CENTER FOR TRANSPORTATION INFRASTRUCTURE AND SAFETY

Numerical Simulation of CFRP-Repaired Reinforced Concrete Columns

by

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16. Abstract

The overarching goal of this study was to investigate the influence of repair to individual reinforced concrete bridge columns on the post-repair seismic performance of the bridge system. A method was developed to rapidly repair an earthquake-damaged RC column with fractured longitudinal reinforcement using externally bonded carbon fiber reinforced polymer (CFRP) sheets. Test results showed that the lateral strength and drift capacity of the column were partially restored. This report presents the results of the first phase of this study in which a method was developed to model the repaired RC bridge column, and models of the undamaged (original) and repaired columns were validated with the experimental results. Nonlinear fiber element models were developed using Open System for Earthquake Engineering Simulation (OpenSees) software to simulate the response of the undamaged and repaired columns. The undamaged column was modeled using currently available techniques, while a technique was developed to model the repaired column. Analytical results were validated with experimental results. In the second phase of this work, the developed models will be implemented in a model of a prototype bridge structure to investigate the post-repair seismic response of the bridge structure. This study was sponsored by the University of Missouri Research Board and the National University Transportation Center at the Missouri University of Science and Technology in Rolla, Missouri.

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ABSTRACT

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1. INTRODUCTION

An extensive number of studies have been conducted on seismic repair and retrofit of reinforced concrete (RC) bridge columns, considering that they are the primary source of energy dissipation for a bridge structure during an earthquake. Seismic retrofit is conducted for RC bridge columns constructed in the U.S. prior to 1970s since they are not detailed to resist seismic loads. Methods commonly used to retrofit RC bridge columns include applying RC jackets [1], steel jackets [2], or fiber reinforcement polymer (FRP) composite jackets [3]. More recently, efforts have been focused on detailing of RC bridge structures to prevent collapse during an earthquake. RC bridge columns are designed to undergo cracking, spalling or crushing of concrete, yielding or bucking of reinforcing bars, or even fracture of some of the reinforcing bars during a strong earthquake. Repair techniques for earthquake-damaged RC bridge columns typically involve epoxy injection into concrete cracks [4], repair of spalled and crushed concrete, and/or application of jackets as external reinforcement. Similar to retrofit of RC bridge columns, reinforced concrete [5], steel [6], and FRP [7] are commonly used as jacketing materials for repair of RC bridge columns with different damage levels.

Local modifications (interventions) from the retrofit or repair of an individual RC column member can change the performance of the member, which in turn can influence the performance of the bridge structure in which the column is included, especially under seismic loading. In general, the seismic performance of a bridge structure will be improved when the retrofit or repair is carried out uniformly for all the members. Modifications to a single member or only some of the members of a bridge structure, on the other hand, may result in a stiffness irregularity, which can result in an unbalanced seismic demand on the members of the structure. To date, most research on seismic repair or retrofit of RC bridges has focused on assessing the response of individual columns (member level), not the bridge structure (system level), due to limitations in modeling and especially testing of full bridge structures. Thus, the need exists to develop techniques to model the response of the repaired column so that the effects of the intervention on the entire bridge structure can be determined. The availability of increasingly powerful computers has provided an opportunity to implement numerically intensive modeling strategies. In particular, analytical tools based on the fiber element method have been developed to model the nonlinear behavior of RC structures under cyclic loading, and studies have shown that the fiber element method can be effective in simulating the response of RC members under seismic loading [8-10].

The overarching goal of this study was to investigate the influence of repair to individual reinforced concrete bridge columns on the post-repair seismic performance of the bridge system. A method was previously developed by the authors to rapidly repair earthquake-damaged RC bridge columns using externally bonded carbon FRP (CFRP) sheets with fibers oriented in both the column transverse and longitudinal directions [11-13]. Five severely-damaged 1/2-scale RC columns with different damage conditions were repaired using the developed repair method. As discussed in the work by He et al. [12], the repair method proved effective in repairing damaged columns without fractured longitudinal bars, though factors such as bending-torsion interaction and failure mode played a role in the level of restoration. However, the method was only partially successful in repairing a column with fractured longitudinal bars located near the base of the column, in which case a large force demand was required for the CFRP strengthening system, as well as a substantial anchorage system to develop it. This report presents the results of the first phase of this study in which a method was developed to model repaired RC bridge columns, and models of the undamaged (original) and repaired columns were validated with the experimental results. In the second phase of this work, the developed models will be implemented in a model of a prototype bridge structure to investigate the post-repair seismic response of the bridge structure.

