STRUCTURAL EVALUATION OF GFRP-CONFINED REINFORCED CONCRETE HOLLOW COLUMNS

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ABSTRACT

Although hollow-core reinforced concrete (RC) bridge piers are frequently used in practice, no guidelines are available for the design of their upgrade. Also, literature on hollow columns is scarce. Recent studies confirmed that the failure of hollow columns is affected by premature bars buckling and concrete cover spalling. This paper discusses a pilot experimental investigation on full-scale RC hollow columns confined with glass FRP (GFRP) and subjected to pure axial load. Hollow column specimens were characterized by an overall length of 3.05 m [10 ft], cross-sectional dimensions of 508 mm by 737 mm [20 in by 29 in] and a wall thickness of 114 mm [4.5 in]. Three specimens were tested: one unstrengthened (benchmark) and two GFRP-jacketed with two different confining reinforcement ratios. The specimens were designed according to dated codes (i.e., prior to 1970) for gravity loads only. In particular, steel longitudinal bar area and steel tie spacing were designed following the ACI 318-63 code-mandated minimum amount of longitudinal reinforcement and minimum tie area at maximum spacing for solid elements. The objective of this study was to determine how the confinement by means of externally bonded FRP sheets impacts column performance. Specifically, the investigation aimed at characterizing: the confinement contribution in terms of axial strength and prevention of instability of longitudinal bars; and the sensitivity of any enhancement in performance to the confining reinforcement ratio. It is demonstrated that the FRP confinement contributes to an increase in deformability as a function of the confinement reinforcement ratio.

KEYWORDS

confinement, deformability, FRP laminates, RC hollow-core columns, strengthening.

INTRODUCTION

Hollow-core concrete columns are an economically-attractive solution in concrete bridges to maximize the structural efficiency of the strength-mass and stiffness-mass ratios and to reduce the mass contribution of the column to seismic response and high carrying demand on foundations. To-date many concrete bridges incorporate hollow-core concrete piers particularly in areas where high seismic activity and natural boundaries require high-elevation infrastructures. However, modern codes of practice oriented to seismic design do not address any specific problem related to hollow sections. Many bridge hollow piers represented a threat in regions under high seismic risk due to insufficient shear or flexural strength, low ductility or inadequate seismic detailing and have been in need of retrofitting. Lately, retrofitting with fiber reinforced polymer (FRP) materials have become a valuable alternative to concrete or steel jackets as solution for strengthening, repairing and adding ductility with no significant increase of structural masses, no traffic disruption, rapid execution, long-term durability, and lower life-cycle costs (Karbhari and Zhao, 2000). Many studies have been carried out on FRP-confined reinforced columns (RC) circular and prismatic solid columns in the past years, and several analytical models have been proposed. On the other hand, the behavior of FRP-jacketed RC hollow columns has been much less investigated.

A research by Osada et al. (1999) showed that the FRP jacket is capable of increasing the shear strength of the column and of avoiding premature buckling of longitudinal bars. Tests by Mo et al. (2004) showed that both ductility and shear strength can increase with the number of FRP sheets, preventing all shear cracks and changing the failure mode of the specimen from shear to flexure. Lignola et al. (2007a) investigated the behavior of square hollow columns subjected to flexure combined with compression and strengthened with CFRP wraps.
Test outcomes proved that composite wrapping is able to enhance concrete strength and ductility and to delay premature mechanisms of failure such as compressed bar buckling and concrete cover spalling. Strength improvement was found to be more relevant in the case of specimens loaded with a small eccentricity, while ductility improvement was more relevant in case of bigger eccentricity. Lignola et al. (2007b) also discussed the behavior of unstrengthened and FRP-jacketed square hollow concrete piers subjected to combined axial load and flexure uncoupled from shear focusing on the analysis of cross-section curvature, member deformability, specific energy, and model restraints. Based on the above-cited studies, Lignola et al. (2009a) proposed a computation algorithm to model the nonlinear behavior of RC hollow columns confined with FRP.

The objective of this study was to understand how the presence of the external FRP confinement impacts column performance. Specifically, the effect of confinement was investigated with respect to strength and deformability enhancement, prevention of instability of longitudinal bars, and confinement sensitivity to the confining reinforcement ratio.

**EXPERIMENTAL PROGRAM**

**Test matrix**

The test matrix is designed considering different factors, summarized in Table 1: shape factor (side-aspect ratio), volume factor (volume-aspect ratio based on a benchmark volume of 0.61×0.61×3.05 m³ solid column), type and amount of FRP sheet plies. The specimens are intended to represent scaled concrete piers designed according to dated codes (i.e., prior to 1970) for gravity loads only. In particular, the amount of longitudinal reinforcement and tie spacing are designed using the ACI 318-63 building code. Hollow column specimens are 3.05 m in length with cross-sectional dimensions of 508 by 737 mm and a wall thickness of 114 mm. Three specimens are considered: one unstrengthened RC column used as benchmark, and two columns confined with glass FRP (GFRP) reproducing a 5-ply and a 8-ply confinement ratio. The salient properties of the GFRP system are reported in Table 2.

