REPAIR OF DAMAGED PRECAST RC BRIDGE COLUMNS WITH GROUTED SPLICE SLEEVE CONNECTIONS USING CFRP SHELLS AND PLASTIC HINGE RELOCATION

J.E. Parks¹, D.N. Brown¹, M.J. Ameli¹, C.P. Pantelides² and L.D. Reaveley²

ABSTRACT

A repair technique for damaged precast reinforced concrete (RC) bridge columns with grouted splice sleeve (GSS) connections has been developed that utilizes prefabricated carbon fiber-reinforced polymer (CFRP) shells and epoxy anchored headed mild steel bars to relocate the column plastic hinge. Undamaged columns with two different GSS systems were tested to failure using cyclic quasi-static loads. One GSS system was used to connect an RC bridge pier cap to a column. The second GSS system was used to connect an RC footing to a column. Failure of the two original specimens occurred at drift ratios between 6% and 7% with longitudinal bars fracturing in the column plastic hinge region. The column plastic hinge region was then repaired by increasing the column cross section from a 21 in. octagonal section to a 30 in. diameter circular section with an 18 in. length. The repair was constructed using prefabricated CFRP shells. Headed mild steel bars were epoxy anchored into the pier cap and footing inside the CFRP shells. Nonshrink concrete was used to fill the void between the original columns and CFRP shells. Compared to the undamaged assemblies, the repaired specimens failed at the same drift ratios and greater ultimate load values. The plastic hinge was successfully relocated to the original column section adjacent to the repair and the failure mode was bar fracture in the relocated plastic hinge region. The method is a viable technique for seismic repair or seismic retrofit of precast GSS assemblies in column to footing or column to pier cap connections.

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ABSTRACT

A repair technique for damaged precast reinforced concrete (RC) bridge columns with grouted splice sleeve (GSS) connections has been developed that utilizes prefabricated carbon fiber-reinforced polymer (CFRP) shells and epoxy anchored headed mild steel bars to relocate the plastic hinge in the column. Undamaged columns with two different GSS systems were subjected to cyclic quasi-static loading and tested to failure. One GSS system was used to connect an RC bridge pier cap to a column. The second GSS system was used to connect an RC footing and column. Failure of the two original columns occurred at drift ratios between 6% and 7% with the extreme longitudinal bars fracturing in the plastic hinge region of the column in both cases. The plastic hinge region of the columns was then repaired by increasing the column cross section from a 21 in. octagonal section to a 30 in. diameter circular section over an 18 in. length. The repair was constructed using prefabricated CFRP shells. Headed mild steel bars were epoxy anchored into the pier cap and footing inside the CFRP shells. Nonshrink concrete was used to fill the void between the original columns and CFRP shells. Compared to the undamaged assemblies, the repaired specimens failed at the same drift ratios and greater ultimate load values. The plastic hinge was successfully relocated to the original column section adjacent to the repair and the failure mode was bar fracture in the relocated plastic hinge region. The method is a viable technique for seismic repair or seismic retrofit of precast GSS assemblies in column to footing or column to pier cap connections.

Introduction

Repairing damaged bridge elements following an earthquake is a viable alternative to replacement. The benefits include cost savings, reduction in construction time and decreased interruption for emergency services and the general public. The objective of bridge repair is to rehabilitate the damaged bridge elements to a performance level similar to the original performance by restoring the load and displacement capacity. Current bridge design philosophy promotes damage to columns; hence, the post-seismic repair studied is focused on column repair. In the past, repair techniques for damaged bridge columns included the use of externally bonded CFRP sheets [1], steel jacketing [2] and concrete jacketing [3]. However, until recently it has been assumed that when longitudinal bars within the column buckle or fracture the column must be replaced [4].

Accelerated Bridge Construction (ABC) is gaining acceptance because of reduced construction time and minimal traffic interruption. Recently, Grouted Splice Sleeves (GSS) have been gaining attention as a possible precast concrete connection method for ABC in seismic regions. Research that is currently being conducted is investigating the performance of GSS connections for bridges built in seismic regions [5, 6]. The use of GSS connections in seismic regions is anticipated to begin shortly and a practical post-earthquake repair is needed to accompany this new technology. Findings for this current GSS research indicate that columns connected using GSS concentrate the column damage and decrease the effective plastic hinge
length compared to traditional monolithic construction. Both of these damage characteristics are advantageous for repair purposes, leaving a relatively undamaged column section for plastic hinge relocation.