2. MODELING OF RC BRIDGE COLUMNS

The analytical models for both the undamaged (original) and repaired columns were described in this section. Open System for Earthquake Engineering Simulation (OpenSees) software was utilized in this study. Currently available techniques were used to model the undamaged column, while a technique was developed to model the repaired column. The developed models were validated by comparing the calculated responses with measured test data from different studies [11, 12, 14]. The original column test specimen was tested to failure under quasi-static reversed cyclic lateral load and a constant axial load of approximately 150 kips (667 kN) (7% of the axial load capacity) [14]. The column was then repaired and retested under the same load protocols [11, 12].

2.1 Modeling of Original Column

2.1.1 Fiber Section Properties

The original column section was constructed as a fiber section object, which is composed of fibers, with each fiber containing a prescribed uniaxial material, an area, and a location. The details of the column geometry and reinforcement are shown in Figure 1 and are discussed in detail elsewhere by the authors [11, 12]. The fiber discretization of the cross-section is shown in Figure 2. The core concrete was discretized to 25 strips in both directions. The cover concrete was discretized to 25 strips along the edge direction and two strips in the thickness direction. For the longitudinal reinforcing steel bars, the analysis was based on one mesh size. The core concrete, cover concrete, and longitudinal steel fibers were each defined by a uniaxial stress-strain model corresponding to the material they represent.

The Linear Tension Softening Concrete02 material in OpenSees was used to model both the unconfined and confined concrete. Mander's model [15] was used to determine the material properties of the confined concrete. The compressive stress-strain relationship of this material model is based on the uniaxial Kent-Scoff-Park concrete material model [16, 17]. The tensile stress-strain relationship is bilinear with the same modulus as the compression stress-strain relationship in the increasing region.

The reinforcing steel is modeled using the Giufre-Menegotto-Pinto constitutive model [18] available in OpenSees. The model has a bilinear backbone curve with a post-yield stiffness proportional to the modulus of elasticity of the steel, $E_{sh}=b \cdot E$, and accounts for the Bauschinger effect in the cyclic response of the material. Despite the simplicity of the model, it does not account for the yield plateau of the reinforcing steel or the degradation of the steel strength due to bar buckling or rupture.

Moment-curvature relationship from the fiber section was compared to the measured data from the experiment as shown in Figure 3, which illustrated the effectiveness of the discretization scheme with the chosen material models.

2.1.2 Column Numerical Model

The numerical model developed for the original column is illustrated in Figure 4. The column member was modeled as a nonlinear beam-column element with a fiber discretized section shown in Figure 3. For a RC column subjected to a lateral load, it is well established that the total lateral deflection can be attributed to deformations due to flexure, shear, and bond slip [19]. In this model, the shear and bond slip deformations were considered by adding zero-length springs.

The equation proposed by Correal et al. [20] was used to calculate the shear stiffness of column in the zero-length spring for shear:

$$K_v = \frac{K_{v,45}}{n_{pr} L_{pz}} \tag{1}$$

where n_{pr} is the number of plastic hinge regions (1 for cantilever columns), and L_{pz} is the length of each plastic hinge zone. L_{pz} was estimated as 1.5 times the column cross-section dimension based on Caltrans [21]. $K_{v,45}$ is the shear stiffness of RC members with 45° diagonal cracks, which was computed by Equation 2 [22]:

$$K_{v,45} = \frac{\rho_v}{1 + 4n\rho_v} E_c b_w d \tag{2}$$

where ρ_v is the transverse reinforcement ratio calculated as A_v/sb_w, and n is the modular ratio calculated as E_s/E_c, A_v is the transverse reinforcement area, s is the tie pitch, E_s is the elastic modulus of steel, E_c is Young's modulus of concrete, and b_wd is the web area to resist shear.

