Evaluation and repair of precast RC bridge column connections utilizing grouted splice sleeves

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ABSTRACT: The research described in this paper utilizes Grouted Splice Sleeves (GSS) to connect precast reinforced concrete (RC) columns to footings and cap beams. An evaluation of the performance of the as-built and repaired connections is described. Columns with two different GSS systems were subjected to cyclic quasi-static loading to failure. One GSS system (FGSS) connected a cap beam to a column using mild steel bars threaded into the sleeve at one end and grouted at the other. The second GSS system (GGSS) connected a footing to a column using mild steel bars grouted at both ends of the sleeve. Two column-to-footing and two column-to-cap beam connections were tested. Splice sleeves were located in the column ends for each connection for two of the specimens; for the other two specimens the sleeves were located inside the footing and inside the cap beam, respectively. The performance of all four test specimens was satisfactory with the GGSS specimens performing at a higher level. A repair technique utilizing a prefabricated carbon fiber-reinforced polymer (CFRP) shell and headed mild steel bars was used to relocate the plastic hinge in the column for two of the as-built specimens after they had been tested and severely damaged. The plastic hinge of the as-built columns was repaired by increasing the column cross section and changing its shape from an octagon to a circle. The CFRP shell provided confinement and acted as concrete formwork. Inside the CFRP shell, headed mild steel bars were epoxy anchored into the cap beam and footing to increase the flexural capacity of the plastic hinge region. Nonshrink concrete was used to fill the void between the as-built columns and CFRP shells. The plastic hinge was successfully relocated to the column section adjacent to the repaired section, thus restoring the strength and displacement capacity of the specimens.

KEY WORDS: Bridge; column; concrete; cyclic; FRP composite; precast; retrofit; repair; seismic; sleeves; tests.

1 INTRODUCTION

Prefabrication of bridge elements and systems is one of the accelerated bridge construction methods. Connections between prefabricated elements are critical in bridges constructed using Accelerated Bridge Construction (ABC). NCHRP report 698 evaluated several connection types applicable to ABC, for moderate-to-high seismic regions [1]. The Grouted Splice Sleeve (GSS) connection was classified as a practical and promising connection requiring more research regarding both strength and displacement capacity. The inelastic behavior of such connections under cyclic loads and sensitivity of the response to sleeve locations were highlighted as issues in need of research. Three specimens, two of which had GSS connections while the third specimen was cast monolithically have recently been tested [2]; the specimens exhibited similar performance in terms of ultimate load capacity and energy dissipation, but not ductility capacity.

Repair of damaged bridge elements following an earthquake is a beneficial alternative to replacement of the damaged members. The benefits include cost savings, reduction in construction time and decreased interruption for emergency services and the public. The objective of bridge repair is to rehabilitate damaged bridge elements to a performance level similar to the original performance by restoring the strength and displacement capacity. Current bridge design philosophy promotes column yielding; because of the damage caused in the as-built tests, the post seismic repair studied is focused on column repair. Retrofit and repair techniques for bridge columns include the use of steel jackets [3], bonded CFRP jackets [4], and reinforced concrete jackets [5]. Recently, CFRP composites were used to repair damaged circular concrete columns using plastic hinge relocation [6].

In the present study, four half-scale specimens were tested under cyclic quasi-static displacements to failure. Two column-to-footing specimens incorporated a GSS where the bars were grouted at both ends (GGSS); two column-to-cap beam specimens used a different GSS type where one bar was threaded into one end of the sleeve and the other bar was grouted into the opposite end (FGSS). Fig. 1 shows the two types of GSS connections utilized to construct the test specimens. Subsequently, a repair procedure was implemented for two damaged specimens: one constructed using a GGSS and one with a FGSS. The repair method developed in this research utilizes materials that are readily available and easy to install including epoxy anchored headed steel bars, unidirectional CFRP sheets and nonshrink concrete. The result is a very cost effective and rapid repair procedure.

