td>
<td>0.112</td>
<td>0.644</td>
<td>No</td>
</tr>
<tr>
<td>SP-S2</td>
<td>21.2</td>
<td>0</td>
<td>0.112</td>
<td>0.258</td>
<td>Yes</td>
</tr>
<tr>
<td>SP-S3</td>
<td>63.6</td>
<td>0</td>
<td>0.112</td>
<td>0.258</td>
<td>Yes</td>
</tr>
<tr>
<td>SP-SS1</td>
<td>21.2</td>
<td>4</td>
<td>0.112</td>
<td>0.258</td>
<td>Yes</td>
</tr>
<tr>
<td>SP-SS2</td>
<td>42.4</td>
<td>4</td>
<td>0.112</td>
<td>0.258</td>
<td>Yes</td>
</tr>
<tr>
<td>SP-SS3</td>
<td>63.6</td>
<td>4</td>
<td>0.112</td>
<td>0.258</td>
<td>Yes</td>
</tr>
</tbody>
</table>

b) Steel properties

Two types of reinforcing steel were used to construct steel cages of the specimens. 8-mm hot rolled deformed bars were applied for longitudinal reinforcement of columns and stubs; also, 4-mm diameter bars were employed to column ties. Mechanical properties of the reinforcing steels were calculated and are summarized in Table 2.

Table 2. Properties of reinforcing steels [10]

<table>
<thead>
<tr>
<th>Bar size (mm)</th>
<th>$f_y$ (MPa)</th>
<th>$\varepsilon_y$</th>
<th>$E_s$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$\varepsilon_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>420</td>
<td>0.001686</td>
<td>2.3719×10^5</td>
<td>640</td>
<td>0.115</td>
</tr>
<tr>
<td>4</td>
<td>300</td>
<td>0.001462</td>
<td>2.051×10^5</td>
<td>500</td>
<td>0.129</td>
</tr>
</tbody>
</table>

c) Concrete properties

The concrete used in the columns was mixed considering a water cement ratio of (0.5). The columns and foundations were cast separately. The average slumps were 120.2-mm and 132.3-mm for columns and foundations respectively. Twelve 152.5 × 305 mm (6×12 in) standard cylindrical samples were tested to obtain the nominal concrete strength. Separately, six cylinders were sampled from both the foundation and column concrete batch. The maximum size of the aggregates was 15-mm. All the columns and footings were cast vertically and were vibrated with a rod vibrator. The average nominal concrete strength at 28 days ($f_{c}^'$) were 18.9 MPa and 16.52 MPa for the columns and footings respectively.

d) Reinforcing details

The structural detailing of the specimens consisted of two segments, a stub cage and a column cage. They were created separately and then connected to each other. Fig.3 displays the reinforcement detailing of the specimens. Column type C1 detailing is according to ACI (pre.1971), and type C2 is in accordance with ACI (318-02). Detailing of both types of column footings were designed strong enough to prevent any damage and/or cracking during the test.
e) FRP properties and installation

Columns were confined by being wrapped with bidirectional GFRP sheets and a special matrix. E-glass fiber reinforced polymer in the form of unidirectional fabric, and vinyl ester resin were utilized in this study. Three of the specimens were strengthened up to a 240-mm (1.5L_d) height of the columns. Moreover, wraps continued to the surface of its stubs. These columns were wrapped by four layers of FRP sheets. They were cleaned and completely dried before the vinyl ester was applied on their surfaces (Fig. 4). In order to prevent stress concentrations that may cause the premature failure of the FRP system, the corners of each column were rounded to an approximately 20-mm radius.

Table 3. Mechanical properties of fibers [10]

<table>
<thead>
<tr>
<th>Coefficient of thermal expansion 10^9/C</th>
<th>Tensile modulus (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Density (N/mm^3)</th>
<th>Strain to failure (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.7</td>
<td>70000</td>
<td>1750</td>
<td>25.5</td>
<td>4.8</td>
</tr>
</tbody>
</table>

The recommended resin was mixed and then was applied on the concrete surface in a thin uniform layer using a roller. A fiber sheet was cut to desired length and width, and was pressed on the concrete using a “bubble roller” (Fig. 5). This act eliminates entrapped air between fibers and resin and ensures the impregnation of the FRP sheet with resin. The mechanical properties of glass fiber-reinforced sheets are

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**Fig. 3. Specimens details [10]**
summarized in Table 3. All the strengthened specimens were cured at least 7 days to ensure that full strength was obtained before testing due to the advice of the manufacturer.