The shear stiffness calculated by Equation 1 was converted to an equivalent rotational stiffness due to difficulties in achieving numerical convergence in dynamic analysis. Equation 3 [23] was used to determine the equivalent rotational stiffness:

$$K_{v\theta} = \frac{K_v H^2}{n_{pr}} \tag{3}$$

in which H is the column height, and the other parameters were defined in the previous equations.

To consider the bond slip from strain penetration effects, the bond-slip spring model [24] was added to the model. In their model, the relationship of bar stress versus loaded-end slip was proposed as a linear relationship for the elastic region and a curvilinear relationship for the post-yield region. The curvilinear relationship was represented by Equation 4:

$$\tilde{\sigma} = \frac{\dfrac{\tilde{s}}{\mu - s}}{\left[\left(\dfrac{1}{\mu \cdot b}\right)^{R_e} + \left(\dfrac{\tilde{s}}{\mu - \tilde{s}}\right)^{\mathrm{R}_e}\right]^{1/R_e}} \tag{4}$$

where $\tilde{\sigma}$ is the normalized bar stress defined as $\tilde{\sigma} = \left(\sigma - f_y\right)/\left(f_u - f_y\right)$, $\tilde{s}$ is the normalized bar slip as defined as $\tilde{s} = \left(s - s_y\right)/s_y$, μ is the ductility coefficient defined as $\mu = \left(s_u - s_y\right)/s_y$, b is the stiffness reduction factor that represents the ratio of the initial slope of the curvilinear portion at the onset of yielding to the slope in the elastic region, f_y and f_u are the yield and ultimate strengths of the steel reinforcing bars, respectively, s_y and s_u are the loaded-end slips when the bar stresses are f_y and f_u, respectively, and the value of factor R_e should be slightly greater than one in order to maintain a zero slope near ultimate strength of the bar.

The bond-slip rotation can be assumed to occur about the neutral axis of the column cross-section at the connection interface [25]. The neutral axis location and the stress in the extreme tension reinforcement corresponding to the desired lateral load are determined from moment-curvature analysis of the section. The rotation occurring at the interface was obtained as the ratio between the slippage [24] and the distance from the extreme steel bar to the neutral axis. Therefore, the relationship between the applied moment and rotation was developed, which was then applied in the analytical model as a zero-length spring.

2.1.3 Model Validation

Both pushover and cyclic loading analysis were conducted using the developed analytical model of the original column. Axial load was applied along the axis of the column linearly up to 150 kips (667 kN) prior to application of the lateral load and then kept constant during the loading process. Results were validated through comparison of the measured and calculated load-displacement relationships. Figure 5a shows the measured envelope of load-displacement results and the calculated pushover results, in which the effects of shear deformation and strain penetration were included in different combinations. It can be seen that the shear deformation is negligible compared to the flexural deformation since the aspect ratio of the column (6.0) was relatively large [26]. The calculated pushover curve of the model with shear deformation and strain penetration implemented was comparable to the envelope of measured data in terms of initial stiffness and base shear capacity. However, the model could not predict the failure of the column associated with fracture of longitudinal bars due to limitations of the steel material model. Figure 5b shows the comparison of calculated and measured hysteresis behavior of the original column. The model predicted results very close to the measured data in terms of the base shear capacity and initial stiffness. However, the model could not well predict the degraded unloading stiffness and pinching effect.

2.2 Modeling of Repaired Column

2.2.1 Damage Prior to Repair and Repair Program

Figure 6 shows the damaged column after the original test. Damage included cracking and spalling of concrete, yielding and straightening of the end hooks in the reinforcing steel ties, and buckling of ten of the twelve longitudinal bars. Additionally, two longitudinal reinforcing bars fractured near the base of the column on opposite corners. The damaged column was repaired by removing and replacing the crushed concrete, and then installing three layers of CFRP sheets on the tension faces of the column with fibers oriented in the longitudinal direction of the column. Then, CFRP was wrapped transversely around the column with a varying number of layers to a height of 60 in. (1524 mm) from top of footing. Above this height, no longitudinal or transverse CFRP was placed, and no repair was made to the concrete. Additional details regarding the damage description and repair of the original column are discussed elsewhere by authors [11, 12].

