SEISMIC ANALYSIS OF REPAIRED RC BRIDGE COLUMN ASSEMBLIES USING ADVANCED COMPOSITES

R. Wu¹ and C. Pantelides²

ABSTRACT

This paper presents analytical models of the cyclic performance of three severely damaged column-to-cap beam connections repaired using a carbon-fiber-reinforced polymer (CFRP) cylindrical shell, headed steel bars, and non-shrink repair concrete. In one specimen cracks were sealed using epoxy injection; in another specimen, a steel collar with shear studs was installed to improve bond of the column concrete to the repair concrete. The bond between the original column and repair system is crucial in the overall structural performance of the repaired specimens, which can be improved significantly using the steel collar with shear studs. Two analytical models, Model Fiber and Model RS, are proposed in this paper. In Model Fiber, distributed plasticity considering bond slip is assumed to be concentrated in the plastic hinge length of the nonlinear beam-column element instead of the total length of the element; this was implemented using the BeamWithHinges element in OpenSees. In Model RS, concentrated plasticity was considered using a non-linear moment rotational spring located at the repaired cross-section. A sectional moment-curvature analysis is performed, based on the damaged steel properties and considering bond slip, to obtain the moment-rotation relationship, which is assigned to the non-linear rotational spring. The two analytical models proposed in this paper include low-cycle fatigue of longitudinal steel bars and bond-slip effects. Numerical simulations show that the analytical results, in terms of hysteretic response, cumulative hysteretic energy, and moment-rotation, simulate with the experimental results accurately. The comparisons between simulations and experimental results also indicate that Model Fiber performs better than Model RS in predicting the pinching effect in the hysteretic response of the repaired cast-in-place specimens; Model RS performs better than Model Fiber for matching the hysteresis curves of the repaired precast concrete specimens.

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Seismic Analysis of Repaired RC Bridge Column Assemblies Using Advanced Composites

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This paper presents analytical models of the cyclic performance of three severely damaged column-to-cap beam connections repaired using a carbon-fiber-reinforced polymer (CFRP) cylindrical shell, headed steel bars, and non-shrink repair concrete. In one specimen cracks were sealed using epoxy injection; in another specimen, a steel collar with shear studs was installed to improve bond of the column concrete to the repair concrete. The bond between the original column and repair system is crucial in the overall structural performance of the repaired specimens, which can be improved significantly using the steel collar with shear studs. Two analytical models, Model Fiber and Model RS, are proposed in this paper. In Model Fiber, distributed plasticity considering bond slip is assumed to be concentrated in the plastic hinge length of the nonlinear beam-column element instead of the total length of the element; this was implemented using the BeamWithHinges element in OpenSees. In Model RS, concentrated plasticity was considered using a non-linear moment rotational spring located at the repaired cross-section. A sectional moment-curvature analysis is performed, based on the damaged steel properties and considering bond slip, to obtain the moment-rotation relationship, which is assigned to the non-linear rotational spring. The two analytical models proposed in this paper include low-cycle fatigue of longitudinal steel bars and bond-slip effects. Numerical simulations show that the analytical results, in terms of hysteretic response, cumulative hysteretic energy, and moment-rotation, simulate with the experimental results accurately. The comparisons between simulations and experimental results also indicate that Model Fiber performs better than Model RS in predicting the pinching effect in the hysteretic response of the repaired cast-in-place specimens; Model RS performs better than Model Fiber for matching the hysteresis curves of the repaired precast concrete specimens.

Introduction

Seismic repair of damaged columns is preferable to replacement; benefits include rapid construction and decreased interruption of regular service in addition to cost-savings [1]. During

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strong earthquakes, longitudinal steel bars yield, buckle, and fracture in plastic hinge regions, which leads to loss of flexural and shear strength of reinforced concrete (RC) columns. Carbon fiber reinforced polymer (CFRP) composites are used for seismic rehabilitation because of their high-strength, light weight, and resistance to corrosion. A CFRP shell also serves as a stay-in-place concrete form and provides continuous confinement to the column concrete and corrosion protection for the steel reinforcement [2].