Fig. 4. Installing of prefabricated composite jacket [10]  
Fig. 5. Retrofitted column and its stub [10]

f) Test setup and instrumentation

The test setup is displayed in Fig. 6. The lateral cyclic load was applied by an actuator with a 50 kN capacity that was mounted on reaction steel frame. Also, the axial load was applied with a 250 kN capacity vertical hydraulic jack. The lateral displacement history was imposed at the 800-mm level of the column height. The column stubs were fastened to the strong floor with eight high strength rods, and each rod was pre-stressed up to 100 kN to prevent any slip or overturning under large lateral load. The columns were tested in vertical position, cantilevered out from a heavily reinforced footing block that was anchored to a strong floor.

![Test setup diagram](image)

Fig. 6. Test setup [11]

Vertical linear variable differential transformers (LVDT s) with a 50-mm length and $200 \times 10^{-6}$/mm sensitivity were employed on the specimens to measure the rotations of the columns relative to footing and the probable vertical displacement of the footings because of the lateral force. Each “as-built” specimen was instrumented with 3 steel strain gauges (Fig. 2). Moreover, the retrofitted columns were instrumented with two strain gauges on the surface of FRP sheets, and all the specimens had 2 strain gauges on the columns.

g) Testing

During the tests, each specimen was subjected to cyclic lateral load through displacement increments, and constant axial load at three levels: (0.05, 0.10, 0.15$A_{gf}c$). Displacement based control for lateral load
was considered. Each cycle of the lateral loading consisted of positive and negative loading. Also, two reversed cycles were applied at each deflection level in order to evaluate the strength and stiffness degradation at repeated lateral load reversals. The applied displacement cycles for all the specimens are given in Fig. 7.

Fig. 7. Lateral displacement history

3. TEST RESULTS AND DISCUSSION

Six specimens (Table 1) were tested under different levels of axial load and the same lateral displacement history. Although there are several measurement curves to evaluate the performance of the specimens, moment versus curvature relationship were selected to be presented here. Moment versus base rotation curves, experimentally observed damage, stiffness of the specimens during the test and also the ductility of the specimens are presented in the following section.

a) Observed damage and behavior

During testing of specimen SP-S1, initial flexural cracks were observed at the column footing interface when displacement reached 1% of the drift ratio, and at 1.5% drift ratio at the end of the splice length. Some flexural cracks appeared along the splice during the test but the length and width of these cracks did not increase significantly (Fig. 8a).

In specimen SP-S2 flexural cracks on the column face started as early as at 1% lateral drift ratio and at this time longitudinal cracks were formed at right and left sides of the specimen. During the second cycle at 3.5% drift, fairly large sections of the concrete cover spalled at the column footing interface and the lateral load strength diminished, indicating that the bond along the spliced bars was deteriorating. Shear cracking was observed at 2.5% drift ratio but these cracks were not significant throughout the test. Damages in this specimen were more serious than in the previous one (Fig. 8b).

Specimen SP-S3 was subjected to a (0.15 $A_g f'c$) constant axial load. Initial flexural longitudinal cracks suddenly appeared at 1.5% drift ratio. In this specimen, cracking was formed later; however, cracking propagation was developed suddenly as the applied lateral displacement was increased. Cover spalled completely at the corner at 4.5% drift ratio and the longitudinal bars were clearly visible. Along the splice length, there was severe damage. Shear cracks were observed at 3.5% drift ratio on the right and the left faces of the columns and concrete crushing and spalling were observed just before the end of the test. In this specimen, damage was more severe than in SP-S1 and SP-S2 (Fig. 8c).
a) Specimen SP-S1  
(Hairline cracks at end of the splice length)  

b) Specimen SP-S2  
(Severe damage at bottom of the column)  

c) Specimen SP-S3  
(Concrete spalling at the corner)  

Fig. 8. Specimens at end of the testing  

d) Specimen SP-SS1  
(Narrow flexural crack just above the jacketing)  

e) Specimen SP-SS2  
(Stress concentration on the fiber)  

f) Specimen SP-SS3  
(Narrow diagonal cracks)  

The retrofitted specimens disclosed significantly improved seismic performance and less damage occurred in them. Vertical and horizontal flexural cracks just above the FRP jacket in the specimen SP-SS1 were first observed at 1.5% drift ratio and were propagated during the test. Cracks were similar to the column footing interface and end of the splice length of SP-S2, appearing at 1% drift ratio. While testing, no crushing or spalling, and stress concentration on the FRP were observed. Very narrow shear cracks were formed at 5% lateral drift ratio at the right and left side of the column.  