2.2.2 Column Numerical Model

Unique challenges exist for the case of modeling the behavior of repaired RC columns compared with undamaged or retrofitted RC columns. Several aspects complicate the simulation such as accounting for the initial damage condition and estimating the mechanical properties of the materials etc. In this study, a new modeling method was developed to simulate the behavior of the repaired RC column, in which prior damage and repair was accounted for according to different damage states and repairs along the column length.

It was illustrated in the study [23] that the reinforcing steel properties should be modified to account for column softening due to earthquake damage. In their study, the elastic modulus of the longitudinal bars was reduced to account for the Bauschinger effect due to the cyclic loading from the previous testing. Five column damage states were defined in their study: flexural cracks (DS1); first spalling and shear cracks (DS2); extensive cracks and spalling (DS3); visible lateral and longitudinal bars (DS4); and imminent failure (DS5). Different reduction factors were proposed to modify the elastic modulus of the longitudinal bars in repaired columns corresponding to the different damage states.

In modeling the repaired column in this study, the modified steel properties, the confinement provided by the CFRP wrap and the longitudinal CFRP in the repaired region, and the cracked concrete in the unrepaired region were considered. Determination of the damage states along the column length is illustrated in Figure 6d, which was used to determine the reduction factors employed for the longitudinal reinforcing bars. The repaired column member was modeled as a nonlinear beam-column element with a fiber discretized section as shown in Figure 7, in which different fiber sections were used to represent the different damage states and repairs along the length. In addition, the same shear stiffness used for original column was used in the repaired column model. Bond-slip deformations from the strain penetration effects were included in the analytical model, in which the damage to the pretested reinforcing bars was considered.

2.2.3 Model Validation

The calculated load-displacement relationship from the pushover analysis is compared to the measured data in Figure 8a. Results in Figure 8a illustrate that the developed model can simulate the initial stiffness and the lateral strength capacity of the repaired column with acceptable discrepancy. Figure 8b compares the measured and calculated hysteresis behaviors of the repaired column. The asymmetry of the measured data during testing is due to the unsymmetrical damage from the original testing. The calculated results of the developed analytical model are symmetric for the reason that the unsymmetrical unrepaired damage was not modeled. The behavior of the repaired column in the direction of positive displacement was well-predicted by the developed analytical model. Although the analytical prediction shows slightly larger energy dissipation capacity, good agreement in terms of both lateral strength and initial stiffness is observed. Moreover, pinching of the hysteresis loops observed in the experimental data is also reflected in the analysis.

3. CONCLUDING REMARKS

In the first phase of this study, a method was developed to model repaired RC bridge columns, and models of the undamaged (original) and repaired columns were validated with the experimental results. The original column was modeled with beam-column elements with fiber section and nonlinear springs incorporating effects of shear deformation and strain penetration. A new technique was developed to model the repaired column, considering the variation of cross-sectional properties along the length of the column depending on the varied damage and repair conditions. The developed column models were validated against corresponding measured data by pushover and cyclic analysis. In the second phase of this work, which is ongoing at this time, the developed models will be implemented in a model of a prototype bridge structure to investigate the post-repair seismic response of the bridge structure. Based on the results presented in this report, the following preliminary conclusions can be drawn:

1. The response of the original column can be predicted by conventional modeling methods with negligible discrepancy;
2. The new technique developed to model the repaired column can reasonably predict the performance of the repaired column.