2 EXPERIMENTAL PROGRAM FOR AS-BUILT SPECIMENS

Half-scale models were designed in accordance with the principles of capacity based design, to simulate typical prototype bridges built in the State of Utah. Fig. 2 shows the specimen details. The octagonal columns were built using six 25 mm longitudinal bars in a circular arrangement, confined with a 13 mm spiral. The longitudinal and transverse
reinforcement ratio was 1.3% and 1.9%, respectively. The footings and cap beams were reinforced using 25 mm longitudinal bars and 13 mm double hoops; they were designed to remain elastic as capacity-protected members. The design criteria followed the AASHTO LRFD Bridge Design Specifications and the AASHTO Guide Specifications for LRFD Seismic Bridge Design [7, 8]. The latter prohibits splicing of the column longitudinal bars inside the plastic hinge region for bridges in high-seismic zones. The response of the subassemblies was studied by changing the location of the splice sleeves. Splice sleeves were placed inside the column ends for specimens GGSS-1 and FGSS-1, whereas for specimens GGSS-2 and FGSS-2, they were placed outside the column plastic hinge region and inside the footing and cap beam, respectively. In Fig. 2, the width of the footing for specimens GGSS-1 and GGSS-2 was 91 cm; the width of the cap beam for specimens FGSS-1 and FGSS-2 was 61 cm. The average compressive strength of the concrete on test day was 39 MPa and that of the grout for the sleeves was 89 MPa.

3 TEST RESULTS FOR AS-BUILT SPECIMENS
Cyclic quasi-static lateral displacements comprised of ascending amplitudes of the predicted column yield displacement were applied as illustrated in Fig. 3, with two cycles at each displacement [9]. Specimens FGSS-1 and FGSS-2 were tested upside down. An axial load, equivalent to 6% of the column axial capacity, was applied by means of a hydraulic actuator using post tensioning rods. Linear variable differential transformers mounted on opposite sides of the column measured vertical displacements at the interface of the column with the footing or cap beam. String potentiometers captured the lateral displacement of the column. The specimens were instrumented with several strain gauges in the plastic hinge region of the column and in the joint area.

The hysteresis loops of the four specimens demonstrated a hysteretic behavior in which bond-slip of the spliced bars was present. Fig. 4 shows the lateral force-displacement curves along with the major damage states, which were: end of crack formation and initiation of spalling, yield penetration, bar fracture, and bar bond failure and pullout. Specimens GGSS-1 and GGSS-2 exhibited full and stable loops during all cycles as opposed to FGSS-1 and FGSS-2, which experienced some pinching, associated with the grout crushing inside the sleeves and bar bond-slip during the last few cycles. Ductile failure was observed for GGSS-1 and GGSS-2, while FGSS-1 and FGSS-2 failed due to bond failure and pullout. Only flexural cracks developed in the columns of specimens with splice sleeves in the columns, GGSS-1 and FGSS-1, whereas both flexural and inclined cracks formed in the columns of specimens with sleeves in the footing and cap beam, i.e. GGSS-2 and FGSS-2. There were very few minor cracks in the cap beams and no cracks in the footings. All major flexural cracks formed by the end of the 3% drift ratio, in addition to considerable concrete spalling. Cracks opened further and concrete spalling intensified after the 3% drift ratio up to failure. Yield penetration occurred during the 6% drift ratio for GGSS-1 at the two extreme column bars. The
Figure 4. Force-Displacement response with damage states.

The test was terminated at the end of the 9% drift ratio when both bars fractured. The test for FGSS-1 was terminated at the end of the 6% drift ratio because the column bars pulled out of the sleeves, as a result of bond failure associated with the grout crushing inside the sleeves. A few inclined cracks developed during the 4% and 5% drift ratio for GGSS-2. This specimen failed when the east column bar fractured during the first cycle of the 7% drift ratio. A combined failure mode was observed for FGSS-2. At the 7% drift ratio, the extreme west column bar fractured during the first push cycle; the lateral force dropped in the subsequent pull cycle when the extreme east column bar pulled out of the sleeve. Fig. 5 shows the final damage state for all as-built test specimens.

The displacement ductility of the specimens was obtained based on the concept of equal energy of an idealized elasto-plastic system [10]. The average backbone curve was constructed using the peak values of the first cycle for each drift ratio. The idealized elasto-plastic curve was then generated in order to calculate the displacement ductility. The yield displacement was found using a secant stiffness that includes the smaller of first yield of column bar or 70% of the maximum idealized force. The ultimate displacement was taken as the displacement corresponding to a 20% or larger drop in the lateral force [11]. For the specimens with sleeves in the columns, GGSS-1 and FGSS-1, the displacement ductility was 5.9 and 4.4, respectively; for specimens with sleeves in the footing and cap beam, GGSS-2 and FGSS-2, the displacement ductility improved to 6.1 and 5.8, respectively.

The area enclosed by the hysteresis loops is the hysteretic energy of the system. Fig. 6(a) compares the hysteretic energy per drift level for all specimens. This is an indication of the quality of the hysteretic response and load-displacement characteristics. All specimens dissipated nearly the same amount of energy up to a 3% drift ratio. Specimens GGSS-1 and GGSS-2 demonstrated better hysteretic energy dissipation during the subsequent cycles up to a 6% drift ratio.