The response of the SP-SS2 under cyclic loading was similar to SP-SS1, but in this specimen at 4% drift ratio shear cracks were formed and widened during the test and stress concentration appeared at the column footing interface (Fig. 8e).  

In the specimen, SP-SS3, damage was more serious and propagated rapidly. At 2.5% drift ratio flexural cracks were observed above the FRP jacket and vertical cracks were formed at 3% lateral drift ratio. These cracks quickly developed into flexural-shear cracks as they were extended through the right and the left sides of the column. Shear cracks were clearly visible (Fig. 8f) before the test had been finished. Furthermore, stress concentrated at the column-footing interface and 20-mm of the wrapped fibers at the corner of the column were separated from the column.  

No delaminating or rupture across the fibers of the jackets was observed in any of the specimens.  

b) Base moment-curvature response  

The hysteretic response of the specimens is graphically presented in the form of the relationships between the columns base moment and curvature (Fig. 6). The total moment at the base of the columns was calculated by summing up the moments caused by the lateral load, and the vertical load as given by Eq. (1) follow as:  

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The curvature values obtained by the data measured by two LVDTs were located at the bottom and midpoint of the column (see Fig. 6).

\[ M = F \cdot h + P \cdot \Delta_{\text{Lateral}} \]  \hspace{1cm} (1)
c) Ductility parameters

In seismic design, the inelastic deformation is generally quantified by ductility parameters. To evaluate the performance of the specimens, several empirical member and section ductility parameters which make the comparison between retrofitted and original specimens from the research carried out by Khoury and Sheikh, were used in this study [12].

Although section ductility parameters are more reliable for comparison because of the type of experiment, displacement control and the member ductility parameters are also evaluated.

Figure 10 depicts some ductility parameters including curvature and displacement ductility factors, $\mu_\varphi$ and $\mu_\Delta$, cumulative ductility, and displacement ratios, along with the cumulative ductility ratios $N_\varphi$ and $N_\Delta$, which represent the deformability of the concrete member and section, respectively. Both work damage indicator $W$ and energy damage indicator $E$ quantify the energy dissipation capacities of the entire member and specific hinging section, respectively; these provide an estimate of toughness. Subscripts $t$ and $80$ are added, respectively, to $N_\varphi$, $N_\Delta$, $E$, and $W$ to indicate the values of each parameter until the end of the test, and until the end of the cycle in which the value of shear force or moment were dropped to approximately 80 percent of the maximum value. This is followed by a cycle in which the
capacity loss is significantly greater than 20 percent. In Fig. 10, \( L_f \) represents the length of the most damaged region measured during the test, and \( h \) represents the depth of the column section.

Table 4 includes the curvature and displacement ductility parameter values for 20% reduction in member or lateral loads beyond the peak and until the end of the test.

Table 4. Section and member ductility values

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( \mu_{\phi^{80}} )</th>
<th>( N_{\phi^{80}} )</th>
<th>( N_{\phi t} )</th>
<th>( E_{80} )</th>
<th>( E_t )</th>
<th>( \mu_{\Delta^{80}} )</th>
<th>( N_{\Delta^{80}} )</th>
<th>( N_{\Delta t} )</th>
<th>( W_{80} )</th>
<th>( W_t )</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-S1</td>
<td>†</td>
<td>†</td>
<td>44.6</td>
<td>†</td>
<td>1180.2</td>
<td>4.92</td>
<td>58.6</td>
<td>69.66</td>
<td>650.9</td>
<td>612.4</td>
</tr>
<tr>
<td>SP-S2</td>
<td>4</td>
<td>29.1</td>
<td>38.4</td>
<td>580.7</td>
<td>786.3</td>
<td>4.45</td>
<td>47.2</td>
<td>59.6</td>
<td>400.2</td>
<td>430.6</td>
</tr>
<tr>
<td>SP-S3</td>
<td>3.1</td>
<td>24.1</td>
<td>45.8</td>
<td>165.2</td>
<td>423.9</td>
<td>3.62</td>
<td>45.2</td>
<td>58.6</td>
<td>275.2</td>
<td>354.1</td>
</tr>
<tr>
<td>SP-SS1</td>
<td>4.5</td>
<td>52.4</td>
<td>55.9</td>
<td>1523.3</td>
<td>1862.3</td>
<td>5.44</td>
<td>67.3</td>
<td>73.6</td>
<td>1121.3</td>
<td>1253.5</td>
</tr>
<tr>
<td>SP-SS2</td>
<td>4.5</td>
<td>41.7</td>
<td>53.4</td>
<td>952.6</td>
<td>1423.2</td>
<td>5.12</td>
<td>55.2</td>
<td>65.8</td>
<td>950.3</td>
<td>1086.3</td>
</tr>
<tr>
<td>SP-SS3</td>
<td>4.3</td>
<td>40.7</td>
<td>48.7</td>
<td>890.3</td>
<td>1222.8</td>
<td>4.56</td>
<td>53.9</td>
<td>62.12</td>
<td>593.7</td>
<td>761.4</td>
</tr>
</tbody>
</table>