4. ACKNOWLEDGEMENTS

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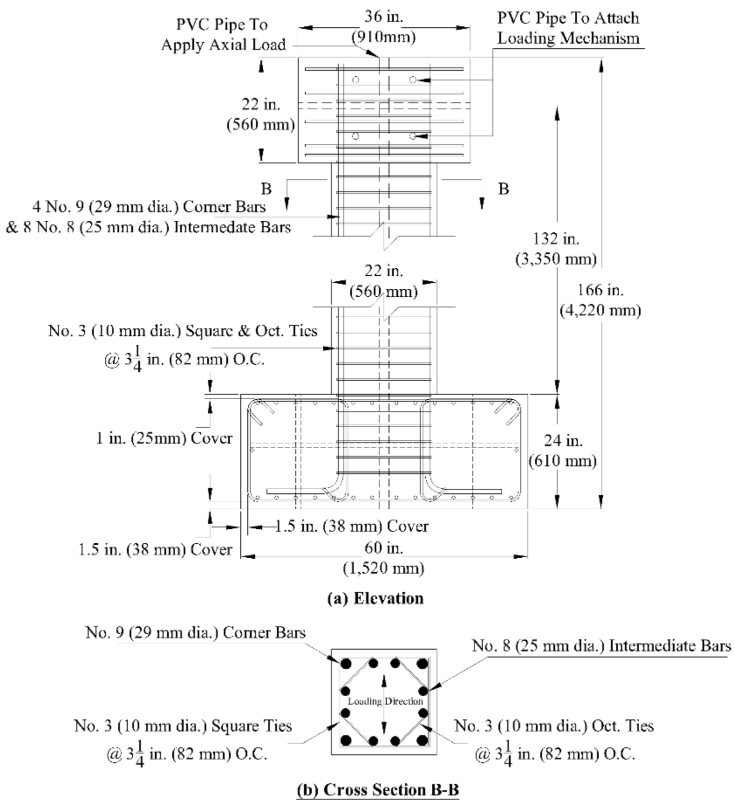


Cross-Section	22 in. × 22 in. (560 mm × 560 mm)
Height	132 in. (3350 mm)
Longitudinal Reinforcing Steel Bars	4 No. 9 (29 mm dia.) & 8 No. 8 (25 mm dia.) (ρ_l=2.13%) f_y=76 *ksi* (524 *MPa*) (No. 8) f_y=67 *ksi* (462 *MPa*) (No. 9)
Transverse Reinforcing Steel Bars	No. 3 (10 mm dia.) @ 3.25 in. (80 mm) (ρ_t=1.32%) f_y=74 *ksi* (510 *MPa*)
Concrete	f_c'=5 *ksi* (34.5 *MPa*)

Figure 1 Geometry and reinforcement details of original column

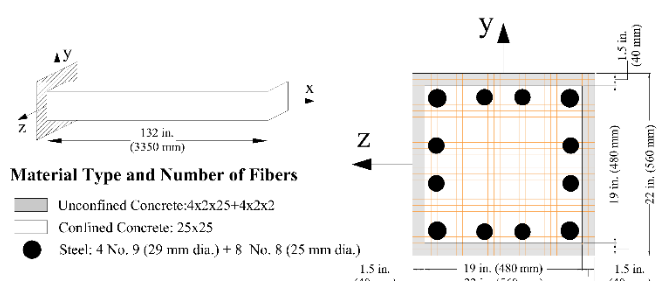


Figure 2 Fiber discretization of the cross-section

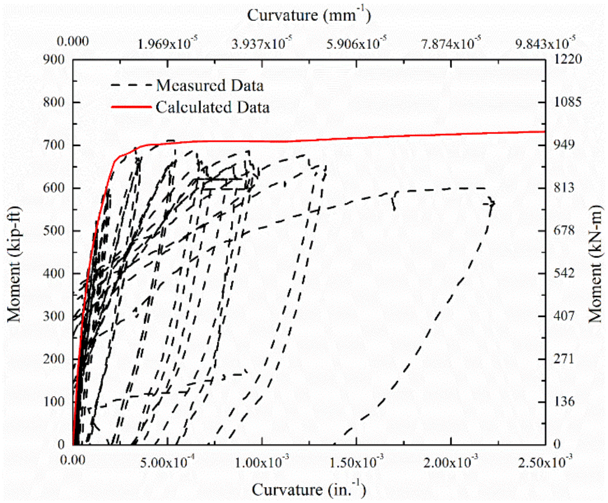


Figure 3 Comparison of measured and calculated moment-curvature relationships for original column