The repair procedure developed has been designed and implemented on two different precast specimens connected using GSS. The repair uses materials that are readily available and easy to install including epoxy anchored headed bars, CFRP shells and nonshrink concrete. The result is a very cost effective and rapid repair procedure, which could be completed in a few days. Due to the robust nature of the repair it is a suitable option for columns of varying damage states.

**Original Test Specimens**

Two precast RC specimens representing half-scale bridge elements, conforming to current seismic bridge design standards [7], were constructed utilizing two different GSS systems. Specimen NM-O is a column-to-footing assembly where the GSS system uses high strength nonshrink grout on both ends of the connection to splice the bars from the footing and column. Specimen LE-O is a column-to-pier cap assembly where the GSS system has a threaded connection on one end and a grouted connection on the other.

**Original Specimen Reinforcement and Geometry**

The geometry and reinforcement of the columns for both NM-O and LE-O are the same, as shown in Fig. 1. The columns are 8.5 ft. tall with a 21 in. wide octagonal cross section. The longitudinal reinforcement consists of 6#8 grade 60 bars arranged in a circular pattern which extend out of the column concrete for a length of 7 in. This 7 in. length is inserted into the GSS in the footing or pier cap and grouted for the connection. A #4 grade 60 spiral at a 2.5 in. pitch is provided for transverse reinforcement. The GSS in both cases are outside the plastic hinge region of the column and are located in the footing and pier cap for NM-O and LE-O, respectively. The footing is 6 ft. long, 2 ft. deep and 3 ft. wide, as shown in Fig. 1 (a). The pier cap is 9 ft. long, 2 ft. deep and 2 ft. wide, as shown in Fig. 1 (b). The material properties for the RC components and the repair are given in Table 1.

**Testing Assembly and Loading Protocol**

Columns in bridges subjected to seismic ground motion experience double curvature bending between the pier cap and the footing in the transverse direction. The test assembly, shown in Fig. 2, applies a lateral load at a point that represents the inflection point of a real bridge column. It should be noted that the pier cap specimen was tested upside down, with the pier cap on the strong floor, for ease of testing. The loading consisted of a constant axial load and a displacement controlled cyclic quasi-static lateral load. The axial load applied was 6% of the axial load capacity of the column.

The lateral load was applied following the loading protocol shown in Fig 2 (c). Two cycles per drift ratio were used and the amplitude was progressively increased until a 20% drop in the lateral load capacity was reached [8].
The pre-existing damage state of the original specimens is a critical parameter for the repair design and subsequent performance. The initial test results of NM-O and LE-O are summarized in Table 2 in terms of maximum lateral load, ultimate drift ratio, displacement ductility, and failure mode. The failure mode for both NM-O and LE-O was fracture of an extreme longitudinal bar. The extreme east longitudinal bar fractured in NM-O and the extreme west longitudinal bar fractured in LE-O. Once the extreme longitudinal bar fractured, the lateral load capacity of the specimens dropped below 20% of the ultimate load capacity. A very well developed plastic hinge was formed at the footing-to-column and column-to-pier cap interfaces, as shown in Fig. 3, where extensive spalling and cracking occurred. For both specimens major structural cracking was isolated to three distinct heights from the footing or pier cap interface, where the highest crack was located approximately 14 in. up the column for NM-O and 12 in. up the column for LE-O. These crack widths ranged from 0.016 in. to 0.060 in.
Figure 2. Test setup: (a) NM-O; (b) LE-O; (c) loading protocol.

Table 2. Original specimen test results.