For severely damaged concrete columns, one seismic repair method utilizes a carbon fiber-reinforced polymer shell and epoxy anchored headed steel bars to relocate the column plastic hinge [3-4]. This repair method has been used for a severely damaged cast-in-place (CIP) column, with damage including severe concrete crushing and longitudinal bar buckling and fracture; the second specimen was a severely damaged precast specimen which suffered damage including extensive cracks and concrete crushing; for the third specimen which was precast concrete, the column was completely separated from the cap beam before repair. The seismic performance in terms of lateral force capacity, displacement capacity, and displacement ductility were successfully restored. Bond slip failure occurs in most RC concrete structures, especially for poorly confined concrete joints [5]. Considering the severe damage of concrete columns, bond-slip between the damaged longitudinal steel bars and surrounding concrete is critical for determining the structural behavior accurately. Damaged steel properties could be implemented with reduction of the elastic modulus according to the maximum strain experienced by the steel bar [6]. To consider bond-slip behavior in analysis, researchers have developed several models with modified steel properties [7–9].

There is little research regarding how to consider bond-slip, longitudinal bar fracture/buckling, and damaged concrete in analytical models of repaired specimens. In addition, the low-cycle fatigue of damaged longitudinal steel bars should be considered in cyclic analysis. Two analytical models are proposed in this paper to address the aforementioned factors for the analysis of the repaired specimens. The bond-slip effect was considered in the distributed plasticity element and concentrated rotational spring element, respectively. Comparisons between numerical simulations from the two analytical models and experimental results in terms of hysteresis curves and dissipated energy are presented. The benefits and overall accuracy of the proposed analytical models are also discussed.

**Review of Original and Repaired Specimens**

**Original Specimens**

Three specimens, referred to as CIP-O, PC1-O, and PC2-O, respectively were designed based on current seismic bridge design standards [10-11]. The three specimens are cap beam-to-column connections and letter O stands for original and R for repaired; CIP stands for cast-in-place concrete and PC for precast concrete. The repaired specimen of CIP-O is referred to as CIP-R; the first and second repair of PC1-O are referred to as PC1-R1 and PC1-R2, respectively; the repaired specimen of PC2-O is referred to as PC2-R. Considering the previous damage and theoretical plastic hinge length, a 19 in. (483 mm) CFRP shell with seven layers in the hoop direction and two layers in the vertical direction was used for the repaired specimens CIP-R, PC1-R2, and PC2-R [12-13]. Six headed steel bars were provided to meet the flexural moment...
demand at the repaired section. Since the column and cap beam were completely separated in specimen PC2-R, a steel collar with steel studs was provided around the column to increase the bond between the column and repair concrete. The repair design details can be found elsewhere [12-13].

The experimental results for the original and repaired specimens are summarized in Table 1. For the repaired specimens, the plastic hinge was successfully relocated above the repaired section, as shown in Figure 1. For CIP-R, concrete crushing extended 18.5 in. (470 mm) above the repair section and 3.0 in. (76 mm) inside the CFRP shell. For PC1-R2, concrete crushing occurred in the column up to 14 in. (343 mm) above the CFRP donut and 1.0 in. (25 mm) inside the CFRP shell. For PC2-R, at a drift ratio of 8.7%, the specimen failed due to bar fracture on both the west and east side with damaged regions of the column up to 24 in. (610 mm) above the CFRP donut on the east side and 9.0 in. (229 mm) on the west side.

![Figure 1. Plastic hinge relocation: (a) CIP-R; (b) PC1-R2; (c) PC2-R.](image)

<table>
<thead>
<tr>
<th>Description of Two Analytical Models</th>
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**Table 1. Original and repaired specimen test results.**

<table>
<thead>
<tr>
<th></th>
<th>CIP-O</th>
<th>CIP-R</th>
<th>PC1-O</th>
<th>PC1-R1</th>
<th>PC1-R2</th>
<th>PC2-O</th>
<th>PC2-R</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Averaged maximum lateral force, kip (kN)</strong></td>
<td>37.8 (168)</td>
<td>45.6 (203)</td>
<td>41.0 (182)</td>
<td>49.9 (222)</td>
<td>53.4 (238)</td>
<td>39.7 (177)</td>
<td>48.8 (217)</td>
</tr>
<tr>
<td><strong>Ultimate drift ratio, (%)</strong></td>
<td>9.3</td>
<td>8.1</td>
<td>6.7</td>
<td>5.6</td>
<td>7.8</td>
<td>5.5</td>
<td>7.6</td>
</tr>
<tr>
<td><strong>Failure mode</strong></td>
<td>East and west bar fracture</td>
<td>Severe concrete crushing</td>
<td>GSS bar pullout</td>
<td>CFRP jacket crack</td>
<td>East bar fracture</td>
<td>GSS bar pullout</td>
<td>West and east bar fracture</td>
</tr>
<tr>
<td><strong>Displacement ductility</strong></td>
<td>9.9</td>
<td>6.8</td>
<td>3.1</td>
<td>5.1</td>
<td>5.6</td>
<td>4.9</td>
<td>7.1</td>
</tr>
</tbody>
</table>