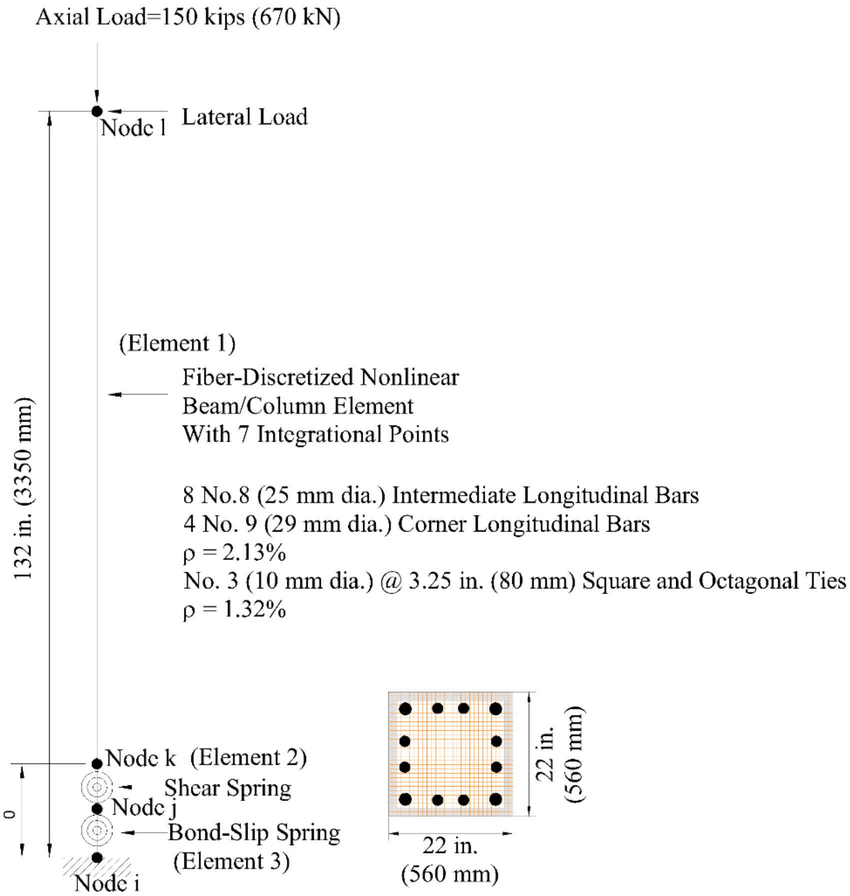
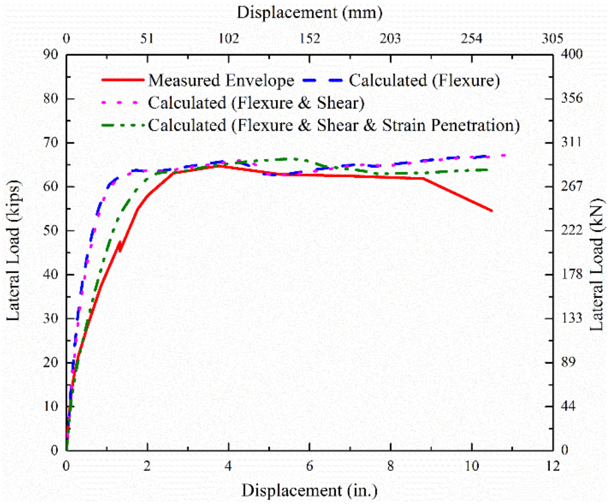
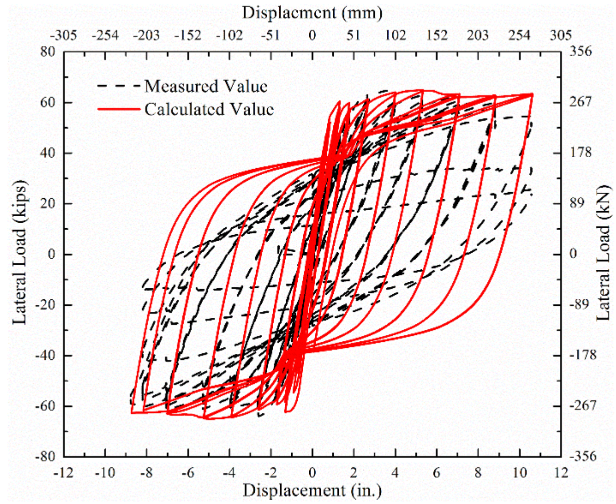