The effective stiffness was calculated in each cycle using the peak displacement and corresponding force. The average of the stiffness values was then obtained for both cycles at each drift ratio. Fig. 6(b) displays the average effective stiffness at each drift level for all specimens. A similar trend was observed in the stiffness reduction of all specimens. The degradation rate was much higher during the early cycles mainly because of column bar yielding. For example, there was a 60% reduction in stiffness for specimen GGSS-2 by the end of the 2% drift ratio. It is evident that specimens GGSS-1 and FGSS-1 had a slightly higher stiffness than the other two specimens, at every drift ratio.

Figure 5. Final damage state for as-built specimens.
EXPERIMENTAL PROGRAM FOR REPAIRED SPECIMENS

The objective of the repair was to strengthen the plastic hinge region by increasing the cross section from a 53 cm octagonal section to a 76 cm diameter circular section. The circular cross-section was constructed by post-installing epoxy anchored headed bars for additional tensile reinforcement, and filling prefabricated carbon fiber reinforced polymer (CFRP) shells placed around the column with nonshrink concrete, as shown in Fig. 7. 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 bending moment capacity of the as-built column during testing. In 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-cap beam joint interface, $h_{col}$.

$$M_{Joint} = \frac{M_{PH}}{\left(1 - \frac{h_{repair}}{h_{col}}\right)}$$  \hspace{1cm} (1)

Using the minimum repair height possible is advantageous for limiting the bending 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 sufficient for the purpose of relocating the new plastic hinge to an adjacent column cross-section with minimal damage.

Headed bars were designed to develop the tension required for the enlarged cross-section to withstand the new increased joint bending moment produced by the repair. The headed bar length drilled into the footing or cap beam 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 six 25 mm headed bars of 400 MPa yield strength, as shown in Fig. 7. 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 served as a shelf to wrap subsequent layers of CFRP around, one layer was provided for shear [4], and two layers were provided for confinement and prevention of strain softening [12, 13]. A 13 mm gap was left between the bottom of the jacket and footing or cap beam surface to ensure that there was no bearing of the CFRP shell on the concrete during large displacements.

The first step in the repair procedure was to create the prefabricated CFRP shells. A single layer of a 46 cm-wide unidirectional CFRP sheet was wrapped and cured around a 76 cm diameter sonotube to create the proper shape. Holes were core drilled and headed bars epoxy anchored around the column (Fig. 8(a)). After the CFRP shells had cured they were split into two half cylinders (Fig. 8(b)). A 30 cm long by 46 cm wide piece of CFRP sheet was used to splice the two halves of the CFRP shell. Subsequently, three CFRP layers were applied with the fibers in the hoop direction (Fig. 8(c)), and once the CFRP shell had cured, nonshrink concrete was added in the space between it and the column (Fig. 8(d)). The nonshrink concrete was cured for a minimum of 28 days.
As-built specimens GGSS-2 and FGSS-2, had developed a plastic hinge at the footing-to-column and column-to-cap beam interfaces, as shown in Fig. 5, where extensive spalling and cracking had occurred. For both specimens, major structural cracking was isolated to distinct heights from the footing or cap beam interface, where the highest crack was located approximately 36 cm up the column for GGSS-2 and 30 cm up the column for FGSS-2; the crack widths ranged from 0.4 to 1.5 mm. For FGSS-2, the diameter of the repaired cross-section was larger than the width of the as-built cap beam. Wooden forms were placed alongside the cap beam in order to provide sufficient width for the repair as shown in Figs. 8(b) and 8(c). The wooden forms were then removed once the nonshrink concrete had cured.

5 TEST RESULTS FOR REPAIRED SPECIMENS

Since the damage of GGSS-2 and FGSS-2 was similar, the design of the repair and the repair procedure described previously was used for both specimens. The repaired specimens for GGSS-2 and FGSS-2 are referred to as GGSS-2R and FGSS-2R, respectively. The test assembly and loading protocol used to test the as-built specimens were used to test the repaired specimens. Plastic hinge relocation was successfully achieved for both specimens, as shown in Fig. 9 and Fig. 10.

The hysteretic response of GGSS-2R is shown in Fig. 11(a) with the hysteretic response of as-built specimen GGSS-2 superimposed. Fracture of the extreme west column longitudinal bar followed by fracture of the extreme east column longitudinal bar during the same displacement step was the failure mode. It should be noted that the east longitudinal bar fractured only 55 cm above the original fracture location in GGSS-2. Thus, the repair scheme provided sufficient confinement and clamping force to develop the longitudinal bar in a shorter distance than expected. Major events during the tests included the onset of significant spalling at a 3% drift ratio and transverse CFRP cracking at a 4% drift ratio. The latter was located approximately 8 cm 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 was not affected by the transverse crack in the CFRP shell.