<table>
<thead>
<tr>
<th>Test Criteria</th>
<th>NM-O</th>
<th>LE-O</th>
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<tbody>
<tr>
<td>Max Lateral Load (kips)</td>
<td>38.8</td>
<td>37.7</td>
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<tr>
<td>Ultimate Drift Ratio (%)</td>
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<td>Displacement Ductility</td>
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<tr>
<td>Failure Mode</td>
<td>Bar Fracture</td>
<td>Bar Fracture</td>
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</table>
Repair Design and Procedure

The objective of the repair was to strengthen the original plastic hinge region through increasing the cross section from a 21 in. octagonal section to an additionally reinforced 30 in. diameter circular section. The 30 in. diameter circular cross-section was constructed by filling prefabricated carbon fiber reinforced polymer (CFRP) shells with nonshrink concrete and post-installing epoxy anchored headed bars for additional tensile reinforcement, as shown in Fig. 4. To form the new plastic hinge, a bending moment referred to as, \( M_{PH} \), must be resisted at the desired plastic hinge location. \( M_{PH} \) can be determined from moment curvature analysis or by recording the ultimate moment capacity of the original column during testing. From Eq. 1, the bending moment experienced at the column joint, \( M_{Joint} \), is proportional to the length of the repair, \( h_{repair} \), and the height of the column from the point of inflection to the column-footing or column-pier cap joint, \( h_{col} \).

\[
M_{Joint} = \frac{M_{PH}}{1 - \frac{h_{repair}}{h_{col}}}
\]  

(1)

Therefore, using the minimum possible repair height is advantageous for limiting the moment demand at the repair joint and for decreasing the rotational demand on the column for a given displacement. On the other hand, the height of the repair must be long enough relocate the new plastic hinge to a minimally damaged cross-section.
Headed bars were designed to withstand the new increased joint moment produced by the repair. The headed bar length drilled into the footing or pier cap was determined so that the epoxy anchorage would develop the bars in tension. Similarly, the length of headed bar extending into the repair was checked for adequate development length. These parameters led to the design of 6 #8 grade 60 headed bars which were post installed around the column as shown in Fig 4.

CFRP shells were designed to provide confinement and shear strength, and were used as stay-in-place formwork for the nonshrink concrete. Four layers of unidirectional CFRP sheets oriented in the hoop direction were provided where one layer was provided as a shell to wrap subsequent layers of CFRP around, one layer was provided for shear [1], and two layers were provided for confinement and prevention of strain softening ([9], [10]). A 0.5 in. gap was left between the bottom of the jacket and footing or pier cap surface to ensure there was no bearing of the CFRP shell on the concrete during large displacements.

The first step in the repair was to create the prefabricated CFRP shells. A single layer of 18 in. wide CFRP sheet was wrapped and cured around a 30 in. diameter sonotube to create the proper shape. While the CFRP shells were curing, the holes for the post installed headed bars were core drilled and the headed bars were epoxy anchored into place around the column, as shown in Fig. 5 (a). After the CFRP shells had cured they were split in two half cylinders and brought around the column as shown in Fig. 5 (b). The sonotube inside the shell in Fig. 5 (b) was used to ensure that the shell maintained its shape, while the additional layers of CFRP were applied; it was removed once all CFRP layers had cured. With the split CFRP shell around the column a 12 in. long by 18 in. wide piece of CFRP sheet was used to splice the two halves of the CFRP shell. Once the first layer of the CFRP shell was spliced, the additional 3 layers of CFRP were added as shown in Fig. 5 (c). This was the last step in completing the construction of the CFRP shell which acted as stay-in-place formwork for the repair. Once the CFRP shells had fully cured, nonshrink concrete was added to the space between the column and CFRP shells as shown in Fig. 5 (d). The nonshrink concrete was cured for at least 28 days before testing.

For LE-O, the diameter of the repair was larger than the width of the pier cap. Wooden forms were placed alongside the pier cap in order to provide sufficient width for the repair as shown in Fig. 5 (b), (c). The wooden forms were then removed once the nonshrink concrete had cured. In practice, the pier cap would be oriented above the column and the gap between the repair and pier cap would provide an inlet for the nonshrink concrete and the gap between the column and the repair would need to be sealed.

**Repairs Specimen Test Results**

Since the original results from both NM-O and LE-O were similar, the repair design and procedure described previously was used for both specimens. The repairs of NM-O and LE-O are referred to as NM-R and LE-R, respectively. Both the test assembly and the loading protocol used to test the original specimens were used to test the repaired specimens. Plastic hinge relocation was successfully achieved for both NM-R and LE-R, as shown in Fig. 6.
Specimen NM-R

The hysteretic response of NM-R is shown in Fig. 7 (a) with the hysteretic response of NM-O superimposed. The failure mode for NM-R was fracture of the extreme west column longitudinal bar followed by fracture of the extreme east column longitudinal bar during the same displacement step. It should be noted that the east longitudinal bar fractured only 21.5 in. above the original fracture location in NM-O. This implies that the repair scheme provided sufficient confinement and clamping force to develop the longitudinal bar in a shorter distance than expected. Other major events during the test included the onset of significant spalling at a 3% drift ratio and transverse CFRP cracking occurring at a drift ratio of 4%. The transverse CFRP cracking was located approximately 3 in. below the top of the repair, adjacent to the top of the headed bars, and extended half way around the jacket circumference on the east side.
The hysteretic response of the specimen remained seemingly unaffected from the transverse crack in the CFRP shell.