Figure 4 Numerical model for original column

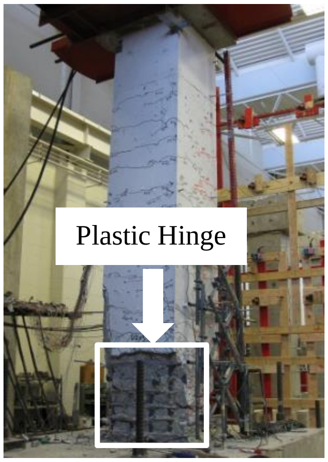


(a) Pushover analysis



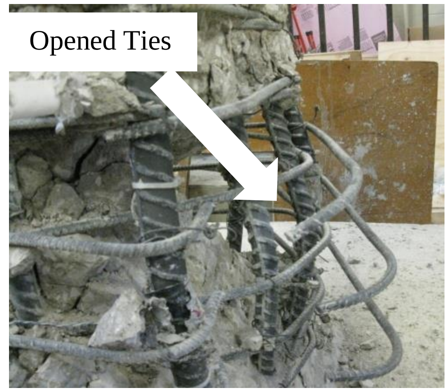
(b) Hysteresis analysis

Figure 5 Comparison of the measured and calculated response for original column



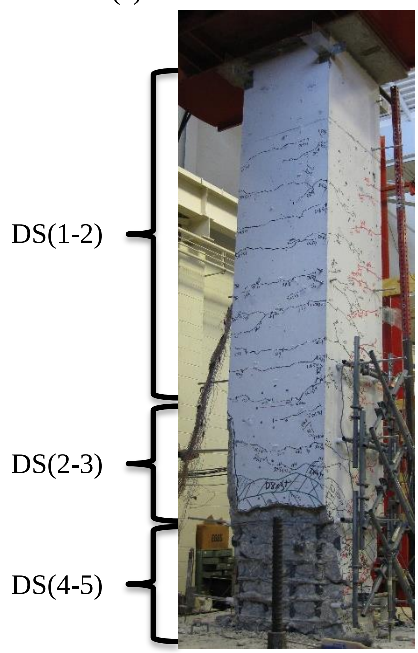
(a) Damaged column

(b) Longitudinal reinforcement rupture



(c) Failure of ties

DS1: Flexural cracks
DS2: First spalling and shear cracks
DS3: Extensive cracks and spalling
DS4: Visible lateral and longitudinal bars
DS5: Imminent failure