**Description of Bond-slip**

Research has been conducted on the bond-slip between steel bars and concrete and several bond-
slip models have been proposed \([14-15]\). Zhao and Sritharan \([7]\) proposed a modified steel stress-slip bar model incorporating bond-slip in OpenSees \([15]\), which could be implemented through the Zero Length Section Element \([7]\). Considering bond slip relationships provided in a model code \([16]\), Braga et al. \([17]\) developed a modified steel bar model. After the plastic hinge length is determined, the modified steel stress-slip curve is converted to a modified steel stress-strain curve, which could be used as input in the section properties. Only residual bond strength, instead of a full bond stress-slip curve, was considered in the modified steel bar model.

Two analytical models, Model Fiber and Model RS, are proposed in this section considering the bond-slip effect, damaged steel, and cracked concrete. Based on previous research, five-level damage states (DS) were used to evaluate the damage state of the original specimens \([6]\). The damage states used are: flexural cracking (DS-1), first spalling with possible shear cracking (DS-2), extensive cracking and spalling (DS-3), visible lateral and/or longitudinal bars (DS-4), and initiation of core damage indicating imminent column failure (DS-5). Specimen CIP-O had reached a damage state designation of DS-5 due to the severe concrete crushing and longitudinal bar fracture and buckling; repaired specimen PC1-R1 reached DS-5 with 1/8 in. (3.2 mm) wide cracks revealed after removal of the repair concrete; PC2-O also reached DS-5 due to the fact that all longitudinal bars were severed.

Considering the measured strain of longitudinal steel bars in the damaged specimens, modified steel properties of the repaired columns were used to account for the Bauschinger effect. The slope of the first branch was taken as a fraction of the steel modulus of elasticity; modification factors equal to 0.4, 0.6, and 0.6 times the original steel modulus of elasticity were used for CIP-R, PC1-R2, and PC2-R, respectively.

**Model Fiber**

Distributed plasticity considering bond slip is assumed to be concentrated in a plastic hinge length of the nonlinear beam-column element instead of the total length of the element; this was implemented using the BeamWithHinges \([16]\). The plastic length, \(L_{pl}\), can be defined based on the damaged region observed from tests or empirical relationships from the literature. The analytical model based on this element is referred to as Model Fiber in this paper. In addition, fatigue material was used to consider accumulation of damage of the longitudinal steel bars \([17]\).

A schematic of Model Fiber for the rehabilitated column is shown in Figure 2(a). The octagonal column was approximated by a circular section of equal cross-sectional area to simplify the sectional discretization process. There were 40 circumferential subdivisions for the core concrete, cover concrete and CFRP confined concrete, and 20, 5, and 10 radial subdivisions for the core concrete, cover concrete, and CFRP confined concrete, respectively.

The one-dimensional bond-slip model, as shown in Figure 2(b), is modified based on recent research \([8]\). In this bond-slip model, the bond strength-slip relationships for steel in confined and unconfined concrete were obtained from the CEB-FIB model (CEB-FIB 2010). The total deformation of the steel bar, \(\Delta s\), including both elongation and slip was obtained. The strain, \(\varepsilon\), was calculated based on Equation 1.
\[ \varepsilon = \frac{\Delta s}{L_{pt}} \]  

where \( L_{pt} \) is the plastic hinge length. The modified steel stress-strain curve with consideration of initial damage and bond-slip is thus obtained, which was used for the steel bars in the plastic hinge region.

**Model RS**

In Model RS, concentrated plasticity was considered using a non-linear moment rotational spring located at the repaired cross-section. A sectional moment-curvature analysis was performed, based on damaged steel properties and considering bond slip, to obtain the moment-rotation relationship, which is assigned to the non-linear rotational spring. The model with a rotational spring, is referred to as Model RS, and is schematically shown in Figure 3.
For this nonlinear rotational spring, a moment-rotation curve can be considered as the input, as shown in Figure 4 [20-21]. The *Hysteretic* material in *OpenSees* is used to represent moment-rotation relationships. The moment-rotation at the peak point was derived from sectional analysis considering bond-slip.