<table>
<thead>
<tr>
<th>Specimen code</th>
<th>Cross-section geometry</th>
<th>Internal steel reinforcement</th>
<th>Shape Factor</th>
<th>Volume factor</th>
<th>Type of fibers</th>
<th>Yield of plies</th>
</tr>
</thead>
<tbody>
<tr>
<td>HR-control</td>
<td>0.51 by 0.74 m²</td>
<td>8 #8 longitudinal bars and #4 ties at a spacing of 406 mm</td>
<td>1.45</td>
<td>0.61</td>
<td>as-built benchmark specimen</td>
<td></td>
</tr>
<tr>
<td>HR-5G</td>
<td>corners chamfered with a radius of about 1 in</td>
<td></td>
<td></td>
<td></td>
<td>Glass</td>
<td>5</td>
</tr>
<tr>
<td>HR-8G</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Glass</td>
<td>8</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fiber type</th>
<th>Filament yarn properties</th>
<th>Laminate properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Glass fabric</td>
<td>E-Glass &amp; high-performance Glass</td>
<td>Thickness (mm) 0.25, Weight (g/m²) 600</td>
</tr>
</tbody>
</table>

**Instrumentation and test procedure**

The instrumentation in all the specimens consists of electrical strain gauges located on the longitudinal and transverse steel reinforcement, and on the FRP jacket at critical locations (corner areas and mid-section on each face of the prismatic specimens) along the perimeter of the cross-section, at the mid-height section of the strengthened specimens. Additionally, a total of ten linear variable differential transformer (LVDT) sensors were used to measure the vertical displacement of the specimen, and to evaluate the horizontal (in-plane) dilation at the mid-height cross-section, along the two side and the two diagonal directions. The load is applied concentrically under a displacement control rate of 0.15 mm/min, and the load is applied in five cycles at increments of one fifth of the expected capacity of each specimen.

**RESULTS AND DISCUSSIONS**
The control specimen failed abruptly due to longitudinal steel buckling and concrete cover splitting. Failure happened before the longitudinal steel reinforcement reached the yield point. Both the FRP-confined specimens failed due to rupture of the FRP jacket localized in a limited region in the proximity of a corner. The peak load was reached at a strain level close to yielding of the steel reinforcing bars. Cracking of the concrete progressed during the post-peak phase, while longitudinal bar buckling and concrete cover splitting were significantly restrained by the GFRP jacket. In Figure 1a, Figure 2a and Figure 3a, the axial load is plotted versus the axial deformation for specimens HR-control, HR-5G and HR-8G, respectively. Photographs in Figure 1b-d, Figure 2b-d and Figure 3b-d show the failed specimens.
The test results are summarized in Table 3. The following is reported: average concrete compressive strength, $f_c$; maximum load applied to the column specimen, $F_{c,peak}$; average axial concrete stress (defined as the ratio between the maximum applied load and the gross cross sectional area), $\sigma_{c,peak}$; normalized axial stress (defined as the ratio between the average axial stress and the average concrete compressive strength), $\sigma_{c,peak}/f_c$; ratio between the average axial confined concrete stress with respect to the control specimen for the reference series, $\sigma_{c,p}/\sigma_{c,pc}$; axial deformation at the peak load, $\Delta_{c,peak}$; post-peak axial deformation at failure, $\Delta_{c,max}$; ratio between the average axial stress at failure and the average concrete compressive strength; ratio between the post-peak axial deformation and the axial deformation at peak, $\Delta_{c,max}/\Delta_{c,peak}$. Figure 4 also shows the plot of the normalized axial stress with respect to the axial deformation for all the specimens.
Table 3 – Test results

<table>
<thead>
<tr>
<th>Spec. ID</th>
<th>$f_c$ [MPa]</th>
<th>$F_{c,peak}$ [kN]</th>
<th>$\sigma_{c,peak}$ [MPa]</th>
<th>$\sigma_{c,peak}/f_c$</th>
<th>$\sigma_{c,peak}/\sigma_{c,p}$</th>
<th>$\Delta_{c,peak}$ [mm]</th>
<th>$\Delta_{c,max}$ [mm]</th>
<th>$\sigma_{c,peak}/f_c$</th>
<th>$\Delta_{c,max}/\Delta_{c,peak}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>HR-control</td>
<td>43.0</td>
<td>7,894</td>
<td>34.6</td>
<td>0.805</td>
<td>---</td>
<td>5.59</td>
<td>5.91</td>
<td>0.795</td>
<td>1.06</td>
</tr>
<tr>
<td>HR-5G</td>
<td>44.4</td>
<td>8,439</td>
<td>37.0</td>
<td>0.832</td>
<td>1.033</td>
<td>6.30</td>
<td>18.3</td>
<td>0.529</td>
<td>2.90</td>
</tr>
<tr>
<td>HR-8G</td>
<td>46.7</td>
<td>9,208</td>
<td>40.4</td>
<td>0.864</td>
<td>1.073</td>
<td>5.08</td>
<td>16.4</td>
<td>0.616</td>
<td>3.23</td>
</tr>
</tbody>
</table>

No significant increment in concrete strength due to confinement has been observed. The normalized axial stress is 0.805 for the control specimen and varies between 0.832 and 0.864 for the confined ones. The increment in concrete strength due to confinement evaluated with the respect to the corresponding control specimen ranges between 3.3% and 7.3%. On the other hand, a significant improvement in post-peak deformability was experienced by the two confined specimens, as the ratio between axial deformation at failure and peak axial deformation is 2.90 and 3.23 for specimens HR-5G and HR-8G, respectively.

CONCLUSIONS

Based upon the experimental evidence gained through the full-scale experiments presented in this paper, the following conclusions are drawn.

- Effective confinement of hollow-core concrete columns with GFRP composite laminates does not result in any significant enhancement of the compressive strength.
- FRP jackets provide excellent confinement, thereby enhancing deformability by restraining buckling of the vertical reinforcing bars, and delaying concrete cover splitting.

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