(d) Determination of damage states prior to repair

Figure 6 Damage to original column prior to repair

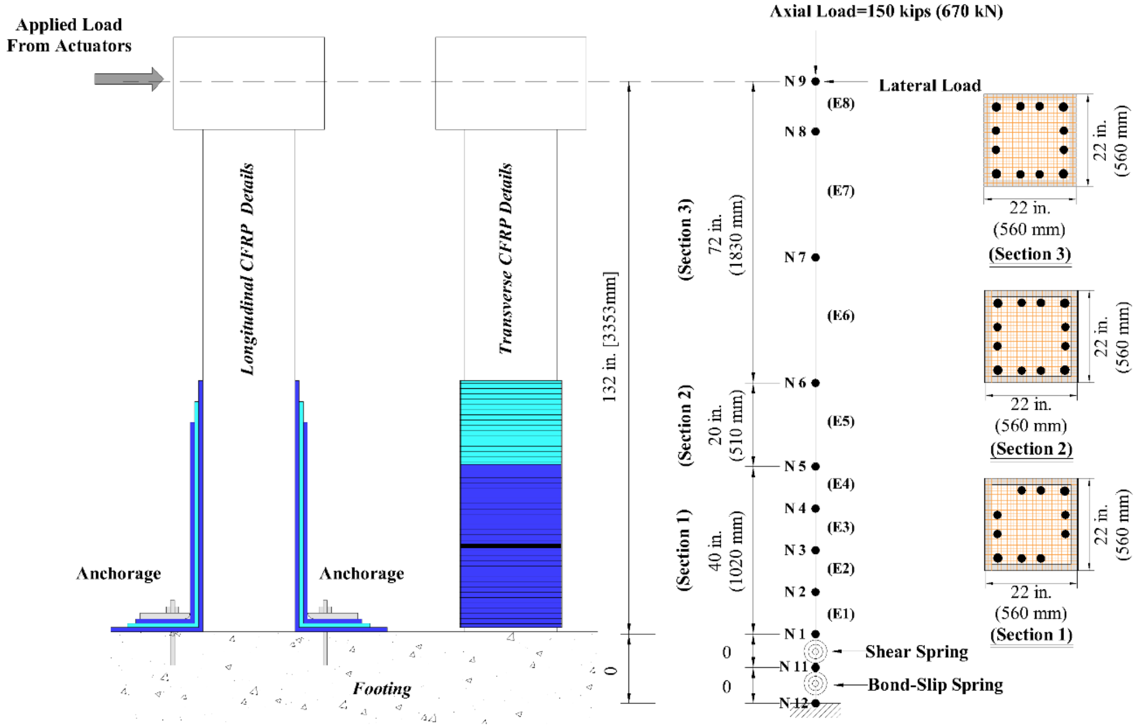
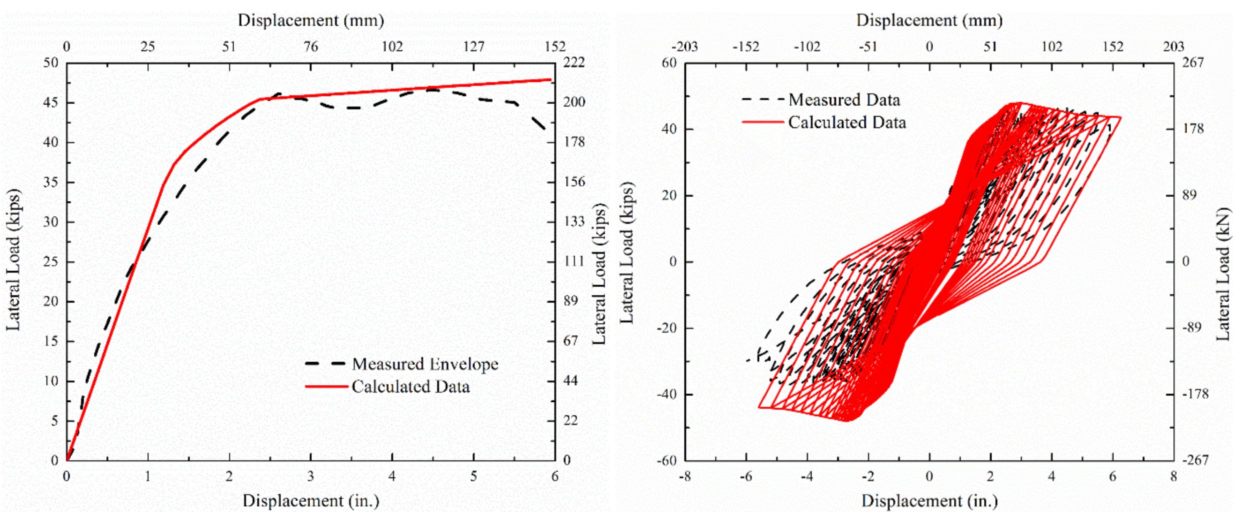


Figure 7 Numerical model for repaired column



(a) Pushover analysis (b) Hysteresis analysis

Figure 8 Comparison of the measured and calculated response for repaired column