To further examine the performance of NM-R, the hysteretic energy dissipation and stiffness degradation characteristics were investigated and compared to NM-O, as seen in Fig. 8 (a) and (b), respectively. The cumulative energy dissipation of NM-R is slightly greater than NM-O for all drift ratios where at the completion of the 6% drift ratio NM-R has dissipated 15% more energy than NM-O. Similarly, the stiffness degradation characteristics of NM-R and NM-O are very similar when normalized to the 0.5% drift ratio stiffness. The normalized stiffness of NM-R is slightly larger than NM-O at all drift ratios and thus the rate of stiffness degradation is slightly lower. The normalization was carried out to portray the degradation of stiffness rather than the numeric stiffness value because NM-R will always have a higher stiffness due to the shorter column length. Both of these parameters further confirm that the repair restored the assembly to a performance level similar to the original condition.
Specimen LE-R

It was originally planned to test LE-R following the loading protocol in Fig. 2, but due to operator error the column was unexpectedly pushed to the east monotonically to a drift ratio of 6.9% before the actuator could be stopped. The monotonic pushover curve from this event is shown in Fig. 7 (b). Although the column was displaced to a drift ratio beyond the ultimate drift ratio of LE-O, no longitudinal bars fractured in the column. There was major spalling on the east side of the column, as shown in Fig. 6 (d), which extended to a 20 in. height up the column and exposed the spiral reinforcement.

Even with the repaired column already damaged in one direction from the monotonic pushover test, the specimen was subsequently tested following the loading protocol of Fig. 2. The hysteretic response of LE-R is shown in Fig. 7 (b) with the hysteretic response of LE-O superimposed. The right side of the hysteresis for LE-R shows an irregular response due to the previous damage from the monotonic pushover. The left side of the hysteresis for LE-R however, seems to be less affected. The failure mode for LE-R was fracture of the extreme east column longitudinal bar. Similar to the behavior of NM-R, the onset of significant spalling on the west side of the column occurred at a drift ratio of 3% and the onset of transverse CFRP cracking occurred at a drift ratio of 4%. The transverse CFRP cracking was located approximately 3 in. below the top of the repair, correlating to the top of the headed bars, and extended half way around the jacket circumference on the west side. Although the hysteretic response in the west direction was irregular, the specimen remained seemingly unaffected from the transverse crack.

Table 3 shows a comparison of all the tests. In all cases, the repaired specimens were able to regain the strength of the original specimens while still performing in a ductile manner. For the case of NM-R a 15% increase in the maximum lateral load was obtained while still maintaining the ultimate drift ratio capacity and displacement ductility. For the case of LE-R it is difficult to make direct comparisons to LE-O, because of the initial static pushover test. However, by examining the performance of LE-R from both the static pushover and cyclic tests, it is clear that it performed at least as well as LE-O.

Conclusions

A rapid repair procedure tailored to post-earthquake damage has been developed for severely damaged bridge columns connected using GSS. The repair converts the original plastic hinge region from a 21 in. octagonal section to a 30 in. diameter circular section thereby relocating the

<table>
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<th>LE-O</th>
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new plastic hinge to a minimally damaged section adjacent to the repair. The repair extends over the column height for a length sufficiently long to cover the original plastic hinge region and is reinforced with headed bars and a CFRP shell. This repair procedure was implemented and tested for previously damaged bridge column-to-footing and column-to-pier cap joints; it successfully restored the diminished performance of the specimens in terms of displacement capacity, load capacity, energy dissipation and stiffness. The tested repair method is a technique for seismic repair or retrofit of precast GSS columns in column-to-footing or column-to-pier cap connections.

Acknowledgments

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References