The hysteretic energy dissipation and stiffness degradation of GGSS-2R were investigated and compared to GGSS-2, as shown Fig. 12(a) and (b), respectively. The cumulative energy dissipation of GGSS-2R was slightly higher than GGSS-2 for all drift ratios; at the completion of the 6% drift ratio GGSS-2R had dissipated 15% more energy than GGSS-2. The stiffness degradation characteristics of GGSS-2R and GGSS-2 were very similar when normalized to the 0.5% drift ratio stiffness. The normalized stiffness of GGSS-2R is larger than GGSS-2 at all drift ratios and the rate of stiffness degradation is slightly lower. The normalization was carried out to show the stiffness degradation rather than the numeric stiffness values; this is because GGSS-2R has 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 as-built condition.

Due to operator error, FGSS-2R was unexpectedly pushed to the east monotonically to a drift ratio of 6.9%. The monotonic curve for this event is shown in Fig. 11(b) as a single line and labeled as “pushover”. Although the column was displaced to a drift ratio beyond the ultimate drift ratio of FGSS-2, no longitudinal bars fractured in the column.
was major spalling on the east side of the column, as shown in Fig. 10(b), which extended to a 51 cm height up the column and exposed the spiral reinforcement. Even with the column damaged on the east side from the monotonic pushover test, the specimen was subsequently tested following the cyclic loading protocol of Fig. 3. The hysteretic response of FGSS-2R is shown in Fig. 11(b) with the hysteretic response of the as-built specimen FGSS-2 superimposed. The right side of the hysteresis for specimen FGSS-2R shows an irregular response due to the damage caused by the monotonic pushover. The left side of the hysteresis for FGSS-2R however, seems to be less affected. The failure mode of the repaired specimen was fracture of the extreme east column longitudinal bar. Similar to the behavior of GGSS-2R, 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 at a section approximately 8 cm below the top of the repaired section, near the top of the headed bars, and extended half-way around the jacket circumference on the west side. Although the hysteretic response was irregular, the specimen remained unaffected by the transverse crack.

Table 1 shows a comparison of the performance of the as-built and repaired specimens. The repaired specimens were able to regain the strength of the as-built specimens while still performing in a ductile manner. For the case of GGSS-2R a 15% increase in the maximum lateral force was obtained while still maintaining the ultimate drift ratio and displacement ductility. For the case of FGSS-2R it is difficult to make direct quantitative comparisons to the as-built specimen FGSS-2, because of the initial static pushover test. However, by examining the performance of FGSS-2R from both the static pushover and subsequent cyclic test, it is clear that it performed at least as well as FGSS-2.

6 CONCLUSIONS

The hysteretic performance of the four as-built specimens was acceptable, with displacement ductility levels that satisfy code requirements. Thus, the GSS connection sleeves for column-to-footing and column-to-cap beam joints are deemed good candidates for implementation using Accelerated Bridge Construction methods in regions of moderate to high seismic activity. Specimens with sleeves inside the footing or cap beam achieved higher displacement ductility than the corresponding specimens with sleeves in the column. Ductile failure was observed for GGSS-1 and GGSS-2 with the grouted/grouted splice sleeves, while FGSS-1 and FGSS-2 with the threaded/grouted splice sleeves failed due to bond-slip of the spliced bars and bar pullout. Specimens GGSS-1 and GGSS-2 exhibited full and stable loops during all cycles as opposed to FGSS-1 and FGSS-2, which experienced some pinching, associated with grout crushing inside the sleeves.
was partially mitigated for specimen FGSS-2, in which the threaded/grouted splice sleeves were placed in the cap beam. A rapid repair technique tailored to address post-earthquake damage has also been developed for heavily damaged bridge columns connected using GSS. The repair converts the original plastic hinge region from a 53 cm octagonal section to a 76 cm diameter circular section with a height equal to 46 cm, thereby relocating the new plastic hinge to a minimally damaged section adjacent to the repaired section. The repair extends over the column height for a sufficient height to cover the original plastic hinge region and is reinforced with headed bars for tensile reinforcement and a CFRP shell for concrete confinement. This repair technique was implemented and tested for previously damaged bridge column-to-footing and column-to-cap beam joints; it successfully restored the performance of the specimens in terms of maximum displacements, lateral force capacity, energy dissipation and stiffness. The method is rapid and cost effective and is considered to be a successful technique for seismic repair or retrofit of precast GSS columns in column-to-footing or column-to-cap beam connections.

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