\( \dagger \) = moment did not drop to this level

1. **Effect of GFRP retrofitting on the defective columns:** Influence of glass composite sheets on the strengthening of deficiently built square columns was assessed by comparing similar unretrofitted columns, tested under identical loading conditions. Specimens SP-S2 and SP-SS1 had identical detailing, and both were subjected to identical loading, but only SP-SS1 was strengthened. Experiments indicated that the section and member ductility parameters \( \mu_{\phi^{80}}, \mu_{\Delta^{80}} \), and 80% \( N_{\phi^{80}}, N_{\Delta^{80}} \), total cumulative ductility ratio \( N_{\phi t}, N_{\Delta t} \) and energy indicator \( E_{80}, W_{80} \) were increased as shown in Table 4. In this Table, if one compares two specimens, SP-SS2 and SP-S2, with identical loading conditions, it would reveal that the total cumulative ductility and damage indicator parameters of the first specimen were much greater than the second.

2. **Effect of axial load:** Specimen SP-S1, SP-S2 and SP-SS1 were similar in level of axial load; also, SP-S3 and SP-SS3 were similar too. Axial load were \((0.05Agf'c)\) and \((0.15Agf'c)\) for SP-S2 and SP-S3 respectively, and as shown in Table 4, in view of higher axial load in SP-SS3, substantial reductions in curvature and displacement ductility factors \( \mu_{\phi^{80}} \) and \( \mu_{\Delta^{80}} \) were observed in comparison with SP-SS1. Moreover, the cumulative curvature ductility ratio revealed significant reductions from 29.05 to 24.13 for \( N_{\phi^{80}} \), 45.8 to 38.4 for \( N_{\phi t} \), 55.2 to 53.9 for \( N_{\Delta^{80}} \), and 65.8 to 62.12 for \( N_{\Delta t} \) in SP-SS3 as a result of increased axial load. Similar conclusions can be drawn through energy and work damage indicators \( E_{80}, E_t, W_{80}, W_t \). Energy damage indicator differences between specimens because of the axial load were more considerable. For instance, the dissipated energy in specimen SP-SS1 measured by \( E_{80} \) and \( E_t \) is 2 to 3 times as much as the energy dissipated in SP-S2. A higher axial load led to a noticeable decrease in the energy dissipation in the “as-built” and retrofitted specimens; in addition, axial load augment adversely affected the cyclic performance of the strengthened and “as-built” specimens.

d) **Residual lateral strength**

Response of reinforced concrete columns to a cyclic loading depends strongly on the ability to sustain large inelastic deformation without significant degradation of lateral and axial load-carrying capacity [13]. In this respect, residual lateral displacement plays an important role in the stable behaviour of a structure. For this reason, residual displacements were measured at the end of each test.

At the end of each test, specimens were pushed 50-mm to positive direction, and then were allowed to return to the first place. Some remaining displacement was observed in all the specimens. Measured
permanent displacement represents reduced strength in the specimens. Comparison between the deflection of columns before and after the tests explains residual strength and stiffness. It can be assumed as a reliable parameter for confirming the effectiveness of the strengthening technique to prevent soft storey in buildings. Table 5 displays the residual displacement of the specimens.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Residual displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-S1</td>
<td>8.5</td>
</tr>
<tr>
<td>SP-S2</td>
<td>17.2</td>
</tr>
<tr>
<td>SP-S3</td>
<td>20.5</td>
</tr>
<tr>
<td>SP-SS1</td>
<td>8</td>
</tr>
<tr>
<td>SP-SS2</td>
<td>11.3</td>
</tr>
<tr>
<td>SP-SS3</td>
<td>12.2</td>
</tr>
</tbody>
</table>

e) Cumulated energy dissipation

Energy dissipation at each cycle was calculated by taking the area under each loop of moment-curvature curves, and cumulating whole areas of the cycles. Since imposed drift ratios were numerous and high, high energy dissipation was observed.