![Backbone curve for hysteretic rotational spring](image)

**Figure 4.** Backbone curve for hysteretic rotational spring (after Haselton et al. 2016).

**Analysis Results and Comparisons**

The results for CIP-R, PC1-R2, and PC2-R from Model Fiber and Model RS compared to the experimental results in terms of hysteretic response, hysteretic energy, and moment-rotation relationship are shown in Figures 5-10.

![Hysteretic response of CIP-R](image)

**Figure 5.** Hysteretic response of CIP-R: (a) Model Fiber and test; (b) Model RS and test.

Both Model Fiber and Model RS satisfactorily predicted the backbone curve and hysteretic energy. For specimen CIP-R, Model Fiber is better than Model RS at capturing the pinching behavior of the hysteresis, as shown by comparing Figure 5(a) to 5(b). For specimen PC1-R2 and PC2-R, Model RS is better than Model Fiber for matching the hysteresis curve obtained from the test, as shown by comparing Figure 7(a) to 7(b), and Figure 9(a) to 9(b).
Regarding moment-rotation, as shown in Figures 6, 8, and 10, at the repaired section, the moment-rotation curves from the test were stopped being measured after the peak moment reached. The calculated results for Model RS and the output results from Model Fiber not only matched well with the experimental results, but also predicted the results after softening.

Figure 6. Hysteretic energy and moment-rotation of CIP-R: (a) hysteretic energy at each drift ratio; (b) moment-rotation results.

Figure 7. Hysteretic response of PC1-R2: (a) Model Fiber and test; (b) Model RS and test.

Low-cycle fatigue of column longitudinal steel bars in Model Fiber is also predicted; in Model Fiber of specimens PC1-R2 and PC2-R, extreme longitudinal bars fractured due to the low-cycle fatigue, which is the consistent with the experimental results [12-13].
Figure 8. Hysteretic energy and moment-rotation of PC1-R2: (a) hysteretic energy at each drift ratio; (b) moment-rotation results.

Figure 9. Hysteretic response of PC2-R: (a) Model Fiber and test; (b) Model RS and test.

Figure 10. Hysteretic energy and moment-rotation of PC2-R: (a) hysteretic energy at each drift ratio; (b) moment-rotation results.
For the analysis of the specimen with pinching and bond-slip due to debonding between the column and repair system, Model Fiber would be more appropriate than Model RS with a concentrated plasticity spring for simulating the structural behavior. For structures without significant bond-slip and debonding, Model RS with a concentrated plasticity spring would be preferable to Model Fiber.

Summary and Conclusions

A seismic repair method was developed for three severely damaged bridge columns. The damaged specimens suffered bar pullout, buckling and fracture of reinforcing bars, and considerable concrete cracking and crushing. Epoxy injection was used to fill the cracks for one of the severely damaged specimens; a steel collar was provided to increase the bond between the original column and the repair donut. Based on the results of this research the following conclusions can be made:

1. A CFRP shell and epoxy anchored headed steel bars were used to relocate the column plastic hinge above the repair section; the repair method successfully restored strength capacity, displacement capacity, and energy dissipation. The steel collar strengthened the bond between the original column and repair concrete.

2. A forced-based beam-column element distributing plasticity over a certain plastic hinge length was used in Model Fiber with consideration of damaged steel properties and bond-slip effects. The low-cycle fatigue of steel bars was also included to match the experimental results and this model was effective in representing the cyclic behavior of the specimens.

3. Model RS using plasticity and a concentrated spring element with a simplified moment-rotation relationship to represent the structural behavior of the repaired specimens was also effective.

4. Both analytical models reproduced the experimental results of the repaired specimens adequately in terms of hysteretic response and energy dissipation. The proposed analytical models reproduced the local results, such as moment-rotation relationships, matched the global experimental results very well, and could be used for predictive purposes.

5. Model Fiber performed better than Model RS for matching the seismic performance of structures with a pinching hysteresis curve; Model RS performed better than Model Fiber for matching the hysteresis curves of structures with very good bond conditions.

Acknowledgements

The authors would like to acknowledge the financial support of the Mountain Plains Consortium under contract MPC-491. In addition, they would like to acknowledge the support of MJ Ameli, Mark Bryant, Priyank Sankholkar, Trevor Nye, Anurag Upadhyay, Joel Parks, and Tran Quang Duc, and Ryan Barton of the Department of Civil and Environmental Engineering at the University of Utah.
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