Despite energy dissipation calculated as described did not represent the total energy dissipated by the specimens, it can be used for comparison purposes. Cumulated energy dissipating of specimens is presented in Table 5.

A general survey of the values of cumulated areas under loops of the moment-curvature curves in Table 6 reveals a substantial improvement in inelastic response in all upgraded specimens. As an illustration, energy dissipation in specimens SS1 and SS3 are 3.98 and 4.32 times larger than the corresponding values of the original specimens. Also, by increasing axial load level, differences of values between retrofitted and unretrofitted columns were augmented.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cumulated areas (kN-Rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP-S1</td>
<td>656.6</td>
</tr>
<tr>
<td>SP-S2</td>
<td>342.2</td>
</tr>
<tr>
<td>SP-S3</td>
<td>202.2</td>
</tr>
<tr>
<td>SP-SS1</td>
<td>1360.5</td>
</tr>
<tr>
<td>SP-SS2</td>
<td>1059.3</td>
</tr>
<tr>
<td>SP-SS3</td>
<td>874.22</td>
</tr>
</tbody>
</table>

f) Stiffness degradation on specimens

Stiffness for each cycle was calculated as the average of the stiffness for the positive and negative directions (Fig. 8). Stiffness for each cycle was normalized with respect to the stiffness of the first cycle and is plotted versus tip displacement in Fig. 7.

![Fig. 11. Calculation of column stiffness [3]](image)

Stiffness degradation versus tip displacement of the columns after normalization are shown in Figs. 12 and 13. These figures demonstrate progressive stiffness degradation, and a significant reduction of strength in “as-built” specimens under imposed loads.
4. SUMMARY AND CONCLUSIONS

Slender reinforced concrete columns with insufficient lap splice length and poor confinement are not capable of dissipating seismic energy due to lack of necessary ductility, and can fail without enough warning during major earthquakes. In this paper, the consequences of an experimental program on half scale square R/C columns confined with an FRP system were presented. To evaluate the performance of retrofitted R/C columns with GFRP, six square concrete columns according to ACI (pre.1971) and ACI (318-02) were constructed and tested under constant axial load and cyclic lateral displacements. On the basis of the above investigation the following conclusions could be drawn:

1- GFRP wraps were effective for increasing the flexural strength and ductility capacity of the concrete columns.

2- In the retrofitted specimens, the rate of stiffness deterioration under large reversed cyclic loading was lower than that of the corresponding original columns.

3- Use of GFRP sheets as a retrofit technique was successful in changing the failure mode of the “as-built” columns from the slip along the splice length, and severe flexural and shear cracks to yield at the starter bars and flexural damage above the retrofitted region and shifting failure away from the interface from the column and the footing.

4- Ductility, flexural strength, and energy dissipation of the R/C columns according to ACI (318-02) are significantly higher than those according to ACI (pre.1971).

5- In all the specimens, the rate of stiffness degradation was dependent on the level of the axial load and applied drift ratio; nevertheless, the strengthened columns have shown a lower enhancement of strength and stiffness with respect to the “as-built” columns.

6- Cyclic behavior of the specimens revealed that rotation capacity has been considerably increased in the retrofitted columns.

7- Residual displacement at the end of the tests indicated that this retrofit technique can be a proper and practical method to prevent soft storey mechanism in the R/C buildings with the mentioned defective detailed columns.

8- Strain gauges records on the reinforcements, curvature and displacement ductility, and observed damages of the retrofitted columns demonstrated that GFRP sheets by proper confinement prevented buckling, slippage or bond failure of longitudinal bars or any brittle damage in critical zones.
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NOMENCLATURES

\( A_g \)  
\( A_{sh} \)  
\( E \)  
\( W \)  
\( E_s \)  
\( f'_c \)  
\( f_u \)  
\( f_y \)  
\( L_f \)  
\( L_d \)  
\( N_\phi \)  
\( N_\Delta \)  
\( P \)  
\( \varepsilon_u \)  
\( \varepsilon_y \)  
\( \mu_\phi \)  
\( \mu_\Delta \)  
\( \rho_{sh} \)  
\( \phi \)  
\( \phi_1 \)  
\( \phi_2 \)  
\( \Delta_f \)  
\( \Delta_1 \)  
\( \Delta_2 \)  

REFERENCES


